Nigerian Research Journal of Engineering and Environmental Sciences 5(2) 2020 pp. 714-725

Nigerian Research Journal of Engineering and Environmental Sciences
Journal homepage: www.rjees.com

Original Research Article

Assessment of Road Pavement Failure Along Amuro-Okigwe Trunk Road, Imo State Nigeria

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ARTICLE INFORMATION

Article history:
Received 22 Sep, 2020
Revised 18 Oct, 2020
Accepted 29 Oct, 2020
Available online 30 Dec, 2020

Keywords:
Assessment
Road pavement
Failure
Amuro-Okigwe
Trunk road

ABSTRACT

This paper studied the geotechnical properties of soils along Amuro to Okigwe trunk road, Imo State Nigeria to assess the causes of incessant road pavement failure. Samples were collected from five boreholes and sub-grade soil 0.16 to 0.40 m depth range and subjected to soil tests using standard methods. Initial moisture content was 75.25%, plasticity index was 16.63% and colour indicated weak red which showed slight presence of organic matter and iron (III) oxide Fe_2O_3. In sieve analysis, uniformity coefficient (C_U=5.14) was less than 6 and while the coefficient of curvature (C_U=0.40) was below the specified range of 1 to 3. In consolidation test, coefficient of volume compressibility (M_v) was 4.4445×10^{-4} m^2/MN which is evidence of soil of very high compressibility. The coefficient of permeability (k) was 10.333×10^{10} m/s which indicates high soil permeability. The soil experienced compression of 1.546 mm at maximum load of 40 kg during loading process and swell of 0.456 mm after unloading process, which is typical of clay minerals. Shear strength of soil at 60 kg load was 163.5 kN/m^2 showing evidence of firm soil. In the compaction test, optimum moisture content was 19.22% corresponding to maximum dry density of 18.58 kg/m^3. California bearing ratio (CBR) was 28.8% against high quality sub-base value that range between 80% to 100%. It was concluded that pavement failure was due to poor geotechnical properties of soil since results from Atterberg limits, sieve analysis, consolidation and CBR tests results were below standard values for interfacing of engineering structures with soil.

1. INTRODUCTION

In Nigeria, road is one of the infrastructural facilities that is in total collapse (Kekere *et al*., 2012). Although, several factors might contribute to road failure, most materials such as laterite, aggregates and asphalt on which Nigerian roads are built may not be in harmony with highway subgrade specifications (Owoyemi and Adeyemi, 2012). Consequently, studies of pavement failure in some parts of southeastern Nigeria have been attributed to interaction of local road aggregate with water causing swelling, stripping and potholing (Abam *et al*., 2000). Poor construction, problematic soils, poor drainage and poor geotechnical properties of construction materials have been identified to be responsible for road failures in the southwestern parts of Nigeria (Jegede, 2000; Jegede, 2004; Ogundipe, 2008; Kekere *et al.*, 2012).

Deterioration of highway pavement is a very serious problem that causes unnecessary delay in traffic flow, distorts pavement aesthetics, damages of vehicles and most significantly, causes road traffic accident that had resulted in loss of lives and properties (Van der Merwe and Ahronovitz 2005). It is noted that, it is impossible to design a road pavement which does not deteriorate in some way with time and traffic, hence the aim of pavement structural design is to limit the level of pavement distress, measured primarily in terms of riding quality, rut depth and cracking, to predetermined values in (DSSRBW, 2001; AASHTO, 2002; Van der Merwe and Ahronovitz, 2005). Flexible pavements are intended to limit the stress created at the subgrade level by the traffic traveling on the pavement surface, so that the subgrade is not subject to significant deformations (LNEC for European Road Federation, 2006).

A variety of factors contributing to pavement deterioration were investigated by many researchers (Little *et al.*, 1995; Sebesta, 2002; Abhijit *et al.*, 2011; Okigbo, 2012; Omer *et al.*, 2014). On the other hand, lack of proper supervision of the construction, low quality construction materials, poor workmanship were the main causes of pavement failure attributed to construction (Okigbo, 2012). Fatigue cracking and deformations of pavements are the major defects caused by heavy traffic due to the increased traffic overloading which is more than the design load. As stated by Croney and Croney, (1998), deterioration of pavements arises from deformation generally associated with cracking under heavy commercial vehicles. However, the action of traffic continues to wear the surface texture and thus gradually reduces the high-speed skidding resistance (Oguara, 2010). The effect of rain on pavements can be destructive and detrimental as most pavements are designed based on a certain period of rainfall data. In addition, rainfall is a well-established factor affecting the elevation of the water table, the intensity of erosion, and pumping and infiltration (Zumwari, 2015).

Plasticity of soils is vital in classification of soils and hence can be used as a guide indicating how much a soil is likely to settle or consolidate under load. As plasticity is the most important characteristic of fine-grained soils in order to describe engineering behavior, classification of plasticity is differentiated as; < 4 for non-plastic, 4-7 for very low plasticity, 7-10 for low plasticity, 10-25 for medium plasticity, 25-50 for high plasticity, 50-150 for very high plasticity and > 150 for extremely high plasticity (Okkels, 2019).

Therefore, this paper studied the geotechnical properties of soil along Amuro-Okigwe trunk road in order to assess the causes of road pavement failure in this area.

2. MATERIALS AND METHODS

2.1. The Study Area

Amuro-Afikpo and its environs is located in Ebonyi State, South Eastern Nigeria. The study area is accessible by Enugu-Abakaliki expressway and some track roads and footpaths. The area is located within the lower Benue trough. Geomorphologically, the area is composed of sediments of the Upper Cretaceous and the formation encountered is the Ezeaku, which has both shale and sandstone and the Aus-River group which is composed of shale and intercalation of sands (Okoye, 2018). The igneous activity found in the area show a doleritic intrusion (silt) probably of the initial phase of the rift formation. The Amuro area offers a unique opportunity to study and understand the deformational processes and to determine the tectonic stresses active in the region. The study area lies between longitudes 7° 52 E and 7° 56 E, and latitudes 5° 53 N and 5° 58 N within the Afikpo syncline of the Cross-River basin of the Benue trough. It covers an area of 60 km^2 (sixty square kilometers). Major access roads into the area are through the Abakaliki - Afikpo Road, Okigwe - Afikpo Road, and Okposi - Amasiri - Afikpo Road. A good network of roads links up such areas as Abaomege, Akpoha, Ibi, Amasiri, Amuro and Afikpo. The major towns in the area are Amuro, Amasiri and Ozara ukwu. Figure 1 show the map of study area and sampling points as borehole numbers 1 to 5 (BH1 -

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BH5) at chainages (1 + 250), (2 + 375), (4 + 125), (5 + 450) and (7 + 250) respectively, one chainage being equivalent to 500.00 m, with 625 m, 750 m, 825 m and 800 m being the distances between random sampling points.

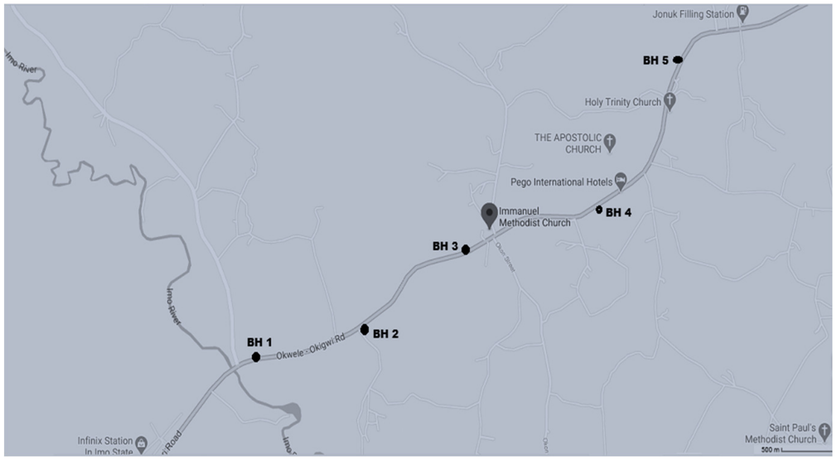

Figure 1: Map of study area

2.2. Sampling Techniques

The apparatus used in collecting the samples is the Spiral-Type auger. It consists of a flat thin metal strip, machine twisted to a spiral configuration of uniform pitch; having at one end, a sharpened or hardened point, with a means of attaching a shaft or extension at the opposite end. With this instrument, five samples were collected in each of the five boreholes along the asphalt road at chainages (1 + 250), (2 + 375), (4 + 125), (5 + 450) and (7 + 250), and was based on visual characteristics along the soil profile which are the various strata present at every borehole studied. Auger boring was made by rotating and advancing the desired distances into the soil. The auger was withdrawn from the hole and the soil removed for examination and tests. The empty auger was returned to the hole and the procedure was repeated. This sequence was continued until the required depth of 1.5 m was reached. Five samples were collected from five boreholes along the road at different chainages and were subjected to Atterberg limits tests. Sample with the highest plasticity index being the most critical was subjected to other tests such as the moisture content, colour test using the Munsell colour chart, particle size analysis, consolidation, shear strength, standard compaction and California Bearing Ratio (CBR) tests respectively. All tests were carried out in accordance with BS 1377 (1975) which contain the methods of test for soils for civil engineering purposes. The samples were stored in firmly closed polythene bags to prevent moisture loss before commencement of experiments.

2.3. Moisture Content Test

Two samples of the soil were placed in an oven and heated to a temperature of 105℃ until a constant weight was observed after two successive weighing and average initial moisture content (w) in per cent of soil was determined from Equation 1.

$$w = \frac{M_w}{M_s} \tag{1}$$

Where M_w is the mass of water and M_s is the mass of solids

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2.4. Atterberg limits tests

In the Atterberg limits tests, plasticity index was determined from the relationship;

$$PI = LL - PL \tag{2}$$

Where PI is the plasticity index (%), LL is the liquid limit (%) and PL is the plastic limit. Here if the field moisture is near the liquid limit, a lot of settlement is likely. The opposite is true if the field moisture is near or below the plastic limit.

2.5. Particle Size Analysis

In particle size analysis, particle size was plotted against per cent finer on a semi-logarithmic graph. Coefficient of uniformity, (C_U) and coefficient of gradation, (C_C) were computed from the relationships;

$$C_U = \frac{D_{60}}{D_{10}} \tag{3}$$

$$C_C = \frac{D_{30}^2}{D_{60} \times D_{10}} \tag{4}$$

Where:
D_{60} = Diameter corresponding to 60% finer in the grain size distribution
D_{30} = Diameter corresponding to 30% finer in the grain size distribution
D_{10} = Diameter corresponding to 10% finer in the grain size distribution
Using the Unified Soil Classification System (USCS), secondary descriptions (coarse grained), we have the under listed classifications
M = Silty: > 12% fines, PI < 4 or plots on or above "A" line
C = Clayey: > 12 % fines, PI > 7 and plots on or above "A" line
P = Poorly Graded: < 5% fines, $C_U < 6$ and/or $1 > C_C > 3$
W = Well Graded: $<$ 5% fines, $C_U \geq 6$ and $1 \leq C_C \leq 3$

2.6. Consolidation Test

Moisture content of soil was determined after compression using the procedure outlined in BS 1377 (1975), and the expressions for determination of soil consolidation parameters are presented in Equations (5) to (13). Result of moisture content was found to be 19.43%.
Specific gravity of soil Gs determined from test = 2.73, Thickness of sample, $H_0 = 20\ mm$
Void ratio at the end of test:

$$e_1 = w_1 G_s \tag{5}$$

Void ratio at start of test:

$$e_0 = e_1 + \Delta e \tag{6}$$

$$\frac{\Delta e}{\Delta H} = \frac{1+e_0}{H_0} = \frac{1+e_1+\Delta e}{H_0} \tag{7}$$

Coefficient of volume compressibility $M_v\ (m^2/MN)$ was determined from the expression:

$$M_v = \frac{1}{(1-e)} \frac{(e_0 - e_1)}{(\sigma_1' - \sigma_0')} \tag{8}$$

Where σ_0' to σ_1' is an increase in effective stress (kN/m^2) while e_0 to e_1 is the corresponding decrease in void ratio.

Compression Index (C_c) was determined as the slope of the linear portion of the $e - log\sigma'$ plot and is dimensionless. For any two points on the linear portion of the plot:

$$C_c = \frac{e_0 - e_1}{log\frac{\sigma_1'}{\sigma_0'}} \quad (9)$$

Coefficient of consolidation (c_v) was computed from:

$$c_v = \frac{0.848d^2}{t_{90}} \quad (10)$$

Where, d is half of the average thickness of the specimen for any particular load increment and t_{90} the time for which 90% of draining have taken place.

Coefficient of permeability, (k) was calculated from the relationship:

$$k = c_v m_v \gamma_w \quad (11)$$

2.7. California Bearing Ratio (CBR)

California bearing ratio was determined from the relationship:

$$\text{CBR} = (P_T/P_S) \times 100 \quad (12)$$

Where P_T = Corrected test load corresponding to the chosen penetration from the load penetration curve, and P_S the standard load for the same penetration.

3. RESULTS AND DISCUSSION

Figure 2 presented plots of moisture content against number of blows and the moisture content corresponding to 25 blows was the liquid limit for that sample. The results of the plastic, liquid limits derived from the plots and the plasticity indices for the samples are shown in Table 1. In all the Atterberg limits tests, borehole number 2 collected from chainage (2 + 375) was the most critical with highest plasticity index of 16.63%. The sample used was the one collected at depth between 0.16 - 0.40 m which is the subgrade soil. This soil was subjected to other tests such as moisture content, colour, particle size analysis, consolidation, shear strength, standard compaction and California Bearing Ratio (CBR). It can be seen that the soil falls under medium plasticity which ranges between 10-25% and subgrade of this nature has the tendency to collapse under traffic load (Taskiran, 2010).

Table 2 presented the soil colours across the soil profile which are the various strata present at any borehole location. Actually, soil color does not affect the behavior and use of soil (Osunade, 2007); however, it can indicate the composition of the soil and give clues to the conditions that the soil is subjected to (Osunade, 2007). Also, it does not deal with strength characteristics of soil but provides reliable information as to the likely distribution of pH, EC and organic matter content, indirectly via a soil pedon type assignment (Osunade, 2007). From Table 2, the first two profiles 0 - 0.40 m depth indicates the presence of oxidized ferric iron oxides in the soils. From 0.41 - 0.80 m in the third profile show a mixture of oxidized ferric iron oxides and organic matter content. In the fourth and fifth profiles, oxidized ferric iron oxides and organic matter were indicated. The presence of organic matter in this soil should be one of the reasons for frequent failure of Amuro pavement (Esmaeilzadeh and Ahangar, 2014).

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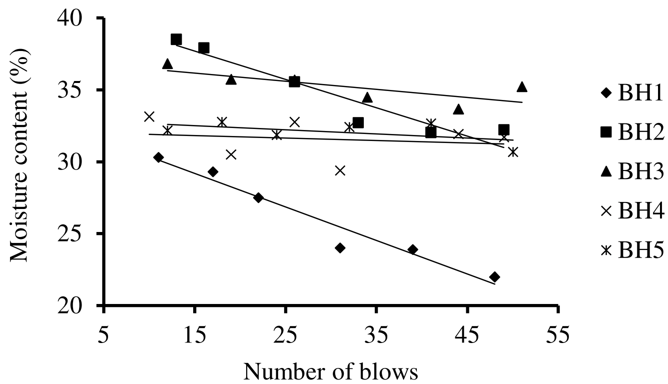

Figure 2: Liquid limit plots for borehole numbers (BHs) 1 to 5

Table 1: Atterberg limits test results from trial pits

Can No.	Borehole No. 1		Borehole No. 2		Borehole No. 3		Borehole No. 4		Borehole No. 5	
	30	6	1	19	35	55	3	11	35	64
Wt. of can + wet soil (g)	19.60	23.50	20.50	17.80	39.50	39.67	20.32	22.80	40.05	41.68
Wt. of can + dry soil (g)	17.30	20.40	18.20	15.60	36.60	36.09	17.90	19.78	37.50	38.50
Wt. of can (g)	8.60	9.50	7.25	7.10	27.50	25.10	7.30	8.23	26.90	26.30
Moisture content (%)	20.90	22.14	17.36	20.56	24.17	24.20	18.59	20.73	19.39	20.68
Plastic Limit (%)	21.52		18.96		24.19		19.66		20.03	
Liquid Limit (%)	26.80		35.59		35.52		31.61		31.44	
Plasticity Index (%)	5.28		16.63		11.33		11.95		11.41	

Table 2: Color across the soil horizon and predominant mineral contents (Borehole No. 2)

Profile	Depth from surface (m)	Hue	Value	Chroma	Munsell notation	Color Name	Remarks
1	0 - 0.16	10R	4	4	10R 4/4	Weak Red	Fe_2O_3
2	0.16 - 0.40	10R	5	4	10R 5/4	Weak Red	Fe_2O_3
3	0.41 - 0.80	2.5YR	2.5	4	2.5YR 2.5/4	Dark reddish brown	Fe_2O_3/organic Matter
4	0.81 - 1.40	5YR	4	6	5YR 4/6	Yellowish red	Fe_2O_3
5	1.41 - 1.50	7.5YR	5	4	7.5YR	Brown	Organic matter

Table 3 presented results of the moisture content tests. Average moisture content was found to be 75.25%, an indication of soil with very high water retention prone to intermittent swelling and shrinkage and soils with this characteristics are subject to collapse when interfaced with engineering structures.

Figure 3 show particle size distribution of the soil and particle size diameters at 10, 30, and 60% finer, in which D_{10}, D_{30} and D_{60} were 0.14 mm, 0.20 mm and 0.72 mm respectively. Results indicated that both coefficient of uniformity and gradation fall below standard since $C_U = 5.14 < 6$ and $C_C = 0.40$ which does not fall within the specified range $1 > C_C > 3$. This can be seen as one the reasons for pavement failure along Amuro town.

Table 3: In-situ moisture content

Sample no.	1	2
Can No.	28	31
Wt. of empty can(g)	25.11	26.72
Wt. of can + wet soil (g)	125.89	131.86
Wt. of can + dry soil (g)	82.70	86.63
Wt. of water (g)	43.19	45.23
Wt. of soil (g)	57.59	59.91
Moisture content (%)	74.99	75.50
Average moisture content (%)	75.25	

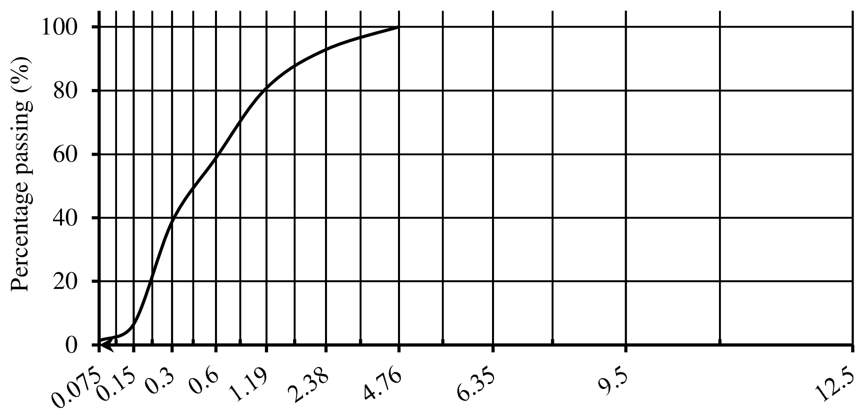

Figure 3: Particle size distribution of soil

The initial moisture content of soil before consolidation was 75.25%, so that per cent decrease in water content was 55.82%, and specific gravity of soil, Gs determined from test = 2.73

Thickness of sample, $H_0 = 20\ mm$

From Equation (5):

$e_1 = 0.1943 \times 2.73 = 0.530$

and from Equation (6):

Void ratio at start of test = $e_0 = e_1 + \Delta e$

Recall Equation (7):

$\frac{\Delta e}{\Delta H} = \frac{1+e_0}{H_0} = \frac{1+e_1+\Delta e}{H_0}$

$\therefore \ \frac{\Delta e}{1.546} = \frac{1.530+\Delta e}{20}$

$\Delta e = 0.13$

$e_0 = 0.53 + 0.13 = 0.66$

$\Rightarrow \ \frac{\Delta e}{\Delta H} = \frac{1 + 0.66}{20} = 0.083$

Therefore, the general relationship between Δe and ΔH is given by the expression;

$$\Delta e = 0.083\Delta H \tag{13}$$

Table 4: Derived values from consolidation test

Time (hrs)	0	24	48	72	96
Effective stress (kN/m^2)	0	24.98	49.96	99.92	199.85
Log P	0	1.3976	1.6983	1.9997	2.3007
Dial gauge ΔH (mm)	0.000	0.350	0.718	1.091	1.546
e	0.660	0.631	0.600	0.569	0.532

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Table 4 show derived data from consolidation test, plot of void ratio as a function of effective stress shown in figure 4 was used to compute coefficient of volume compressibility (M_v) which was found to be $4.4445 \times 10^{-4}\ m^2/MN$. This fall under highly organic alluvial clays and peat of very high compressibility with lower limit of $1.5 \times 10^{-6}\,m^2/MN$ (Carter and Bentley, 1991). The very high compressibility of this soil could be attributed to frequent collapse of the pavement after several reconstruction. Figure 5 which show variation of void ratio with logarithm of applied pressure, was used to derive the compression index. The value $(C_C = 0.051)$ did not fall within the specified ranges of soils as presented by Narendranathan and Lee, (2015) so that no inference can be deduced from the soil behavior from their work. On the other hand, the soil falls under homogenous sand between the range 0.05 to 0.06, while the average value of coefficient of permeability(k) was $10.333 \times 10^{-10}\,m/s$ typical of inorganic clays, silty clays and sandy clays of low plasticity (Swiss Standard SN 670 010b, 2013), prone to giving rise to pavement failure.

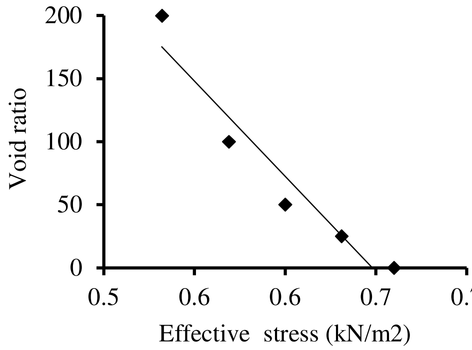

Figure 4: Variation of void ratio with effective stress

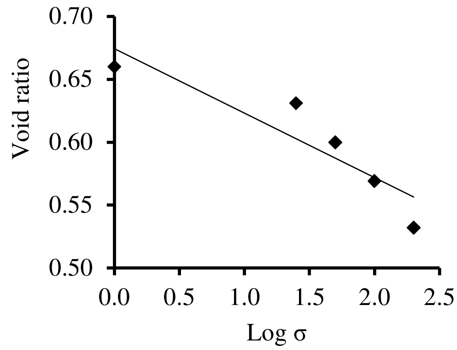

Figure 5: Variation of void ratio with log. pressure

$$C_c = \frac{e_0 - e_1}{log\frac{\sigma_1^1}{\sigma_0^1}} = 0.051$$

$$\therefore M_v = \frac{1}{1+0.631}\left(\frac{0.631-0.544}{140-20}\right) = 44445 \times 10^{-4}\ m^2/MN$$

Figures 6 to 9 plots dial gauge readings against time for 5 kg, 10 kg, 20 kg and 40 kg compression loads respectively. From these plots, the root of time for which 90% of draining have taken place, $\left(\sqrt{t_{90}}\right)$ for the respective compression loads were 8.0, 13.0, 8.5 and 5.25 minutes respectively. Average value of coefficient of consolidation, C_v was $0.72\,m^2/yr$ which fall between the range $0.51 - 8.2\ m^2/yr$, generally described as soft blue clay (CL - CH) with high permeability (Kozlowski and Ludynia, 2019). The high permeability of this soil should be instrumental to incessant collapse of Amuro pavement.

Figure 10 is a plot for consolidation test for unloading process which indicated the amount of consolidation that have taken place after all the loads were removed. On application of maximum load of 40 kg, the soil experienced total compression of 1.546 mm after 24 hours. The loads were removed in order of application from 40 kg to the least load of 5 kg load and it was observed that on removal of 5 kg, dial gauge reading was 1.090 mm after 24 hours indicating a swell of 0.456 mm which is characteristic of clay minerals. This implies that this soil tends to swell up during the rainy regime and shrink during the harmattan periods. This behavior does not favour soils meant for engineering purposes as frequent shrinkage and swelling would give rise to cracks and consequent road failure (Zumrawi et al., 2017).

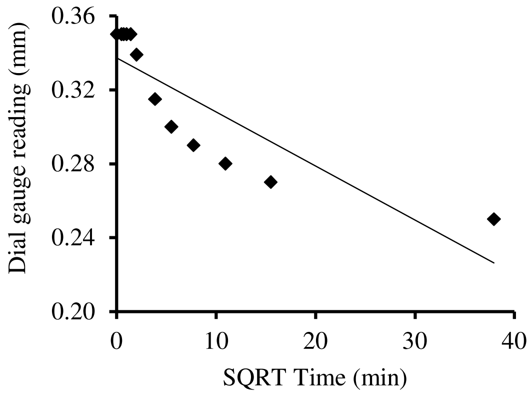

Figure 6: Variation of dial gauge reading with $\sqrt{t}$ for 5 kg compression load

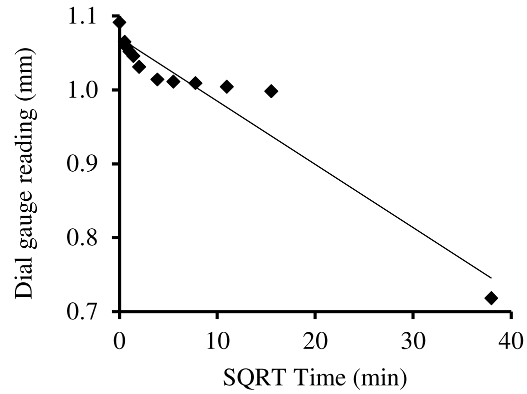

Figure 7: Variation of dial gauge reading with $\sqrt{t}$ for 10 kg compression load

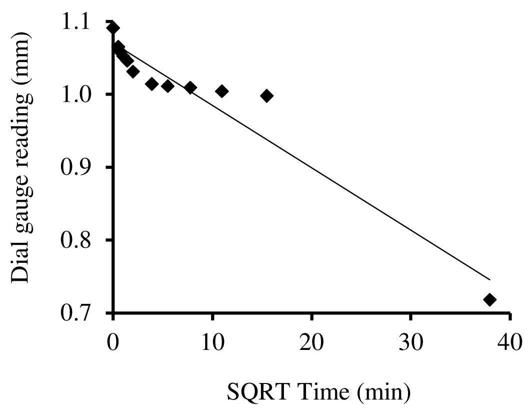

Figure 8: Variation of dial gauge reading with $\sqrt{t}$ for 20 kg compression load

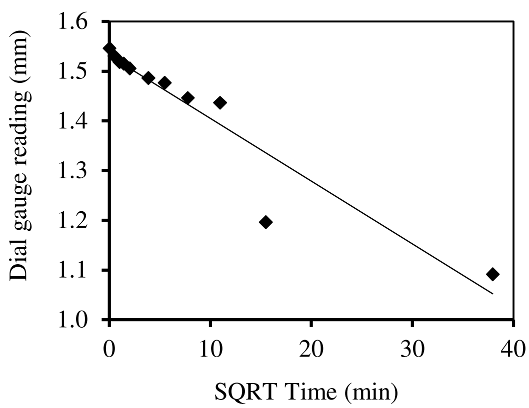

Figure 9: Variation of dial gauge reading with $\sqrt{t}$ for 40 kg compression load

Shear box test result is presented in Figure 11, From the plot of $\tau_f(\sigma_f')$, $\phi' = tan^{-1}\, 0.203 = 11.48°$ and $c' = 15.2\ kN/m^2$. Therefore, shear strength of soil is given by the expression $\tau_f = 15.2 + 0.203\sigma_f'$, giving a maximum shear stress of $48.39\ kN/m^2$ for normal stress of $163.5\ kN/m^2$ which corresponds to maximum of 60 kg load used in this study. This soil falls in the 40 - 75 kN/m^2 shear strength range, an indication of firm soils (Davison and Springman, 2000).

Compaction tests were conducted on soils prior to commencement of road construction. Figure 12 show variation of dry density as a function of moisture content and from the plot, optimum moisture content was 19.22% corresponding to maximum dry density of 18.58 kg/m^3. One could not say whether the contractor met this criterion during construction, otherwise it is a possible cause of pavement failure.

From Figure 13 and Table 5, the loads for 2.5 mm and 5.0 mm penetrations were 3.8 kN and 4.65 kN, their standard loads for the same penetrations were 13.2 kN and 20 kN respectively giving CBR values of 28.8% and 23.3%. CBR value at 2.5 mm penetration was 28.8% which was higher than CBR value for 5 mm penetration of 23.5%, therefore, the CBR value of this soil is 28.8%, a result which fall in the range 20 - 40 kN CBR corresponding to SM (coarse-grained soils) in the USC soil classification. High quality sub-base will have a value of between 80 - 100% maximum and based on this criterion, the road is prone to collapse (BS 1377, 1990).

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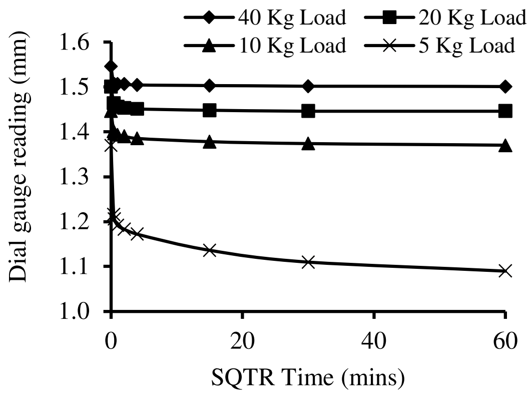

Figure 10: Plot for consolidation test for unloading process

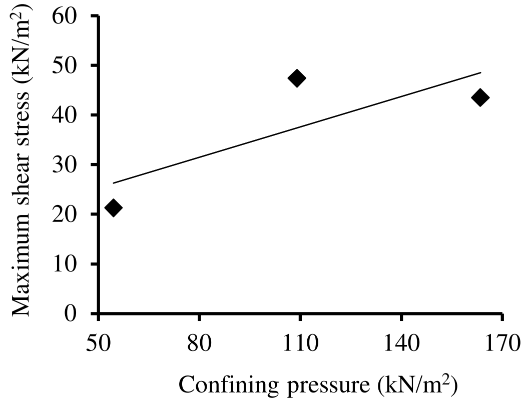

Figure 11: Plot of relationship between maximum shear stress and confining pressure from shear box test

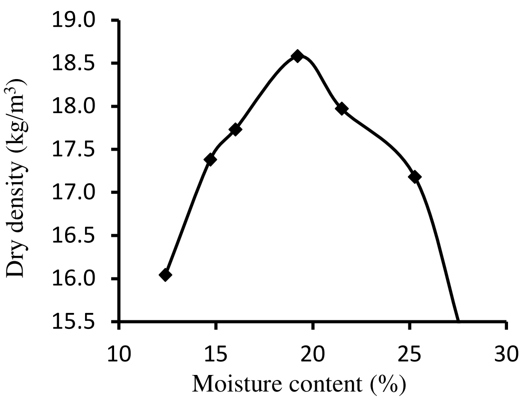

Figure 12: Variation of dry density with moisture content

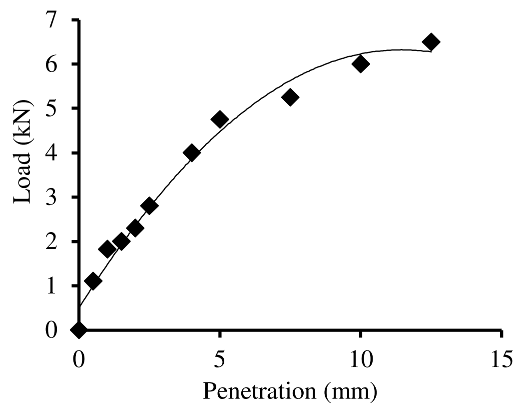

Figure 13: Typical CBR test result curves

Table 5: CBR results

Penetration (mm)	Load (kN)	Standard load (kN)	CBR (%)
2.5	3.8	13.2	28.8
5.0	4.7	20.0	23.5

4. CONCLUSION

From the field observations and results of laboratory tests conducted on the soil samples from five trial pits, it was concluded that some geotechnical properties of soil along Amuro-Okigwe trunk road do not meet with standard specifications for construction of durable pavement. This is evident from the soil colour which showed some presence of organic matter. Coefficients of uniformity and gradation did not satisfy specified standard as C_U was lower and C_C was below the range. Atterberg limits indicated soil of medium plasticity, not adequate for engineering structure to be interfaced with. Coefficients of volume compressibility, permeability and consolidation showed that the soil is characterized by shrinkage and swelling in dry and rainy seasons of the year, while the CBR does not meet up with standard for high quality sub-base thereby making the pavement prone failure.

5. CONFLICT OF INTEREST

There is no conflict of interest associated with this work.

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