

Research Article



Parameters Modeling and Probing of Highway Structural Deterioration: Case study of F-209 Segment of Ondo - Ore Pavement, Southwestern Nigeria

Olumuyiwa Olusola Falowo 

Department of Civil Engineering, Rufus Giwa Polytechnic Owo, Ondo State, Nigeria

Department of Applied Geology, Federal University of Technology, Akure, Ondo State, Nigeria

*Correspondence: solageo2018@gmail.com

Received: 09 February 2023 / Accepted: 19 June 2023 / Published: 28 June 2023

Abstract: The construction of highway route is influenced by geology, geotechnical, topography, and geomorphology of the terrain. However, for pavement that failed structurally, a critical investigation is required to ascertain the cause(s) of its failure, so that such study can assist during the rehabilitation/re-construction phase. Based on this, the incessant failed Ondo – Ore was studied using integrated methods. Electrical resistivity involving four vertical electrical sounding, ten dynamic cone penetration test, three coring by trial pits at 1.0 to 3.0 m offset from the edge of the highway at different chain-age, and laboratory geotechnical-geochemical analysis adopting standard procedures. The VES characterized the geological sequence within the highway alignment to be topsoil, subsoil, weathered layer, and basement rock. The topsoil/subsoil and the weathered layer on which the highway is founded is sandy clay and clay with resistivity less than 200 ohm-m. The depths to basement ranged from 33.5 to 45.1 m. The trial pits recordings are consistent with the results of the VES, geotechnical, and geochemical analysis, distinctly distinguishing the upper 1 m into clayey soil (sandy clay, clayey hardpan) laterite, and silt-clay-sand mixture. The engineering competence of the topsoil/subsoil on which the road is founded is poor, although is inactive SC-SM, A-7-5/A-7-6 lateritic soil type (silica-sesquioxide ratio of 1.67). The clay mineralogy is within the illite – montmorillonite group. The SNG, SN, and SNP contributions of the soil as subgrade, subbase, and base material are very low (<1.0). The regression models of all parameters correlated positively, although weak for RD and DCPI, in-situ CBR and MR, RD and in-situ CBR; while strong for soaked CBR and in-situ CBR, ER and MR. Consequently, based on the GI and CBR values, the expected average thickness of the highway should range between 191 mm (good segment) to 445 mm (for weak segment) (avg. 312 mm) which is far thicker than 274 mm measured along the highway alignment during reconnaissance survey. Therefore, it can be concluded that the failure of the highway is as a result of low soaked CBR/in-situ CBR values with low strength coefficient; and low design thickness across the highway. In addition, lack of drainage at the shoulders of the highway is also causative factor. Hence, the need for effective design of roads (to specification) and maintenance strategy was therefore advocated.

Keywords: Modulus of elasticity, trial pit, parameter modeling, electrical resistivity, penetrative index, highway alignment

INTRODUCTION

The dismal status of most developing nations' road networks is one of their distinguishing qualities, and Nigeria is no exception. The majority of roads in Nigeria are flexible pavement that responds to loads through elasticity and normal deformation. Pavement can be classed as rigid, flexible, or composite in terms of structural performance (Bell, 2007; Wright, 1986; Yoder & Witczak, 1975). A layered structure with minimal flexural strength is what a flexible pavement is. As a result, the external load is predominantly transferred to the subgrade through the lateral distribution as depth increases (Putra Jaya et al., 2021). Because of its low flexural strength, the pavement deflects briefly under force but returns to its normal level when the load is removed (Paige-green & Zyl, 2019). The pavement thickness is chosen in such a way that strains on the subgrade soil are kept within its carrying capacity and excessive deformations are avoided. This suggests that the subgrade is vital in a flexible pavement since it carries the vehicle loads transferred to it through the pavement (Bell, 2007; Kadyali & Lal, 2005). The strength and smoothness of the pavement surface are more dependent on the subgrade's permanent

deformation and resistance to such deformation. The flexural strength or beam strength (modulus of elasticity) of rigid pavement allows the slab to bridge over slight imperfections in the subgrade, subbase, or base upon which it sits. This means that the intrinsic strength of the slab itself must play a significant role in resisting the wheel load. As a result, while developing flexible pavement (the most frequent road type in Nigeria), special attention must be paid to subgrade strength and drainage, which have been identified as factors contributing to the failure of most Nigerian roads (Hopkins, 1994; Gudishala, 2004). Although design life, dependability, traffic variables, climate conditions, road geometry, and other elements are also important (Jegade & Olaleye, 2013; IRC, 2002; Obaje, 2017).

Ondo – Ore road is a Trunk-A highway built over 40 years, and one of the busiest highways in Ondo State linking central to the southern part of the State. The highway also links with Ore – Benin and Ore – Lagos highways which connects Edo and Lagos States respectively. Traffic survey (classified volume counts) conducted by taking records of all vehicles plying the highway per day in each direction, gave daily count of 450 – 600, hence the importance of the highway cannot be overemphasized. The road has contributed immensely to socio-economic development of all the towns it cut across. However, of recent, the incessant failures being experienced along the highway are becoming worrisome. The failed segments along the road alignment are becoming death trap for commuters, zones of traffic congestion, becoming large erosion spots, enhancing kidnaping and robbery, damaging or increasing faulty vehicles. All these culminate in adverse economic development of the areas in the highway domain. Many causes have been suggested or ascribed to the highway's collapse, including poor design and construction, excessive traffic, bad maintenance, inadequate roadway amenities, inadequate laboratory and in-situ tests, and the use of low-quality materials (Okigbo, 2012; Osulale et al., 2012; Owoseni et al., 2019; Okunlola et al., 2015).

However, based on a reconnaissance study and visual assessment, the road appears to meet some criteria (such as design and construction, use of quality materials, adherence to local standard of practice), except for inadequate laboratory and in-situ tests prior to highway construction, heavy traffic, poor highway amenities, and poor maintenance. Therefore, there's a need to investigate the shortcomings of the highway to mitigate such in the future. Hence, the primary goal of this research is to investigate the cause(s) of incessant highway failure along Ondo – Ore pavement, which is a segment of F-209 East – West road, southwestern Nigeria, by evaluating the in-situ soil index/engineering qualities by field surveys and laboratory experiments. The field survey includes an in-situ dynamic cone penetration test, groundwater table computation, trench and pit excavation, and electrical resistivity study. The laboratory investigation included geotechnical and geochemical investigations using relevant acceptable standards. The objectives of the study included identifying and classifying the subsoil (sequence) within the road alignment; evaluating the subsoil geological, geochemical, and geotechnical properties in relation to competence of the soil domain; determining the index properties of the soil; investigating any geological structure that could be inimical to the stability of the highway structure. In addition to this, empirical relationship or correlations and parameters modeling for the highway will be developed. The study's findings are likely to aid in determining acceptable re-construction design or repair for the highway. The composition of the methods adopted for this study (geophysical, geotechnical, geochemical, hydrogeological, and laboratory studies) from available literatures (Ademeso & Ogunjobi, 2021; Aderemi & Adeola, 2021; Adetoro & Abe, 2018; Ifarajimi et al., 2021) have been so productive with landmark success in highway investigation, and in solving a wide variety of engineering and environmental problems.

METHOD

Description of Study Area and Sample Locations

The Ondo - Ore highway is located within Ondo State, Southwestern Nigeria (Figure 1), connecting the central to the southern parts (Ore, Okitipupa, Ode Aye, Igbokoda) of the State, and to Lagos and Edo States. The road is about 40 km stretch, a segment of F-209. For the purpose of this study, the investigated segment starts from Ondo second gate close to Adeyemi College of Education of GPS coordinates: 701597 mE, 784114 mN) to Ore just after the flyover bridge (707967 mE, 745994 mN) of the highway. The highway is generally flat, as the highway descends in altitude towards Ore from Ondo axis. The highway falls within the tropical rainforest climate characterized by rainy and dry seasons. The rainy season starts in March to October, while the dry season commences in November and ends in February. The average annual rainfall and temperature are 1800 mm and 27 °C (Iloeje, 1981; Federal Meteorological Survey, 1982). The months of June and September usually experience heavy rainfall with relative humidity of about 80 %, although could be less than 50 % during the dry season (Iloeje, 1981).

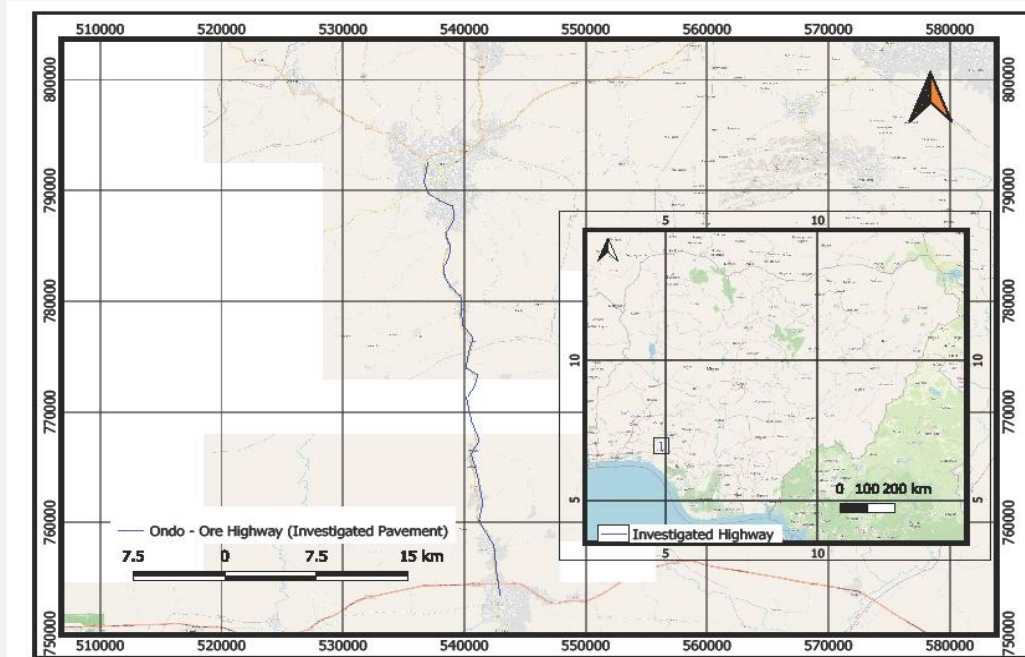


Figure 1. Map of the Studied Highway. Inset: Map of Nigeria showing the location of Akure – Ondo Highway in southwestern Nigeria

Geologically, the highway is underlain by Precambrian Southwestern Basement Complex (Figure 2) with migmatites, coarse porphyritic biotite granite, and shale - sandstone being the major rocks observed within the highway (Figure 3). The highway falls within the Ondo soil association type, which are weathered products of medium grained granites and gneisses, it is well drained, of medium to fine textured, orange brown to brownish red, fairly clayey soils overlying orange, brown and red mottled clay. Along the highway, no noticeable side drainage was observed, but the area is characterized by dendritic and trellised drainage systems.

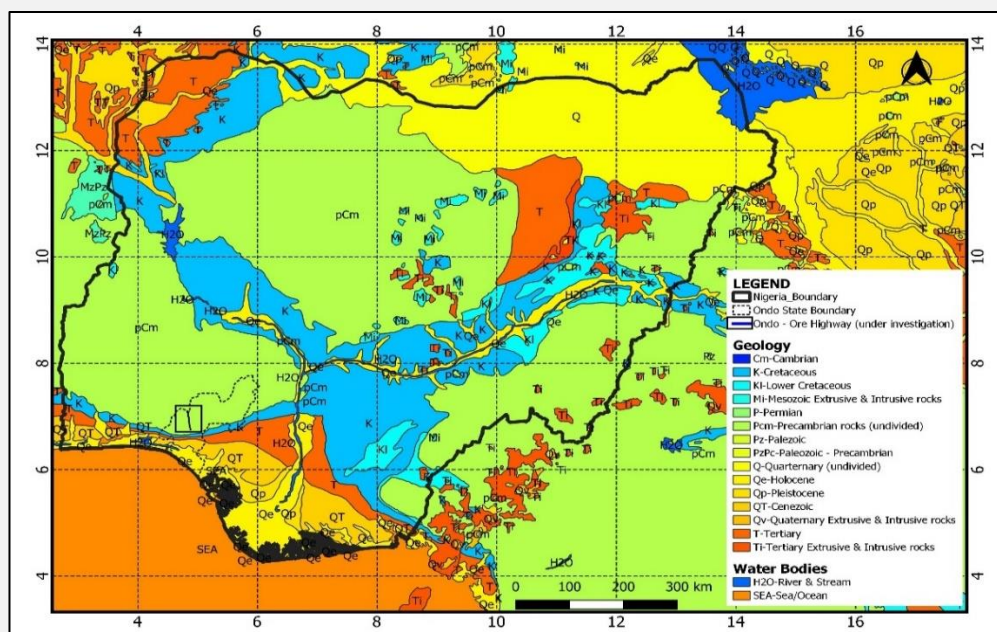


Figure 2. Geological map of Nigeria showing the highway under investigation (modified after Nigerian Geological Survey Agency, 1984)

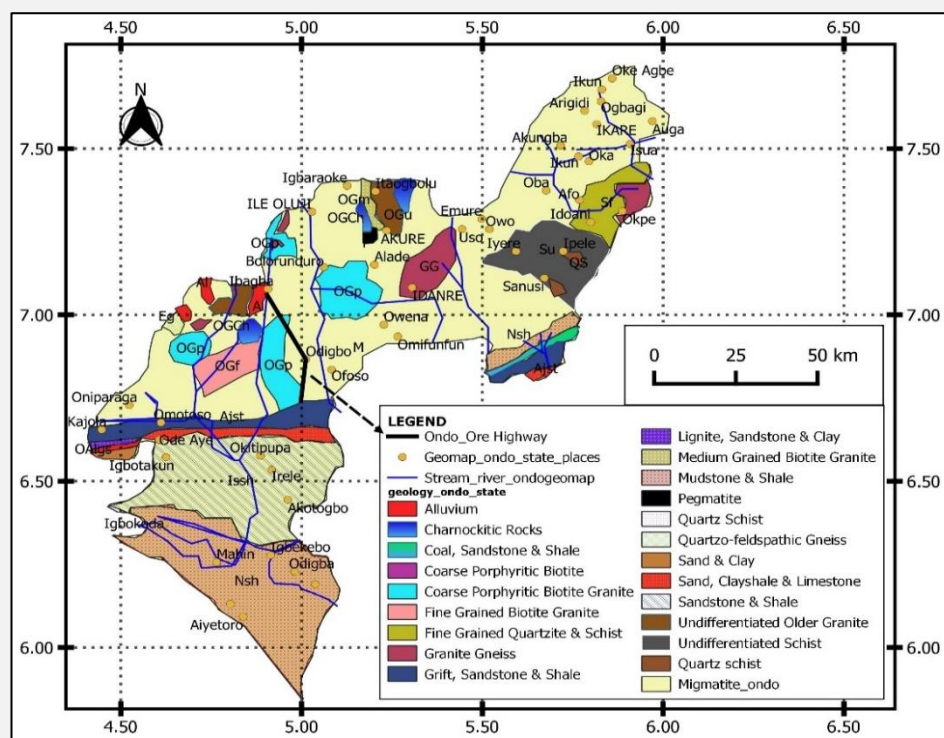


Figure 3. Geological Map of Ondo State showing the road under investigation cutting across migmatite, granite rock unit and terminated on shale and sandstone investigation (modified after Nigerian Geological Survey Agency, 2006)

Data Acquisition and Analysis

The Ondo – Ore was chosen for this study due to perennial failure of the highway, its importance in relation its connectivity to many important cities and towns. The incessant failure of the highway has led to increase in number of accidents, traffic congestion, extended journey time, increase in crime along the failed segments, and increase in faulty vehicles. The methods adopted for this study were divided into two stages: field work/survey and laboratory soil analysis (Murthy, 2007). Before beginning the stages, a literature review of accessible geological, geotechnical, hydrogeological, roadway, and transportation articles or text pertinent to this subject was conducted (Emesiobi, 2000). Electrical resistivity survey utilizing vertical electrical sounding technique, in-situ dynamic cone penetrometer test, soil excavation in form pits and trenches, water table measurement of wells along the highway, and identification of spring/artesian well system were all part of the field activity. Figure 4 depicts the study's data acquisition map. The selection of the sample locations during the field survey was guided by geology, degree of failure, accessibility, and safety. Before, the commencement of the study relevant authority (such as ministry of Transport, ministry of works and housing) were briefed about the nature of the study; and necessary and satisfactory approval were obtained. The data collection methods were safe and environmentally friendly in accordance to American Standard of Testing and Materials (ASTM, 1990) and British Standard procedures (BS 1377) of 1990 (British Standard Institution, 1990).

By detecting variations in the subsurface state, the geophysical inquiry aids in the detection of zones of abnormalities. By measuring certain physical qualities, they are used to ascertain the geological sequence and structure of subsurface rocks and soils (Telford et al., 1991; Attewell & Farmer, 1988). Density, elasticity, electrical conductivity, magnetic susceptibility, and gravitational attraction are the characteristics that are most often used in geophysical investigation (Williams, 1997; Vazirani & Chandola, 2009). In this investigation, vertical electrical sounding was used to measure electrical resistivity at four different points along the roadway (Aderemi & Adeola, 2021). By using two current electrodes to inject an electric current into the ground, this technique measures the difference in potential between two potential electrodes. Instead of measuring both current and voltage for this investigation, the resist-meter was able to measure the apparent resistance in ohms by performing automatic recording of both voltage

and current, stacks the results, computes the resistance in real time and digitally displays them. A 65 m half current spacing was employed with the Schlumberger array which progressively expands the current electrodes' spacing with fixed steps to enable sufficient penetration to the sub-surface earth and enhance structural responses. A bi-logarithmic graph of apparent resistivity value (i.e. the product of the geometric factor and the resistance recorded) obtained from the resist-meter against half the current electrode separation was generated. The data analysis comprised partial curve matching/curve fitting and modeling using WinResist software (Zhdanov & Keller, 1994) for the quantitative interpretation of the sounding curves. The results of the curve matching (layer resistivities and thicknesses) were used as starting model parameters for 1-D forward modeling using RESIST version 1.0. As a result, the electrode spacing at which the graph's inflection points appear provides information on the depth and thickness of the layers' interphases as well as their resistivity. The geoelectric segment of the roadway was built using the modeling results (VES interpretation results) in terms of resistivity and thickness.

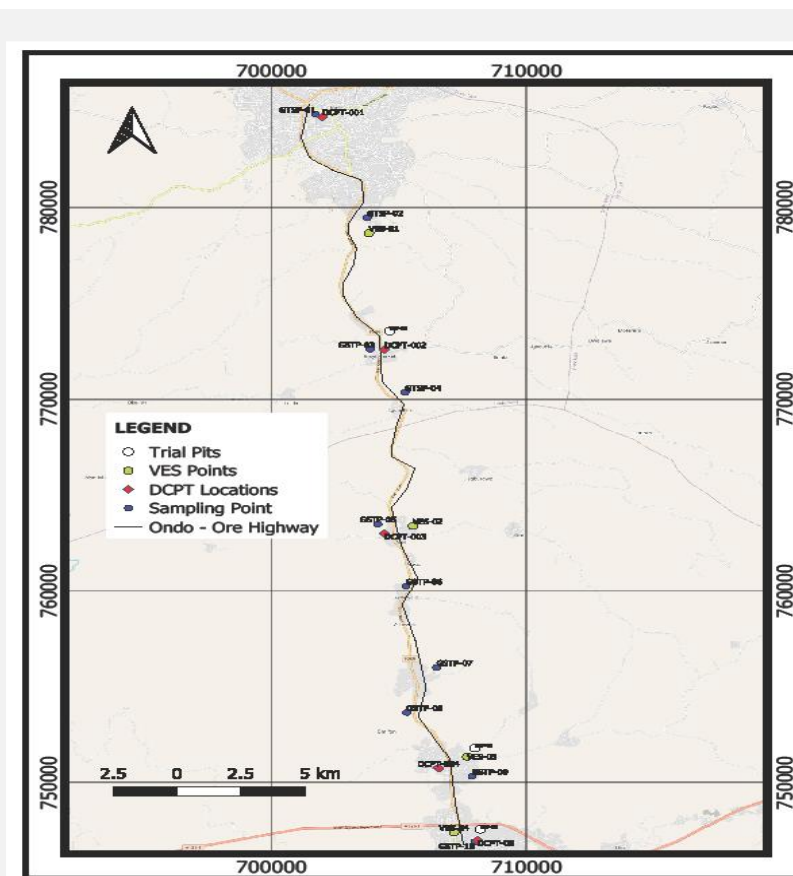


Figure 4. Data acquisition map for the study showing the geotechnical/geochemical sampling points, geophysical locations, and trial pit points

The DCPT was taken along the highway at a distance of between 1.0 and 3.0 m from the edge at both the stable and failed segments. The DCP is a straightforward mechanical device that can provide 45.5 joules of energy and is used for quick in-situ strength testing of roadway structural materials, particularly the subgrade and other unbound layers (Done & Samuel, 2006). When a standard force is applied, it measures the depth to which a standard cone penetrates. Along with the number of blows and depth of penetration, the standard hammer's DCP penetrative index in mm per blow is recorded. The conventional steel cone with a 20 mm diameter and a 60° angle was employed in this investigation. Ten (10) different places along the route were used to conduct the test. Due to the lack of security that typically characterizes failing highways, there were fewer tests conducted. For the analysis and interpretation of the gathered data, the UK DCP 3.1 software was employed. The data gathered at each location was adjusted for moisture content before being used to calculate the CBR using the Transport and Road Research Laboratory (TRL, 1990) relationship (Done & Samuel, 2006), as shown in Table 1. From Test No. 1 through Test No. 10, all of the test locations were given sequential numbers.

Table 1. CBR Adjustment Factor (Done & Samuel, 2006)

Surface moisture	Ratio of in-situ moisture to OMC (modified AASHTO)	Default CBR Adjustment Factor
Wet	1	1
Moderate	0.75	0.71
Dry	0.5	0.51
Very dry	0.25	0.37
Unknown (not assessed or difficult to assess)	-	0.5

The UK DCP 3.1 determined the strength coefficient of the test sites by first converting the penetration rate to the CBR value, then to the strength coefficient, and lastly to the structural number. The TRL equation was used for CBR calculation, as stated in equation 1. The strength coefficient of the subsoil for usage as the base and subbase layers is calculated using equations 2 (for base) and 3 (for subbase).

$$\text{Log}_{10}^{(\text{CBR})} = 2.48 - 1.057 \text{Log}_{10}^{(\text{pen rate})} \quad (1)$$

$$a = 0.0001[29.14 (\text{CBR}) - 0.1977 (\text{CBR})^2 + 0.00045 (\text{CBR})^3] \quad (2)$$

$$a = 0.184 \text{Log}_{10}^{(\text{CBR})} - 0.0444 (\text{Log}_{10}^{(\text{CBR})^2}) - 0.075 \quad (3)$$

The SNG which is referred to as subgrade structural number i.e. the contribution of the subsoil as subgrade material to structural number of a pavement (Done & Samuel, 2006). It is usually derived from CBR just like the base and the subbase layers. The relationship between SNG and CBR is presented shown in equation 4.

$$\text{SNG} = 3.51 \text{Log}_{10}^{(\text{CBR})} - 0.85 \text{Log}_{10}^{(\text{CBR})^2} - 1.43 \quad (4)$$

The relative densities of each subsoil layering were derived using DIN 4094 (1980) model (equation 5, where n_{10} is the number of blows for every 10 cm). The resilient modulus (using Lockwood et al., 1992; George & Uddin, 2000; and Chen et al., 1999 models, as shown in equations 6 – 8 respectively) and Young modulus were obtained from each site along the highway alignment using equation 9.

$$I_D = 0.21 + 0.230 \log n_{10} \quad (5)$$

$$M_R = 10^{3.04758 - 1.06166 \log[DCPI]} \quad (6)$$

$$M_R = 235.3 \times DCPI^{-0.475} \quad (7)$$

$$M_R = 338 \times DCPI^{-0.39} \quad (8)$$

$$E_R = \frac{M_R - 12.69}{1.065} \quad (9)$$

From the results of models, important correlations and parameters modeling were obtained between soaked CBR and in-situ CBR, M_R and E_R , M_R and CBR, DCPI and relative density, and CBR and relative density (Chen et al., 2005; Uz et al., 2015; Mohammad et al., 2007).

Three trial pits were dug along the highway to study the ground conditions, as it gives opportunity to assess directly the weathered rocks (Attewell & Farmer, 1988). The holes were dug with a digger by repeatedly dropping the tool into the ground. The depths range of the trial pits are within the upper 1.0 m, and no groundwater table was encountered. In addition, ten soil samples were taken at different chainage along the study highway as shown in Figure 4. Drilling was done using flight auger. The diameter of drilled boreholes was (15 cm). The disturbed samples were collected from the cutting of the auger at depths between 1 – 2 m. The disturbed soil samples (10 kg each) were collected at each of the locations within the site. The samples were put inside black polythene bags, labelled and packed

under controlled temperature to prevent the escape of moisture. The samples were subjected to geotechnical and geochemical tests at Applied Geology Department, Federal University of Technology, Akure, Nigeria. The geotechnical tests were conducted using ASTM methods/procedures (1990), and these included California Bearing Ratio (D-1883), compaction test (D-1557), particle size analysis (D-422), Atterberg limits (D-4318), moisture content (D-2216) and specific gravity (D-854; D-5550). The geochemical test was only analyzed for mineral oxides of SiO_2 , Fe_2O_3 , and Al_2O_3 using X-ray fluorescence and Atomic Absorption Spectrophotometer (AAS). The sample were initially sieved using 2 mm sieve and 2 g of the sieved sample was taken, after which they were put into digesting tube and digested using and HCl, then with HClO_4 and H_2O_2 . The samples were heated to dryness and make up with distilled water in a 100 ml volumetric flask. The resultant solution was analyzed using X-ray fluorescence and Atomic Absorption Spectrophotometer (AAS). Subsequently, the silica/sesquioxides (se) ratio (Adeyemi, 1995; Falowo & Dahunsi, 2020; Quadri et al., 2012) was calculated to know the type of the soil and classified it, if laterite ($se < 1.33$), lateritic ($1.33 < se < 2.0$) and non-laterite ($se > 2.0$). Traffic survey (classified volume counts) was conducted for seven days taking records of all vehicles plying the highway per day (Brown, 1996; Kadyali & Lal, 2005). It was conducted by noting the number of various classes of vehicles that pass the count point in each direction, hence average of daily traffic was used in estimating design thickness for highway pavement.

RESULT AND DISCUSSION

Electrical resistivity geophysical survey

The summary of the VES is presented in Table 2, while the geoelectric along the highway is shown in Figure 5. The curve types obtained from the highway alignment are H, KH, and HK denoting three-four-layer sequences. Geologically, it is made of topsoil, subsoil, weathered layer, fracture basement/basement rock. The topsoil has resistivity ranging from 98 – 366 ohm-m (avg. 184 ohm-m) and thickness varying from 0.8 – 1.2 m (avg. 0.95 m) and composed of clay, and sand clay (using interpretation Table 3). The subsoil resistivity is between 45 – 302 ohm-m. The weathered layer has resistivity ranging between 64 ohm-m and 406 ohm-m (avg. 159 ohm-m) indicating clayey weathered layer. The depths to basement rock varied from 33.5 – 45.1 m, indicating high weathering profile. Consequently, the topsoil, subsoil, and weathered layer are generally composed of clayey soil material, which can be regarded as incompetent/moderately competent soil material to support the pavement structure. It is observed that the basement relief is valley- like between VES 2 and 4 based on its configuration, this segment could aid flow of water or impound of water especially during the wet season, which can damage the road structure.

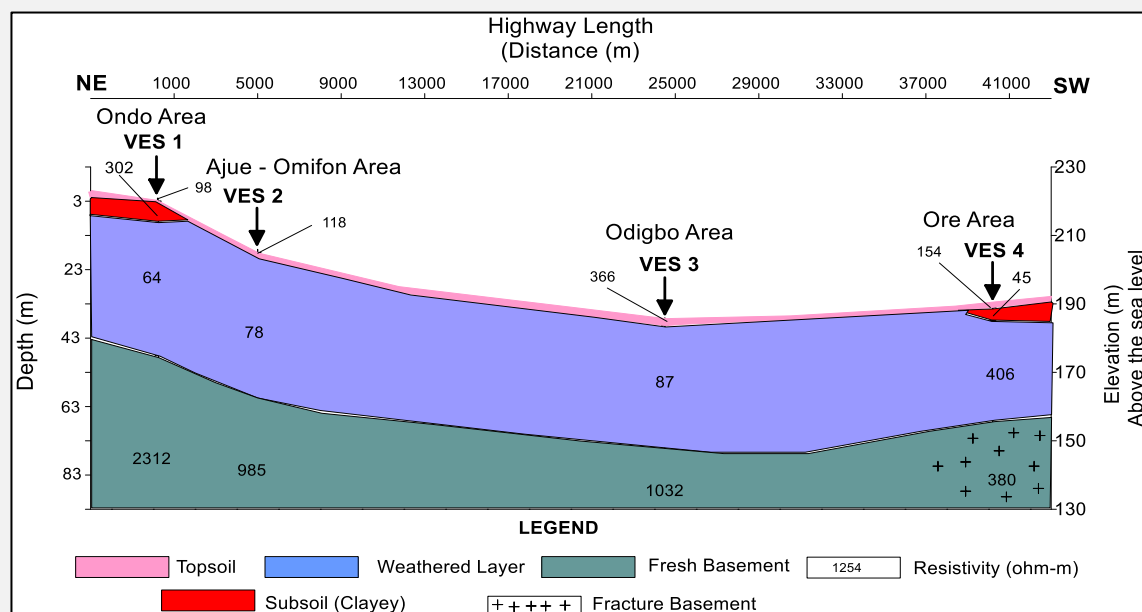


Figure 5. Geoelectric Section along the Highway Alignment

Table 2. Summary of VES results

East	North	Elevation (m)	VES NO.	Resistivity (Ohms-meter)					Thickness (m)				Depth (m)				Curve Type
				ρ_1	ρ_2	ρ_3	ρ_4	ρ_5	h_1	h_2	h_3	h_4	d_1	d_2	d_3	d_4	
703821	777390	245	1	98	302	64	2312	-	0.9	6.3	37.9	-	0.9	7.2	45.1	-	KH
705338	762324	205	2	118	78	985	-	-	0.9	42.8	-	-	0.9	43.7	-	-	H
707866	750595	185	3	366	87	1032	-	-	1.2	36.4	-	-	1.2	37.6	-	-	H
707562	746045	189	4	154	45	406	380	-	0.8	3.2	29.5	-	0.8	4	33.5	-	HK

Table 3. Rating of subsoil competence using Resistivity values

App. resistivity range (ohm-m)	Lithology	Competence rating
< 100	Clay	Incompetent
100 – 350	Sandy clay	Moderately competent
350 – 750	Clayey sand	Competent
> 750	Sand/Laterite/Crystalline Rock	Highly competent

Geochemical Analysis

Highway stability and serviceability performance depends on the mineralogical make-up of the soil (Kézdí & Rétháti, 1988; Bell, 1993). The result of chemical analysis (oxides) of the major elements (SiO_2 , Fe_2O_3 , and Al_2O_3) contained in the soil samples, and silica-sesquioxide ratio is presented in Table 4. The samples are well dominated (in descending order) by SiO_2 - Fe_2O_3 - Al_2O_3 , with an average value of 57.76 %, 17.95 % and 16.71 % respectively. All the samples are very rich in iron oxide (Fe_2O_3) which can attribute to chemical weathering of mafic mineral composition of the parent rock and ferruginization of Fe-bearing minerals, while enrichment of Al_2O_3 can be attributed to the weathering alteration of feldspar to clay mineral causing leaching of Al_2O_3 by infiltrating acid rain/recharge water into the ground (Bell, 2004; Bell, 2007). Silica-sesquioxide (Se) ratio of the ranges from 1.58 to 1.73 (avg. of 1.67), hence lateritic soil type.

Table 4. Result of the Chemical analysis of three major mineral oxide

Mineral oxide	S1	S2	S3	S4	S5	S6	S7	S8	S9
SiO_2	58.2	58.7	55.3	54.8	59.1	59.5	54.5	60.3	59.4
Al_2O_3	15.32	15.45	15.14	16.2	16.8	17.5	16.8	17.9	19.3
Fe_2O_3	18.5	18.9	17.2	18.3	17.5	16.9	17.2	18.65	18.4
S-S ratio	1.72	1.71	1.71	1.59	1.72	1.73	1.60	1.65	1.58
Soil Type	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic

Trial Pit Section

Trial pits can be used for all soil types. It is the cheapest way of site exploration, and do not require any specialized equipment (Brink et al., 1992). In this method a pit is manually excavated and soil is inspected in the natural condition. Five geologic units were observed from the three sites investigated (Figure 6) comprising sandy clay, clayey hardpan, laterite, clay, and silt-clay-sand mixture. At the northern axis of the highway notably at sites O1 and O2, (the geology is not different), thereby the weathering profile in the upper 1 m are the same consisting of sandy clay and clayey hardpan with corresponding depths of 0 - 0.5 m and 0.5 - 1.0 m; and 0 - 0.15 m, and 0.15 m - 0.70 m respectively. At site O2, laterite was observed at depth range of 0.70 - 1.0 m. Clay and silt-clay-sand mixture constitute the weathering profile within 1.0 m at Site O3 with depth range of 0 - 0.8 m, and 0.8 - 1.0 m. Consequently, this result compliment the E_R which delineated clayey topsoil along the highway alignment.

Geotechnical Analysis

Table 5 presents the summary of the geotechnical results. The natural moisture content varied from 18.2 to 23.8 % (avg. 20.71 %). Grain size analysis can be used to characterize the subsoil material for engineering foundation, which can serve as a guide to the engineering performance of the soil type and also provides a means by which soils can be identified quickly. The gravel and sand contents vary from 0 - 1 % (avg. 0.2 %) and 36.4 - 45.8 % (avg. 40.1 %) respectively. The % silt and clay contents ranged from 21.5 to 29.5 % (avg. 26.4 %) and 26.6 to 39.6 % (avg. 33.4 %). The % fines ranged from 54.2 to 63.6 (avg. 59.7). The composition of the soil is dominated by sand, clay, and silt (SC-SM). The plasticity chart (Figure 7) shows that the fines in the samples is dominated by clay of intermediate plasticity/compressibility (Figure 7a). All the soil samples plotted above the A-line. Most of the soil

samples are plotted within the Illite clay mineralogy group (Figure 7b). Like kaolinite, illite also may be of hydrothermal origin. The development of illite, both under weathering and by hydrothermal processes, is favoured by an alkaline environment (Bell, 2007; Chapman, 1981).

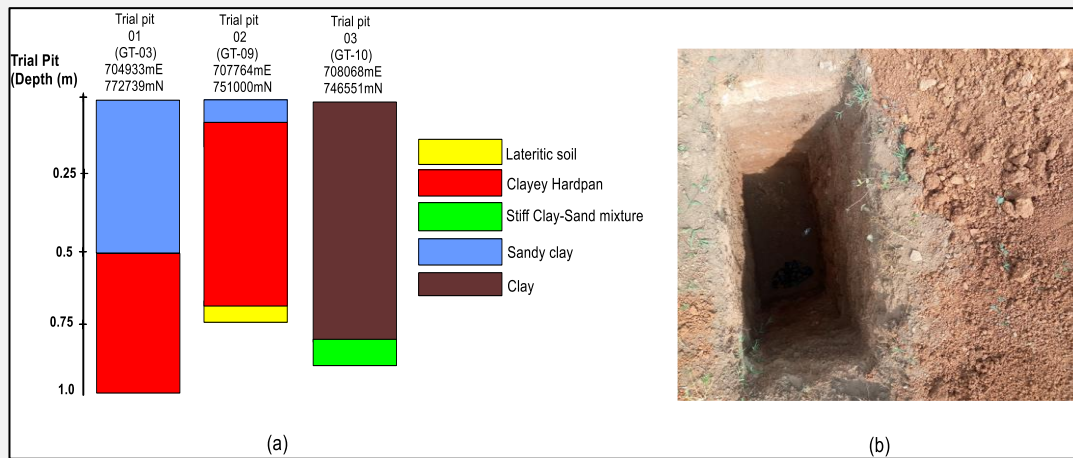


Figure 6. Trial pit of the three sites investigated along the Highway (a) geologic section (b) Photograph of Pit at location 02 (GT-09)

The activity ranged from 0.52 to 0.78 (avg. 0.66) signifying an inactive clay type. The values of specific gravity of the samples ranged between 2.65 – 2.74 (avg. 2.69). According to Wright (1986), the standard range of value of specific gravity of soils lies between 2.60 and 2.80; these values are considered normal. Specific gravity is known to correlate with mechanical strength of soil and may be used as a basis for selecting suitable highway pavement construction materials particularly when used with other pavement construction materials. The liquid limit (LL) values ranged between 42.5 to 50.1 % (47.2 %), plastic limits (PL) ranged between 22.4 to 27.6 % (avg. 25.1 %) and plasticity index (PI) is between to 19.8 to 25.1 % (avg. 22.2 %). The Federal Ministry of Works and Housing (1997) recommends LL of 50% (max.), PI of 20% as (max.), plastic limit of 30 % (max.) and % Fines of 35 maximum for highway subgrade soil. Hence the soils partially satisfied this requirement as subgrade material, since the PI is marginally above 20 % specification. The linear shrinkage ranged between 9.4 to 13.5 % (avg. 11.8 %).

Compaction is concerned with relationships between moisture content, applied effort and density. Compaction is undertaken on the road to enhance the mass density and hence the strength, rigidity and durability of placed materials (FHWA, 2006).

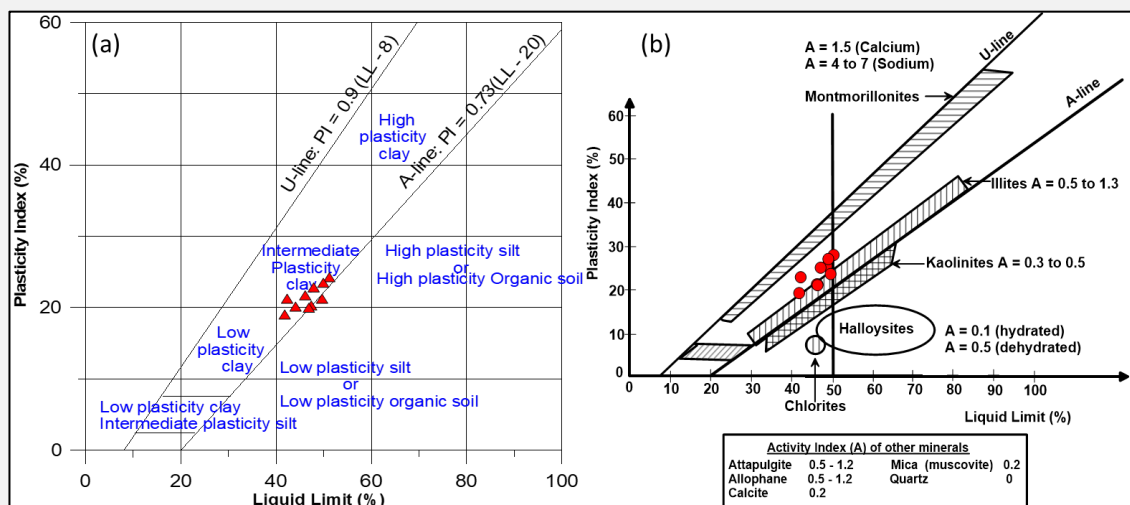


Figure 7. (a) Plasticity Chart for Fine Contents of the soil samples (b) Clay mineralogy group of the soil samples with most within/or near the illite

Table 5. Summary of the Geotechnical Results

Parameters	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
East	701597	703518	704023	705237	704225	705338	706652	705135	707866	707764
North	784114	778553	771879	769554	762577	759139	754892	752971	749432	745691
NMC	18.6	18.2	19.5	22.4	20.7	18.5	22.3	19.9	23.2	23.8
%Gravel	1	1	0	0	0	0	0	0	0	0
%Sand	38.5	36.5	39.2	43.4	38.7	44.2	45.8	36.4	38.1	40.4
%Silt	25.7	22.9	27.4	21.5	27.7	29.2	25.5	29.5	23.6	29.4
%Clay	34.8	39.6	33.4	35.1	33.6	26.6	28.7	34.1	38.3	30.2
%Fines	60.5	62.5	60.8	56.6	61.3	55.8	54.2	63.6	61.9	59.6
SG	2.65	2.7	2.662	2.678	2.71	2.685	2.74	2.696	2.667	2.708
LL (%)	49.6	45.3	47.7	46.2	42.5	47.9	49.5	50.1	49.8	43.7
PL (%)	26.4	24.8	23.8	25.8	22.7	27.6	27.1	25	25.2	22.4
PI (%)	23.2	20.5	23.9	20.4	19.8	20.3	22.4	25.1	24.6	21.3
SL	10.5	12.3	13.5	9.4	12.8	10.3	10.8	12.5	12.7	13.1
CBR soaked	8	8	8	15	6	14	10	14	10	5
In-situ CBR	5	5	6	19	8	12	12	18	7	5
MDD	1722	1699	1801	1878	1835	1863	1851	1785	1798	1752
OMC	23.3	22.8	23.1	26.7	25.5	22.2	25.7	24.4	25.5	26.9
GI	12	11	13	9	10	9	10	15	14	10
GI Class	Poor	Poor	Poor	Poor	Poor	Poor	Poor	Poor	Poor	Poor
Rec. Thickness (mm)	351	351	351	191	419	229	279	229	279	445
AASHTO	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5	A-7-5	A-7-6	A-7-5	A-7-5
USCS	CL	CL	CL	CL	CL	CL	ML	CL	CL	CL
Subgrade Rating	Poor	Poor	Poor	Poor	Poor	Poor	Poor	Poor	Poor	Poor
Activity	0.52	0.72	0.58	0.59	0.76	0.78	0.74	0.64	0.71	0.54
Clay Type	Inactive	Inactive	Inactive	Inactive	Normal	Normal	Inactive	Inactive	Inactive	Inactive
Clay mineralogy	I-M	I	I-M	I	I-M	I	I	I-M	I-M	I-M

In the laboratory compaction testing is undertaken to predict moisture density responses of a material to applied effort and to provide a reference with which to control on-site compaction during construction (Bell, 2004). The maximum dry density (MDD) for the soil samples varied between 1699 and 1878 kg/m³ at standard proctor compaction energy while the optimum moisture content (OMC) ranged between 22.2 and 26.9 %. An important part of the grading of the site often includes the compaction of fill. All the soil samples have moderately high MDD at moderate OMC.

The California Bearing Ratio (CBR) is an empirical test employed in road engineering as an index of compacted material strength and rigidity, corresponding to a defined level of compaction (Bell, 2004; Ampadu, 2007). All compacted samples show soaked CBR values ranging between 5 and 15 % (avg. 9.8 %), with corresponding in-situ values obtained from DCPT ranging from 4 to 19 (avg. 9.7). The Federal Ministry of Works and Housing (1997) recommends a California Bearing Ratio of greater than 10% for subgrade materials. Therefore, using Table, the soils are rated as low as pavement subgrade material. The Group Index (GI) values obtained ranged from 9 to 15 (avg. 11) corresponding to poor subgrade soil. The result shows that the California Bearing Ratio values of the soils are marginally lower than 10%. Using Table 6, the soil can be regarded as subgrade soil with medium strength classification. Based on the GI and CBR values, and the traffic count carried out which placed the highway as Class-E, the recommended thickness of the basement should range from 191 mm (good segment) to 445 mm (for weak segment) (avg. 312 mm) as shown in Figure 8, which is far higher than 274 mm measured along the highway alignment during reconnaissance study.

Table 6. Subgrade strength classification for the studied highway (Carter & Bentley, 1991)

Soaked CBR	Strength classification	Comments
< 1%	Extremely weak	Geotextile reinforcement and separation layer with a working platform typically required
1 % - 2 %	Very weak	Geotextile reinforcement and/or separation layer and/or a working platform typically required
2 % - 3 %	Weak	Geotextile separation layer and/or a working platform typically required
3 % - 10 %	Medium	
10 % - 30 %	Strong	Good subgrades to sub-base quality material
>30%	Extremely strong	Sub-base to base quality material

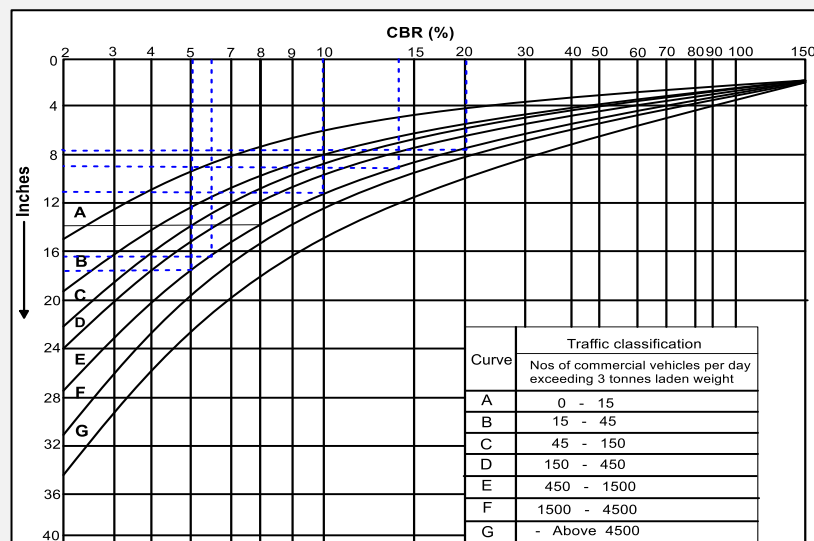


Figure 8. The CBR Chart adopted for determine the recommended thickness across the highway alignment

DCPT Analysis

The result summary of the DCPT is presented in Table 7, while subsoil layering in relation to its depth and in-situ CBR are shown Figure 9. In Table 7, the level of penetration ranged from 951 – 988 mm, with cumulative number of blows ranging from 24 to 36. The penetrative index or rate ranged between 7.67 mm/blow (site 4 at 926 mm depth) - 82 mm/blow (site 2 at 759 mm depth). All the sites are characterized by low - moderate cumulative number of blows in the upper 1 m investigated, signifying a loose/medium soil material. Along road alignment, one layer (sites 3, 9, 10), to two layers (sites 1, 2, 5, 8), and three layers (sites 4, 6, 7) were delineated. The obtained CBR ranged from 4 – 19 %. The most competent layers in terms of the obtained CBR are generally between 408 mm to 956 mm.

The estimated relative densities (RD) gives consistencies of the soil either very dense, dense, medium, loose or very loose (Ilori, 2015). Table 8 showed that the soil is generally loose (with relative density of 0.320) in the upper 800 mm except at site 4 where the soil become consistent of medium soil material (with density of 0.429) at depth of 500 mm. However, the obtained layering not totally consistent with those observed from DCPI except at sites 3, 4, 8, and 10. The SNG contribution of the soil as subgrade material ranged from -0.34 to 0.88. This range of values is less than 1.0 SNG strength coefficient for subgrade pavement layer. Sites 3 and 10 are characterized with very poor subgrade material as shown in their SNG (Table 9).

Consequently, relating the CBR and SNG, the depths of 475 and 408 mm will be appropriate for sites 1 and 2 respectively, 926 mm (site 4), 476 mm (site 5), 956 mm (site 6), 941 mm (site 7), 938 mm (site 8), 926 mm (site 9), and 937 mm (site 10). The strength coefficient of the soil as subbase and base ranged from 0.02 – 0.09, and 0.01 – 0.05, with respective SN and SNP ranging from 0.68 to 2.18 and 0.78 to 1.54; and 0.56 to 0.97 and 0.56 to 0.97. From the values, the strength coefficient is generally low for subbase and base material. The Young modulus and resilient modulus was estimated from Lockwood et al. (2000), Chen et al. (1999), and George & Uddin (2000) and the E_R varied between 10.32 – 144.43 (avg. 44.295), 65.16 – 145.88 (avg. 92.251), and 27.5 – 82.41 (avg. 45.294); the M_R ranged from 23.68 to 166.51 (avg. 59.864), 82.09 to 168.05 (avg. 110.938), and 41.98 to 100.46 (avg. 60.928) respectively. The Lockwood et al. (2000) and George & Uddin (2000) showed closely overlapping values, while Chen et al. (1999) showed a wide variation (Table 10).

Parameters modeling and correlations

The obtained soaked CBR from the laboratory was correlated with in-situ CBR obtained from processing of DCPT data, the plot gives positive correlation coefficient (R^2) of 0.9507 (Figure 10a), and linear regression model (equation 10):

$$\text{CBR (in-situ)} = 1.0432x + 5.0393 \quad (10)$$

In this relationship, $x = \text{CBR (soaked)}$.

The relative density values obtained from “DIN 4094” equation was plotted against in-situ CBR and DCPI. This gives a regression model of equations 11 and 12, with weakly positive correlations (R^2) of 0.0023 (Figure 10b) and 0.0259 (Figure 10c) respectively. This also showed the same trend.

$$\text{CBR (in-situ)} = 8.4746e^{0.397x} \quad (11)$$

$$\text{DCPI} = -10.34\ln(x) + 10.77 \quad (12)$$

In these relationships, $x = \text{relative density}$.

The relationship between E_R derived from “DIN 4094” and average M_R calculated from expressions proposed by Lockwood et al. (1992), Chen et al. (1999), and George & Uddin (2000) is shown by the regression model in equation 13, with R^2 of 0.9612 (Figure 10d).

$$M_R = 74.397 \ln(x) - 223.08 \quad (13)$$

Where x is modulus of elasticity.

The correlation between in-situ CBR and average M_R derived from the expressions of Lockwood et al. (1992), Chen et al. (1999), and George & Uddin (2000) to give equation 14, with correlation coefficient of 0.0796 (Figure 10e); while the plots of the in-situ CBR against each of this authors give R^2 of 0.0739, 0.0848, and 0.0838 (Figure 10f). All the models follow the same trend. The variation in the coefficients is marginally as showed weak positive correlations. The model expressions for these relationships are presented in equations 15 – 17.

$$M_R = 1.6789x + 59.112 \quad (14)$$

$$M_R = 2.4465x + 33.442 \quad (15)$$

$$M_R = 1.5435x + 94.269 \quad (16)$$

$$M_R = 1.0466x + 49.625 \quad (17)$$

Synthesis of results and Summary

The geological layers under the highway consist of clayey topsoil and weathered layer, with appreciable depth to basement rock. The relief of the basement is valley – like in some of the segments especially at failed segments which could aid flow of water or impound of water especially during the wet season, which can damage the road structure. This phenomenon was observed during the reconnaissance survey that surface water and ground water did not drain freely and quickly away from the road causing soil erosion, weaken pavement, destruction of road shoulders and wash out the embankments. This observation was also pointed out in Adams & Adetoro (2014), Adetoro & Akinwande (2014), Ifada et al. (2015), Adlinge & Gupta (2010). Therefore, no matter how beautifully, and anesthetics a highway appears, without taking care of the topographical low sections favourable for water impoundment, the highway would fall short of its design life expectancy.

The composition of the topsoil and weathered layer can attribute to chemical weathering of mafic mineral composition of the parent rock and ferruginization of iron bearing minerals; and alteration of feldspar to clay minerals. This was also verified in the trial pit inspected, as the weathering profile in the upper 1 m consisted of sandy clay, clayey hardpan, laterite, and silt-clay-sand mixture. However, it was clayey subsoil that dominated in all the trial pits, as supported by high fine contents (59%) obtained during the grain size analysis with intermediate compressibility.

The implication is that clayey formation is not good as subgrade because of its high porosity and very low permeability hence it tends to absorb water and swells, this result in high conductivity or low resistivity and differential settlement of the layer which subsequently lead to road failure. Therefore, the deterioration frequently experienced at many segments along the highway could arise from the differential settlement of the subgrade clay. This clayey occurrence has been recorded in many articles as one of the causes of road failure in the basement complex of Nigeria (Aderemi & Adeola, 2021; Ibitomi

et al., 2014; Adeyemo & Omosuyi, 2012; Daramola et al., 2015; Daramola et al., 2018; Meshida, 2006; Jegede, 1997, 1998, 2004; Adiat, 2009; Adesola et al., 2017; Falowo & Dahunsi, 2020).

Table 7. Summary of the DCPT showing the penetrative rate, depth of penetration, and number of blows for all the ten locations along the highway

TEST 1: CH. 0 + 0.500 km RHS												
Blow	0	3	3	3	3	3	3	3	3	3	-	-
Penetration (mm)	29	146	229	292	374	426	504	688	841	953	-	-
Cum. Blows	0	3	6	9	12	15	18	21	24	27	-	-
Depth (mm)	0	117	200	263	345	397	475	659	812	924	-	-
Penetration rate (mm/blow)	0	39	27.67	21.0	27.33	17.33	26.0	61.33	51.0	37.3	-	-
TEST 2: CH. 0 + 5.500 km LHS												
Blow	0	3	3	3	3	3	3	3	3	3	-	-
Penetration (mm)	30	139	243	289	372	437	542	788	924	985	-	-
Cum. Blows	0	3	6	9	12	15	18	21	24	27	-	-
Depth (mm)	0	110	214	260	343	408	513	759	895	956	-	-
Penetration rate (mm/blow)	0	36.33	34.67	15.33	27.67	21.67	35.0	82.0	45.33	20.3	-	-
TEST 3: CH. 0 + 16.400 km LHS												
Blow	0	3	3	3	3	3	3	3	3	-	-	-
Penetration (mm)	30	155	289	423	585	745	822	903	956	-	-	-
Cum. Blows	0	3	6	9	12	15	18	21	24	-	-	-
Depth (mm)	0	125	259	393	555	715	792	873	926	-	-	-
Penetration rate (mm/blow)	0	41.67	44.67	44.67	54.0	53.33	25.67	27.0	17.67	-	-	-
TEST 4: CH. 0 + 18.900 km RHS												
Blow	0	3	3	3	3	3	3	3	3	3	3	3
Penetration (mm)	28	121	163	208	252	334	456	571	668	785	872	931
Cum. Blows	0	3	6	9	12	15	18	21	24	27	30	33
Depth (mm)	0	93	135	180	224	306	428	543	640	757	844	903
Penetration rate (mm/blow)	0	31.0	14.0	15.0	14.67	27.33	40.67	38.33	32.33	39.0	29.0	19.67
TEST 5: CH. 0 + 28.300 km RHS												
Blow	0	3	3	3	3	3	3	3	3	3	3	3
Penetration (mm)	29	138	195	242	296	368	452	505	578	692	771	863
Cum. Blows	0	3	6	9	12	15	18	21	24	27	30	33
Depth (mm)	0	109	166	213	267	339	423	476	549	663	742	834
Penetration rate (mm/blow)	0	36.33	19.0	15.67	18.0	24.0	28.0	17.67	24.33	38.0	26.33	30.67
TEST 6: CH. 0 + 32.400 km RHS												
Blow	0	3	3	3	3	3	3	3	3	3	-	-
Penetration (mm)	32	125	241	292	388	442	568	801	884	933	988	-
Cum. Blows	0	3	6	9	12	15	18	21	24	27	30	-
Depth (mm)	0	93	209	260	356	410	536	769	852	901	956	-
Penetration rate (mm/blow)	0	31.0	38.67	17.0	32.0	18.0	42.0	77.67	27.67	16.33	18.33	-
TEST 7: CH. 0 + 35.000 km RHS												
Blow	0	3	3	3	3	3	3	3	3	-	-	-
Penetration (mm)	30	140	198	433	601	784	861	953	971	-	-	-
Cum. Blows	0	3	6	9	12	15	18	21	24	-	-	-
Depth (mm)	0	110	168	403	571	754	831	923	941	-	-	-
Penetration rate (mm/blow)	0	36.67	19.33	78.33	56.0	61.0	25.67	30.67	6.0	-	-	-
TEST 8: CH. 0 + 37.100 km RHS												
Blow	0	3	3	3	3	3	3	3	3	3	-	-
Penetration (mm)	28	133	269	391	526	693	777	881	936	966	-	-
Cum. Blows	0	3	6	9	12	15	18	21	24	27	-	-
Depth (mm)	0	105	241	363	498	665	749	853	908	938	-	-
Penetration rate (mm/blow)	0	35.0	45.33	40.67	45.0	55.67	28.0	34.67	18.33	10.0	-	-
TEST 9: CH. 0 + 39.500 km RHS												
Blow	0	3	3	3	3	3	3	3	3	-	-	-
Penetration (mm)	29	142	288	403	489	605	721	838	902	955	-	-
Cum. Blows	0	3	6	9	12	15	18	21	24	27	-	-
Depth (mm)	0	113	259	374	460	576	692	809	873	926	-	-
Penetration rate (mm/blow)	0	37.67	48.67	38.33	28.67	38.67	38.67	39.0	21.33	17.67	-	-
TEST 10: CH. 0 + 40.000 km RHS												
Blow	0	3	3	3	3	3	3	-	-	-	-	-
Penetration (mm)	28	166	292	448	591	766	852	965	-	-	-	-
Cum. Blows	0	3	6	9	12	15	18	21	-	-	-	-
Depth (mm)	0	138	264	420	563	738	824	937	-	-	-	-
Penetration rate (mm/blow)	0	46.0	42.0	52.0	47.67	58.33	28.67	37.67	-	-	-	-

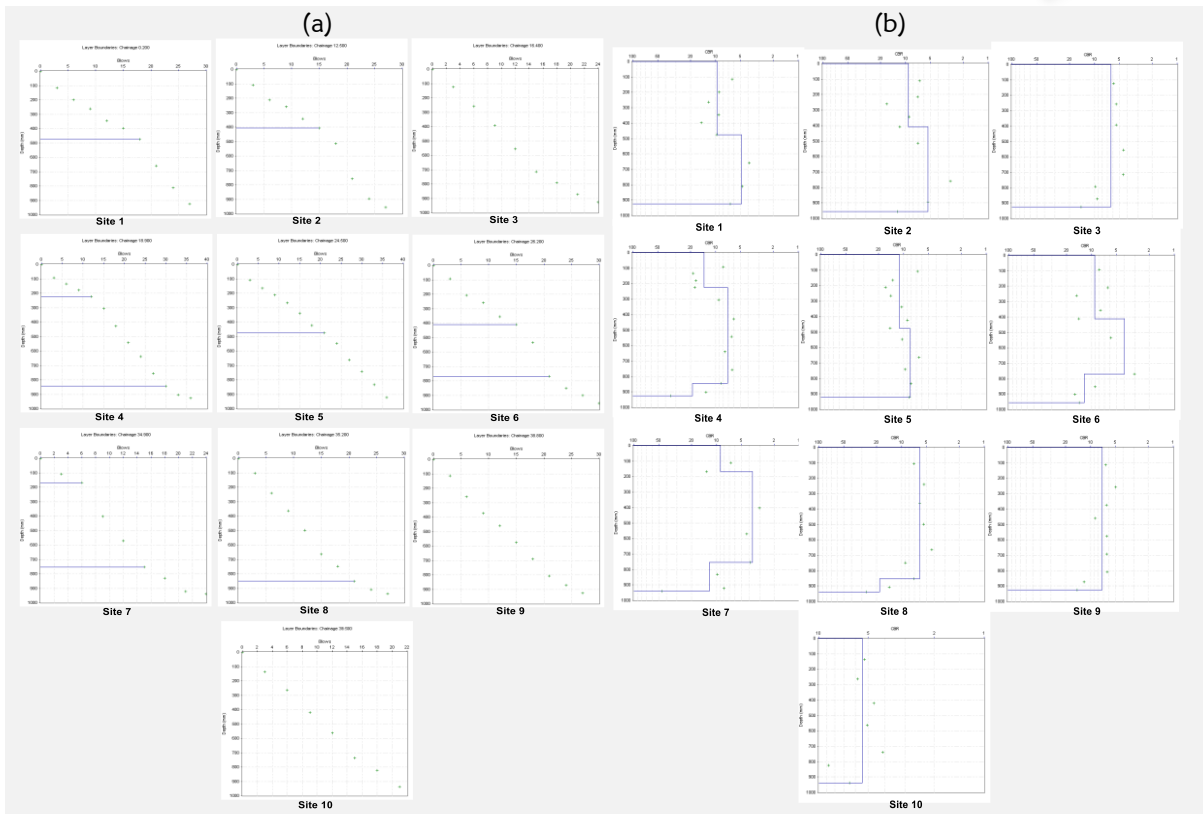


Figure 9. (a) The plot of Cumulative Blows against Depth at Sites 1 – 10 showing the layering within the upper 1.0 m; (b) The plot of CBR against Depth at Test points 1 – 10, showing the CBR of the layers

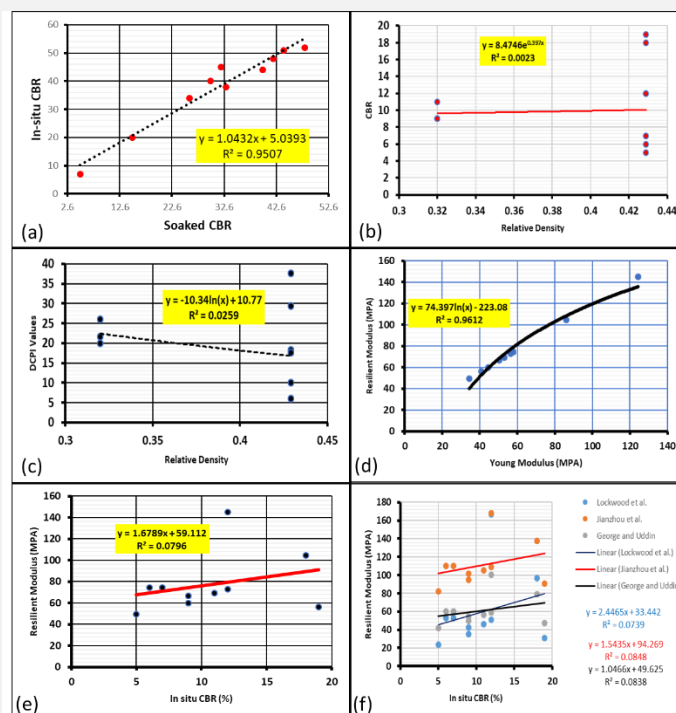


Figure 10. Regression models for (a) CBR lab and in-situ CBR (b) RD and in-situ CBR (c) RD and DCPI (d) E_R and M_R (e) in-situ CBR and M_R (f) in-situ and M_R for Lockwood et al. (1992), Chen et al. (1999), and George & Uddin (2000)

Table 8. DCPT results showing relative densities per every 10 cm, their penetrative rate, and the consistencies of the soil

Test 1									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	6	3	3	3	3	3	3	6
Relative Density	0.320	0.429	0.320	0.320	0.320	0.320	0.320	0.320	0.429
Soil Consistency	Loose	Medium	Loose	Loose	Loose	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	39.0	27.67	-	17.33	-	-	-	-	-
Test 2									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	6	3	3	3	3	3	3	6
Relative Density	0.320	0.429	0.320	0.320	0.320	0.320	0.320	0.320	0.429
Soil Consistency	Loose	Medium	Loose	Loose	Loose	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	34.33	-	-	21.67	-	-	-	-	45.33
Test 3									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	3	3	3	3	3	3	3	6
Relative Density	0.320	0.320	0.320	0.320	0.320	0.320	0.320	0.320	0.429
Soil Consistency	Loose	Loose	Loose	Loose	Loose	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	40.67	-	-	44.67	-	-	53.33	25.67	-
Test 4									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	6	6	3	3	6	6	6	6	6
Relative Density	0.429	0.429	0.320	0.320	0.429	0.429	0.429	0.429	0.429
Soil Consistency	Medium	Medium	Loose	Loose	Medium	Medium	Medium	Medium	Medium
Penetration rate (mm/blow)	36.33	-	-	27.0	-	-	-	29.67	29.33
Test 5									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	6	6	3	3	6	3	3	3	6
Relative Density	0.429	0.429	0.320	0.320	0.429	0.320	0.320	0.320	0.429
Soil Consistency	Medium	Medium	Loose	Loose	Medium	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	36.33	15.67	-	28.0	17.67	-	38.0	-	-
Test 6									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	6	3	3	3	3	3	6	6
Relative Density	0.320	0.429	0.320	0.320	0.320	0.320	0.320	0.429	0.429
Soil Consistency	Loose	Medium	Loose	Loose	Loose	Loose	Loose	Medium	Medium
Penetration rate (mm/blow)	31.0	38.67	-	18.0	-	-	-	77.67	16.33
Test 7									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	6	3	3	3	3	3	3	6
Relative Density	0.320	0.429	0.320	0.320	0.320	0.320	0.320	0.320	0.429
Soil Consistency	Loose	Medium	Loose	Loose	Loose	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	36.67	-	-	78.33	-	-	-	25.67	-
Test 8									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	3	3	3	3	3	3	3	6
Relative Density	0.320	0.320	0.320	0.320	0.320	0.320	0.320	0.320	0.429
Soil Consistency	Loose	Loose	Loose	Loose	Loose	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	35.0	-	-	-	45.0	-	55.67	-	18.33
Test 9									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	3	3	6	3	3	3	6	6
Relative Density	0.320	0.320	0.320	0.429	0.320	0.320	0.320	0.429	0.429
Soil Consistency	Loose	Loose	Loose	Medium	Loose	Loose	Loose	Medium	Medium
Penetration rate (mm/blow)	37.67	-	-	38.33	-	-	38.67	39.0	-
Test 10									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10cm	3	3	3	3	3	3	3	3	6
Relative Density	0.320	0.320	0.320	0.320	0.320	0.320	0.320	0.320	0.429
Soil Consistency	Loose	Loose	Loose	Loose	Loose	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	46.0	-	-	52.0	-	-	-	-	37.67

Table 9. Summary of the CBR results in relation to strength coefficient of the soils as subgrade, subbase, and base material

Test No.	Test Layer No.	CBR (%)	Thickness (mm)	Depth (mm)	Subgrade	Position	Strength coefficient	Pavement Strength/Layer contribution			Position	Strength coefficient	Pavement Strength/Layer contribution		
					SNG			SN	SNC	SNP			SN	SNC	SNP
1	1	9	475	475	0.30	Sub-Base	0.06	1.70	1.70	1.25	Base	0.03	0.73	0.73	0.73
	2	5	449	924	-0.16	Sub-Base	0.03				Base	0.01			
2	1	9	408	408	0.30	Sub-Base	0.06	1.74	1.74	1.23	Base	0.03	0.73	0.73	0.73
	2	5	548	956	-0.16	Sub-Base	0.04				Base	0.01			
3	1	6	926	926	-0.02	Sub-Base	0.04	1.61	1.61	1.04	Base	0.02	0.65	0.65	0.65
4	1	14	224	224	0.64	Sub-Base	0.08	2.18	2.18	1.46	Base	0.04	0.96	0.96	0.96
	2	7	620	844	0.10	Sub-Base	0.05				Base	0.02			
	3	19	82	926	0.88	Sub-Base	0.09				Base	0.05			
5	1	11	476	476	0.45	Sub-Base	0.07	2.29	2.29	1.54	Base	0.03	0.97	0.97	0.97
	2	8	446	922	0.20	Sub-Base	0.06				Base	0.02			
6	1	9	410	410	0.30	Sub-Base	0.06	1.79	1.79	1.20	Base	0.03	0.81	0.81	0.81
	2	4	359	769	-0.34	Sub-Base	0.02				Base	0.01			
	3	12	187	956	0.52	Sub-Base	0.07				Base	0.03			
7	1	9	168	168	0.30	Sub-Base	0.06	1.26	1.26	0.78	Base	0.02	0.64	0.64	0.64
	2	4	586	754	-0.34	Sub-Base	0.01				Base	0.01			
	3	12	187	941	0.52	Sub-Base	0.07				Base	0.03			
8	1	6	853	853	-0.02	Sub-Base	0.04	0.68	1.68	1.02	Base	0.02	0.72	0.72	0.72
	2	18	85	938	0.84	Sub-Base	0.09				Base	0.05			
9	1	7	926	926	0.10	Sub-Base	0.05	1.83	1.83	1.18	Base	0.02	0.73	0.73	0.73
10	1	5	937	937	-0.16	Sub-Base	0.04	1.34	1.34	0.86	Base	0.02	0.56	0.56	0.56

Table 10. Summary of the Modulus of Elasticity and Resilient Modulus at every Chainage where samples were taken

Test No.	Chainage along Highway (km)	In situ CBR	Subgrade SNG	Young modulus M_R values (Lockwood et al., 1992)	Young modulus M_R values (Chen et al., 1999)	Young modulus M_R values (George & Uddin, 2000)	Resilient modulus E_R values (Lockwood et al., 1992)	Resilient modulus E_R values (Chen et al., 1999)	Resilient modulus E_R values (George & Uddin, 2000)
Test No. 1	CH. 0 + 0.5 RHS	9	0.30	21.05	77.15	35.09	35.10	94.86	50.06
Test No. 2	CH. 0 + 5.50 LHS	9	0.30	28.08	83.71	39.34	42.59	101.84	54.59
Test No. 3	CH. 0 + 16.40 LHS	6	-0.02	37.75	91.63	44.56	52.90	110.28	60.14
Test No. 4	CH. 0 + 18.90 RHS	19	0.88	17.09	73.06	32.48	30.89	90.50	47.28
Test No. 5	CH. 0 + 28.30 RHS	11	0.45	31.63	86.75	41.33	46.38	105.08	56.71
Test No. 6	CH. 0 + 32.40 RHS	12	0.52	35.86	90.16	43.58	50.88	108.71	59.10
Test No. 7	CH. 0 + 35.0 RHS	12	0.52	144.43	145.88	82.41	166.51	168.05	100.46
Test No. 8	CH. 0 + 37.10 RHS	18	0.84	78.99	117.38	62.09	96.81	137.69	78.82
Test No. 9	CH. 0 + 39.50 RHS	7	0.10	37.75	91.63	44.56	52.90	110.28	60.14
Test No. 10	CH. 0 + 40.0 RHS	5	-0.16	10.32	65.16	27.50	23.68	82.09	41.98

It is recommended that for any road construction purpose, the clayey materials should be excavated away because of their unstable characteristics and re-filled with more stable materials. Even though, the silica-sesquioxide ratio of the samples signified lateritic soil. But there's possibility that the laterization process of the soil is still ongoing. Most of the soil samples are plotted within the Illite (inactive) clay mineralogy group. Consequently, some degree of expansion is expected from illite dominated clay but might not be as severe as montmorillonite (Okunlola et al., 2015; Obaje, 2017).

The deterioration of the highway could be also as a result of low Group index and soaked CBR values; and corresponding DCPT values, as they are generally less than the standard recommendation of the Federal Ministry of Works and Housing (1997) recommends of minimum of 10% and 5% maximum respectively for subgrade materials. The most competent layers in terms of the obtained CBR are generally between 408 mm to 956 mm. The subgrade structural number (SNG) contribution of the soil as subgrade, the strength coefficient of the soil as subbase and base material are very low. This range of values is less than 1.0. Therefore, poor geotechnical properties of the subsoil is a factor be considered critically during the rehabilitation/reconstruction of the highway, since clayey soil (which is the commonest subsoil material that characterized the basement complex Nigeria) are mechanically unstable in terms of shear strength, bearing capacity, and compressibility. This assertion was also supported by Jegede & Olaleye (2013), Aghamelu & Okogbue (2011), Ubido et al. (2021), Adetoro & Abe (2018), Adiat, 2017; Obaje, 2017).

The expected thickness of the highway structure should range from 191 mm (good segment) to 445 mm (for weak segment) (avg. 312 mm). Hence, this average value is above 274 mm measured along the highway alignment. Definitely, the existing thickness may not be able to sustain the haulage and high axle loads plying the highway presently. The estimated relative densities (RD) gives consistencies of the soil are of medium soil material (with density of 0.429) at depth of 500 mm. However, the strength of the subgrade can be improved when subjected to stabilization measures; or better still, failed segments of the highway with clayey or silty materials should be scooped out from the subsurface to a reasonable depth of at least 1 m from the topsoil and re-fill with competent materials. This should be put into consideration during the reconstruction and rehabilitation of the highway. Other contributing factors to the failure of some the highway segments, such as inadequate drainage system, paucity of maintenance, poor pavement coating of the road, and substandard construction materials are factors should be investigated by the federal government, most especially periodic and seasonal maintenances. On the issue of poor drainage facilities, proper drainage should be constructed to discourage the accumulation of runoff and thereby expose the subsurface material to erosion.

CONCLUSION

The structural deterioration of F-209 Segment of Ondo – Ore pavement had been investigated, and modeling of its geoenvironmental parameters. The main findings from the study showed that the subsoil on which the highway is constructed are characterized by clayey soil material of low soaked CBR/in-situ CBR values, low strength coefficient; and low design thickness across the roadway are the causes of the highway's collapse. The absence of drainage on the highway's shoulders is another contributing cause. The significance of the findings of this study for engineering practice, policy making, scientific knowledge, is that comprehensive or adequate site evaluation must be carried out prior to design and construction highway; in addition, on-site assessment of in-situ materials must be done, for use of such materials for construction. Thus, where inadequacies have been identified during the site assessment, this must be incorporated into the design; and where subsoil are geotechnical - poor or weak (as observed in this study), proper stabilization techniques must be adopted as may be required to achieve compliance with specifications. Nevertheless, it should be noted that, the influence of geology or the parent rock on engineering properties and the position of the horizon within the soil profile determine the engineering properties and behaviour of residual soils.

The limitation of this study was that the VES carried out along highway was insufficient to give an overall geologic profile of the highway, layer thickness and the properties of different pavement layers. Hence, it's advocated that Ground Penetrating Radar should be deployed for further studies. The GPR method provides a high-resolution image of subsurface features in the form of a cross section view that is essentially a map of the variation in ground electrical properties. This can be linked with physical changes such the subsurface layering, soil bedrock boundary, the boundaries between asphaltic layers, and water table.

ACKNOWLEDGEMENT

The Tertiary Education Trust Fund (TETFund) of Nigeria.

COMPETING INTEREST

The author declares that no competing interest exists.

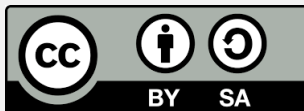
REFERENCES

- Adams, J. O., & Adetoro, A. E. (2014). Analysis of Road Pavement Failure caused by Soil Properties along Ado-ekiti - Akure Road, Nigeria. *International Journal of Novel Research in Engineering and Applied Sciences (IJNREAS)*, 1(1): 1-7.
- Ademeso, O. A., & Ogunjobi, O. S. (2021). Hydraulic Conductivity of Subsoils vis-à-vis Foundation Applications in Akure, Southwestern Nigeria. *Journal of Mining and Geology* 57(1), 217-224.
- Aderemi, F. L., & Adeola, R. O. (2021). Geophysical Investigation of Causes of Road Failure along Abadina Community Road, University of Ibadan, Nigeria. *Journal of Research in Environmental and Earth Sciences*, 7(1), 1-5.
- Adesola, A. M., Ayokunle, A. A., & Adebowale, A. O. (2017). Integrated geophysical investigation for pavement failure along a dual carriage way, Southwestern Nigeria: a case study. *Kuwait Journal of Science*, 44(4), 135-149.
- Adetoro, A. E., & Abe, O. E. (2018). Assessment of Engineering Properties of Ado-Ekiti to Ikere-Ekiti Road Soil, Southwestern Nigeria. *World Wide Journal of Multidisciplinary Research and Development*, 4(6), 191-195.
- Adetoro, A. E., & Akinwande, J. T. (2014). Assessment of some Pavement Indices of Ado Ekiti – Akure Road, South-Western Nigeria. *International Journal of Novel Research in Engineering and Applied Sciences (IJNREAS)*, 1(1), 1-7.
- Adeyemi, G. O. (1995). The influence of parent rock factor on some engineering index properties of three residual lateritic soils in southwestern Nigeria. *Bulletin International Association of Engineering Geology*, 52(1), 3-8.
- Adeyemo, I. A., & Omosuyi, G. A. (2012). Geophysical Investigation of Road Pavement Instability along Akure-Owo express way, South Western Nigeria. *American Journal of Scientific and Industrial Research*, 3(4), 191-197.
- Adiat, K. A. N., Adelusi, A. O., & Ayuk, M. A. (2009). Relevance of geophysics in road failures investigation in a typical basement complex of southwestern Nigeria. *Pacific Journal of Science and Technology*, 51(1), 528-539.
- Adiat, K. A. N., Akinlalu, A. A., & Adegoroye, A. A. (2017). Evaluation of road failure vulnerability section through integrated geophysical and geotechnical studies. *NRIAG Journal of Astronomy and Geophysics*, 6(1), 244-255. <https://doi.org/10.1016/j.nrjag.2017.04.006>
- Adlinge, S. S., & Gupta, A. K. (2010). Pavement deterioration and its cause. *IOSR Journal of Mechanical & Civil Engineering*, 2(4), 9-15.
- Aghamelu, O. P., & Okogbue, C. O. (2011). Geotechnical assessment of road failures in the Abakaliki area, southwestern Nigeria. *International Journal of Civil & Environmental Engineering*, 11(2), 12–24.
- Ampadu, S. I. K. (2007). A laboratory investigation into the effect of water content on the CBR of a subgrade soil. In *Experimental unsaturated soil mechanics* (pp. 137-144). Springer Berlin Heidelberg.
- ASTM. (1990). *Standard Laboratory Methods*. American Society for Testing and Materials.
- Attewell, P. B., & Farmer, I. W. (1988). *Principles of Engineering Geology*. John Wiley & Sons, Inc, New York. <https://doi.org/10.1007/978-94-009-5707-7>
- Bell, F. G. (1993). *Engineering Treatment of Soils*. CRC Press.
- Bell, F. G. (2004). *Engineering Geology and Construction*. CRC Press.
- Bell, F. G. (2007). *Engineering Geology (Second Edition)*. Elsevier.
- Brink, A. B. A., Parridge, J. C., & Williams, A. A. B. (1992). *Soil Survey for Engineering*. Clarendon Press.

- British Standard Institution. (1990). *Methods of test for soils for civil engineering properties (BS 1377)*. British Standard Institution: London, UK., 143.
- Brown, S. F. (1996). Soil mechanics in pavement engineering. *Geotechnique*, 46(3), 383–426. <https://doi.org/10.1680/geot.1996.46.3.383>
- Carter, M., & Bentley, S. P. (1991). *Correlations of soil properties*. Pentech Press Publishers.
- Chapman, R. E. (1981). *Geology and Water: An introduction to fluid mechanics for geologists*. Springer Dordrecht. <https://doi.org/10.1007/978-94-009-8244-4>
- Chen, D. -H., Lin, D. -F., Liao, P. -H., & Bilyeu, J. (2005). A correlation between Dynamic Cone Penetrometer values and pavement layer moduli. *Geotechnical Testing Journal*, 28(1), 42-49. <http://dx.doi.org/10.1520/GTJ12312>
- Chen, J., Hossain, M., & Latorella, T. M. (1999). Use of falling weight deflectometer and dynamic cone penetrometer in pavement evaluation. *Transportation Research Record*, 1655(1), 145-151. <https://doi.org/10.3141/1655-19>
- Daramola, S. O., Malomo, S., & Asiwaju-Bello, Y. A. (2015). Engineering geology of failed sections of Isua-Idoani road Southwestern Nigeria. *Journal of Environment and Earth Science*, 5(19), 38–45.
- Daramola, S. O., Malomo, S., & Asiwaju-Bello, Y. A. (2018). Premature failure of a major highway in southwestern Nigeria: the case of Ipele–Isua highway. *International Journal of Geo-Engineering*, 9(28), 1-12. <https://doi.org/10.1186/s40703-018-0096-9>
- DIN 4094 Part 2. (1980). *Dynamic and Static Penetrometer: Application and Evaluation, Deutsche Norm*.
- Done, S., & Samuel, P. (2006). Department for International Development (DFID) Measuring road pavement strength and designing low volume sealed roads using the dynamic cone penetrometer. *Unpublished Project Report, UPR/IE/76/06 Project Record No R7783*.
- Emesiobi, F. C. (2000). Testing and quality control of materials in civil and highway engineering. *The BluePrint Limited, Port Harcourt, Nigeria*.
- Falowo, O. O., & Dahunsi, S. D. (2020). Geoengineering Assessment of Subgrade Highway Structural Material along Ijebu Owo – Ipele Pavement Southwestern Nigeria. *International Advanced Research Journal in Science, Engineering and Technology (IARJSET)*, 7(4), 1-10. <https://doi.org/10.17148/IARJSET.2020.7401>
- Federal Meteorological Survey. (1982). *Atlas of the Federal Republic of Nigeria, 2nd Edition*. Federal Surveys, 160pp.
- Federal Ministry of Works and Housing. (1997). *Nigerian general specifications for roads and Bridges Vol. II*. FMW&H, Ministry of Works, Abuja.
- FHWA. (2006). *NHI-05-037: Geotechnical aspects of pavement*. US Department of Transportation Federal Highway Administration Pp4-17.
- George, K. P., & Uddin, W. (2000). *Subgrade characterization for highway pavement design* (No. FHWA/MS-DOT-RD-00-131). Mississippi. Dept. of Transportation.
- Gudishala, R. (2004). *Development of resilient modulus prediction models for base and subgrade pavement layers from in situ devices test results*. Louisiana State University and Agricultural & Mechanical College. Retrieved from <https://www.proquest.com/dissertations-theses/development-resilient-modulus-prediction-models/docview/2675221951/se-2>
- Hopkins, T. (1994). Minimum bearing strength of soil subgrades required to construct flexible pavements. In *4th International Conference, Bearing Capacity of Roads and Airfields, FHWA, U of Minnesota, Army Corps of Engineers, NRC Canada, FAA* (Vol. 1).
- Ibitomi, M. A., Fatoye, F. B., & Onsachi, J. M. (2014). Geophysical Investigation of Pavement Failure on a Portion of Okene-Lokoja Highway, North Central Nigeria. *Journal of Environment and Earth Science*, 4(13), 44-51.

- Ifada, P. T., Kasali, I. A., Makanjuola, P. I., & Monye, O. A. (2015). Assessment of Permeability Characteristics of Sub-Grade Soil along Failed Sections of Ado-Ikere Road, South-Western Nigeria. *Nigeria, Ado-ekiti, The Federal Polytechnic, Dept. of Civil Engineering: HND Dissertation (Unpublished)*.
- Ifarajimi, W. T., Oladapo, M. I., Bayode, S. (2021). Geophysical and Geotechnical Evaluation of Subgrade Integrity Classification of Some Failed Section along Sagamu-Benin Highway Southwestern Nigeria. *Minna Journal of Geosciences (MJG)*, 5(1&2), 75-96.
- Iloje, P. N. (1981). *A new geography of Nigeria*. Longman Nigeria Limited, Lagos.
- Ilori, A. O. (2015). Geotechnical characterization of a highway route alignment with light weight penetrometer (LR5 10), in southeastern Nigeria. *International Journal of Geo-Engineering*, 6(7), 1-28. <https://doi.org/10.1186/s40703-015-0007-2>
- IRC. (2002). *Guidelines for the design of flexible pavements*. Indian roads congress Number 37 New Delhi.
- Jegede, G. (1997). Highway pavement failure induced by soil properties along the F209 highway at Omuoke, southwestern Nigeria. *Nigeria Journal of Science*, 31, 121-126.
- Jegede, G. (1998). Effects of some engineering and geological factors on highway failures in parts of Southwestern Nigeria. *Unpublished Ph. D. thesis, Federal University of Technology, Akure*, 251.
- Jegede, G. (2004). Highway pavement failure induced by poor soil geotechnical properties at a section along the F209 Okitipupa-Igbokoda highway, southwestern Nigeria. *Ife Journal of Science*, 6(1), 41-44. <https://doi.org/10.4314/ijfs.v6i1.32121>
- Jegede, O. G., & Olaleye, B. M. (2013). Evaluation of engineering geological and geotechnical properties of sub grade soils along the re-Aligned Igbara-Odo Ikogosi highway, South Western, Nigeria. *The International Journal of Engineering and Science*, 2(5), 18-21.
- Kadiyali, L. R., & Lal, N. B. (2005). *Principles and Practices of Highway Engineering:(Including Expressways and Airport Engineering)*. Khanna Publishers.
- Kézdi, Á., & Rétháti, L. (1988). *Soil mechanics of earthworks, foundations, and highway engineering* (Vol. 3). Elsevier.
- Lockwood, D., de Franca, V. M. P., Ringwood, B., de Beer, M. (1992). Analysis and classification of DCP Survey Data. *Technology and Information Management Programme, CSIR Transportek, Pretoria, South Africa*.
- Meshida, E. A. (2006). Highway failure over talc–tremolite schist terrain: a case study of the Ife to Ilesha highway, South Western Nigeria. *Bulletin of Engineering Geology and the Environment*, 65, 457-461. <https://doi.org/10.1007/s10064-005-0037-7>
- Mohammad, L. N., Herath, A., Abu-Farsakh, M. Y., Gaspard, K., & Gudishala, R. (2007). Prediction of resilient modulus of cohesive subgrade soils from dynamic cone penetrometer test parameters. *Journal of Materials in Civil Engineering*, 19(11), 986-992. [https://doi.org/10.1061/\(ASCE\)0899-1561\(2007\)19:11\(986\)](https://doi.org/10.1061/(ASCE)0899-1561(2007)19:11(986))
- Murthy, V. N. S. (2007). *Advanced foundation engineering*. New Delhi: CBS publishers & distributors.
- Nigerian Geological Survey Agency. (1984). *Geological Map of Southwestern Nigeria*. Geological Survey Department, Ministry of Mines, Power and Steel, Nigeria.
- Nigerian Geological Survey Agency. (2006). *Geological and Mineral Map of Ondo State State, Nigeria*.
- Obaje, S. O. (2017). Appraisal of Pavement Failures on Ado-Ekiti–Ogbagi Road, South-Western Nigeria. *International Journal of Geology and Earth Sciences*, 3(2), 1-9.
- Okigbo, N. (2012). Causes of highway failures in Nigeria. *International Journal of Engineering Science and Technology*, 4(11), 4695-4703.
- Okunlola, I. A., Amadi, A. N., Dindey, A. O., & Kolawole, L. L. (2015). Geotechnical and Geochemical Properties of Lateritic Profile on Migmatite-Gneiss along Ogbomosho-Ilorin Highway, Southwestern Nigeria. *Civil and Environmental Research*, 7(4), 79-85.

- Osuolale, O. M., Oseni, A. A., & Sanni, I. A. (2012). Investigation of Highway Pavement Failure along Ibadan - Iseyin Road, Oyo State, Nigeria. *International Journal of Engineering Research & Technology (IJERT)*, 1(8), 1-6.
- Owoseni, J. O., & Atigro, E. O. (2019). Engineering geological investigation of highway pavement failure in basement complex terrain of southwestern Nigeria. *International Journal of Engineering Science and Invention*, 8(6), 14-22.
- Paige-Green, P., & Van Zyl, G. D. (2019). A review of the DCP-DN pavement design method for low volume sealed roads: development and applications. *Journal of Transportation Technologies*, 9(4), 397-422. <https://doi.org/10.4236/jtts.2019.94025>
- Putra Jaya, R., Abdul Hassan, N., & Erdem, S. (2021). Trends and Advanced Materials for Pavement and Road Infrastructure. *Frontiers in Materials*, 8, 762864. <https://doi.org/10.3389/fmats.2021.762864>
- Quadri, H. A., Adeyemi, O. A., & Olafusi, O. S. (2012). Investigation of the geotechnical engineering properties of laterite as a subgrade and base material for road constructions in Nigeria. *Civil and Environmental Research*, 2(8), 23-31.
- Telford, W. M., Geldart, L. P., Sheriff, R. E. (1991). *Applied Geophysics*. Cambridge University Press.
- Transport and Road Research Laboratory. (1990). *A user's manual for a program to analyse dynamic cone penetrometer data (Overseas Road Note 8)*. Crowthorne: Transport Research Laboratory.
- Ubido, O. E., Igwe, O., & Ukah, B. U. (2021). An investigation into the cause of road failure along Sagamu-Papalanto highway southwestern Nigeria. *Geoenvironmental Disasters*, 8(3), 1-19. <https://doi.org/10.1186/s40677-020-00174-8>
- Uz, V. E., Saltan, M., & Gökalp, İ. (2015, September). Comparison of DCP, CBR, and RLT test results for granular pavement materials and subgrade with structural perspective. In *International Symposium on Non-Destructive Testing in Civil Engineering (NDT-CE) September 15 - 17, Berlin, Germany*.
- Vazirani, V. N., & Chandola, S. P. (1996). *Concise Handbook of Civil Engineering*. S Chand Publishing.
- Williams, L. (1997). *Fundamental of Geophysics*. Cambridge University Press.
- Wright, P. H. (1986). *Highway Engineering, Sixth Edition*. John Willey and Sons, New York.
- Yoder, E. J., & Witczak, M. W. (1975). *Principles of Pavement Design, Second Edition*. John Wiley and Sons, New York.
- Zhdanov, M. S., & Keller, G. V. (1994). *The geoelectrical method in geophysics exploration*. Elsevier, Amsterdam.



Copyright (c) 2023 by the authors. This work is licensed under a [Creative Commons Attribution-ShareAlike 4.0 International License](https://creativecommons.org/licenses/by-sa/4.0/).