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Highway Structural Failure and Development of Parameter Prediction Models: Case Study of Akure – Owo (A–122) Pavement, Southwestern Nigeria

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INFORMATION

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ABSTRACT

Soil domain within Akure–Owo (A–122) highway has been studied in order to understand the causes of incessant failure of the highway structure. The study combined geophysical, geochemical, hydrogeological, and geotechnical investigations. The results revealed that the topsoil/subsoil on which the soil is constructed composed of incompetent/fairly competent clay, sandy clay, clay sand, and laterite. The depth to basement rock ranged between 30.4–42.2 m. The soils are lateritic with silica–sesquioxide ratio of 1.58 (avg.). The clay mineralogy is within the illite (60 %)–illite/montmorillonite group (40 %). The soils are of SC-SM of low–intermediate plasticity and compressibility of moderate to high specific gravity. The avg. GI value of the soils is 6, and adjudged fair subgrade soil material. The in-situ California Bearing Ratio (CBR) (avg. 27 %) and soaked CBR (avg. 38 %) satisfied the 10 % minimum specification for subgrade. The DCPT indicated the soil to be generally of medium/stiff/dense consistencies with penetrative index of 1.33–53.67 mm/blow. It also showed that 378–872 mm depths are the suitable surficial layer to host the road structure based on the CBR and SNG with relative densities of 0.371–0.509. The strength coefficient, SNG, SN, and SNP contributions of the soil are good for subgrade but low for subbase and base courses. The regression models of all parameters gave strong positive correlations for soaked CBR and in-situ CBR, and E_R and M_R; while weak positive correlation for in-situ CBR and M_R, RD and DCPI, RD and in-situ CBR. Based on the GI and CBR values, and the traffic count carried out which placed the highway, the recommended thickness of the highway structure should range from 140 mm (good segment) to 445 mm (for weak segment) (avg. 193 mm) which partly corresponds to 315 mm measured along the highway alignment during reconnaissance survey. This implies that the design thickness of the highway corresponds very well with recommended thickness emanating from this study. Thus the failures in some portions along the highway can be attributed to lack of drainage facility at the shoulders of the highway, topography/basement relief, and usage.

1. Introduction

Nigeria roads are classified into federal, state, and local government roads, based on ownership and management of these roads (Okigbo, 2012). The federal roads are divided into Trunks A and F; the Trunk A are roads from inception are built and managed by the federal government of Nigeria; while Trunk F are roads formerly belonging to the State government but taken over by the federal government, and upgraded to standard federal roads.

The state owned roads are called Trunk B roads. The federal government of Nigeria has made tremendous investment and improvement of its road infrastructure networks (over 200, 000 km) since after independence from Britain in 1960 (Okigbo, 2012) by increasing federal, state, and local government roads to 18 %, 16 %, and 66 % respectively, with the local government taking the largest share, yet without corresponding funds from the federal government to (especially local government) to construct/reconstruct or rehabilitate and maintain many of the roads. This has left many local government roads unpaved across the country, however the Federal and State roads are better because of better funding. The federal roads are generally not in stable condition across the nation (Adetoro and Abe, 2018; Akintayo and Osasona, 2022).

Sometimes a highway repair that requires very low budget would be delayed for many months or years due to many official protocols, many of these delays will give room for continued deterioration of failed portions on the highways, as an adage says "a stitch in time saves nine". Pavement subjected to traffic, deform elastically under load, however the elastic deflection depends on factors such as subgrade soil, moisture content, and composition level of subgrade, pavement thickness, composition, quality and condition, drainage conditions, pavement surface temperature, wheel load (Kadyali and Lal, 2008; Wright, 1986; Yoder and Witczak, 1975; Ubido et al., 2021).

All these factors will determine the nature, form, and severity of failures it will manifest. The subgrade is one the most important structural layers of highway pavement that is usually made of natural soil material or reworked soil (Bell, 2007; Amosun et al., 2018). The natural soil is the cheapest and widely used material in any highway system particularly in non-bituminized roads, either in its natural form or in a processed form (stabilized). Hence road pavement structures rest on soil foundation (Bell, 2004). Definitely with heterogeneous and anisotropic nature of soil with varying engineering properties, this can influence considerably by the presence of water in several varieties (Ibitomi et al., 2014).

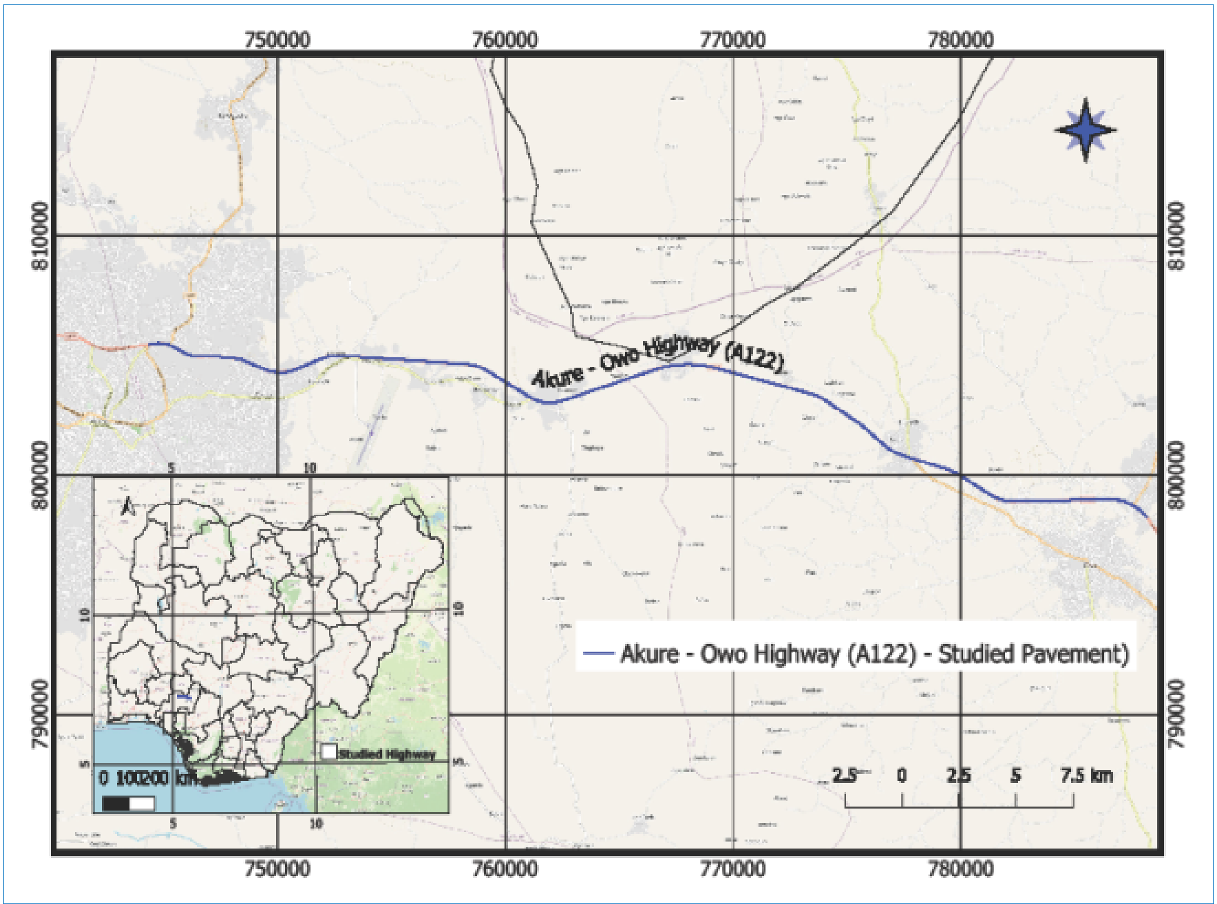

Fig. 1. Map of the Studied Highway. Inset: Map of Nigeria showing the location of Akure–Ondo Highway in southwestern Nigeria

Considering all these aspects, a thorough study of engineering properties of soil is vital and key to successful design of high serviceability pavement. Pavement design is governed by a number of factors, as design life, reliability, traffic factors (wheel load and as its factor), climate factors (weather elements), road geometry, subgrade strength and drainage (Obaje, 2017; Anon, 1952; Brink et al., 1992; Brown, 1996; Cekerevac et al., 2009; Chapman, 1981; Chen, 2000). The Akure–Owo highway is Trunk A road (A-122) connecting central part of Ondo State (like Akure, Ondo, Idanre) to the northern areas like (Owo, Ifon, Akoko) and Benin, Lokoja, Abuja, etc (Fig. 1). It is one of the most important and ever busy highway in Ondo State because of its high economic importance. It is about 50 km from Agbogbo in Akure to Iyere junction in Owo. Presently many segments of the road have failed, hence posing high level of discomfort to human and vehicular movement along the road.

Many of the failed segments manifest as raveling, corrugation, potholes, cracks, and rutting. These failures on the highway, do not befit a highway or expressway which denotes superior type of highway, designed for high speeds (of about 80 km per hour and above), with high traffic volumes and safety. The foremost damaging impact of the failed highway is the rate at which hoodlums and unscrupulous people used to those failed spots to carry out their nefarious activities like robbery and kidnapping; without failing to mention increase in fuel consumption, increase in travel time, damaging of vehicular parts and tyres as a result of jerky motion. Consequently, it has become a matter of outmost importance to assess the index/engineering properties of the soil domain within the highway alignment (Ilori, 2015), as its potential contribution in terms of strength as subgrade, subbase, and base courses (Paige-green and Zyl, 2019; Powell et al., 1984); determine the geological properties of the subsoil and the rock unit beneath the highway (Putra et al., 2021); investigate any geological structure that could be inimical to stability of the highway structure; and make important geotechnical correlations and parameters modeling for the highway in preparation for the rehabilitation/reconstruction of the highway (Vandre et al., 1988; Uz et al., 2015; Ilori, 2015; Kodicherla and Nandyala, 2016).

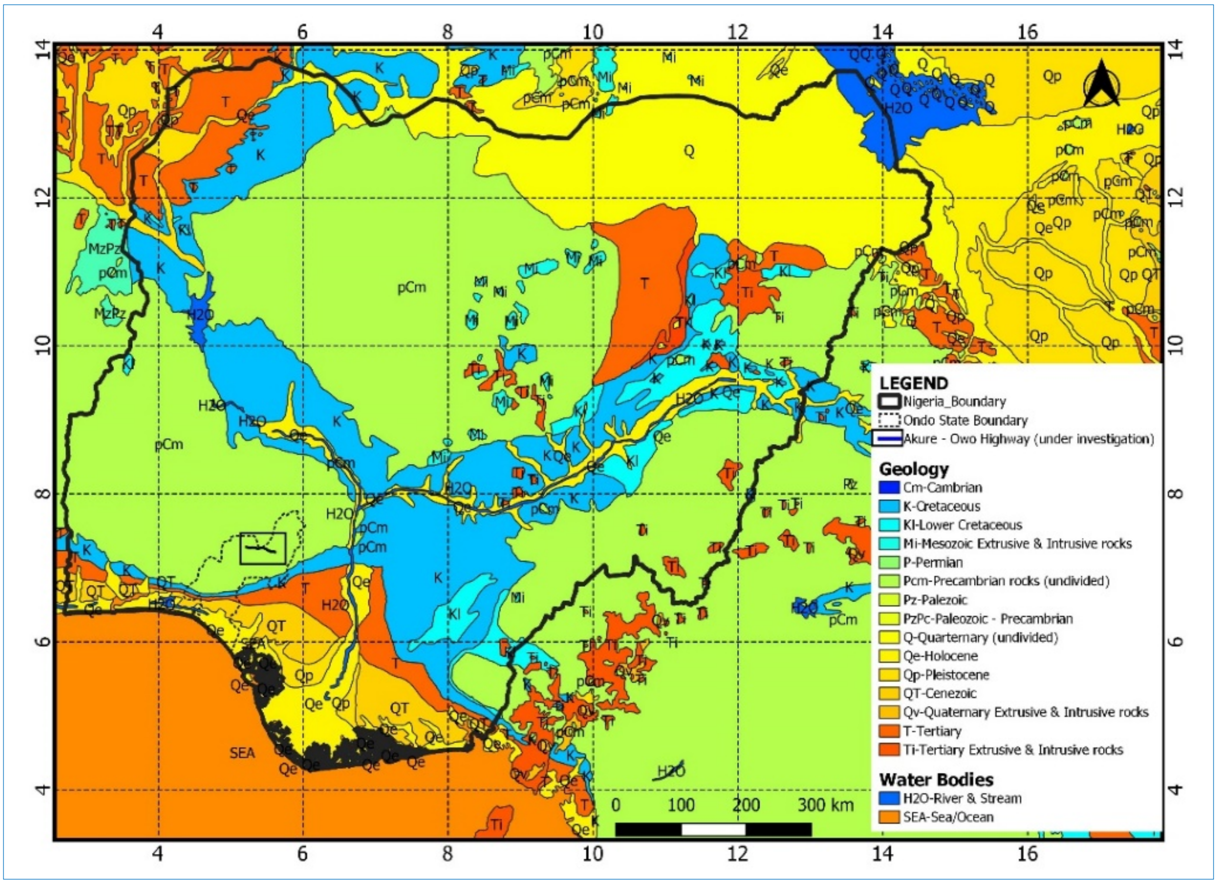

Fig. 2. Geological map of Nigeria showing the highway under investigation (modified after Nigerian Geological Survey Agency, 1984)

2. Materials and Methods

2.1. Site Description and Geology

The Akure–Ondo highway is located within Ondo State, southwestern Nigeria, connecting the central part of the State to the northern parts of the State and Nigeria. Therefore, it connects Akure to Owo, Ipele, Ido ani, Ifon; and Akoko, Benin, Kogi, Lokoja. The road is about 50 km stretch starting (for the purpose of this study) from Agbogbo–Benin garage in Akure (coordinates: 744806 mE, 804710 mN) to Iyere junction in Owo (788572 mE, 797872 mN) of the highway (Fig. 1).

The highway is generally flat, with elevation varying from 312 to 345 m above the sea level. The highway falls within the tropical rainforest climate characterized by rainy and dry seasons (Iloeje, 1981). 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 (Federal Meteorological, 1982). The months of June and Septembers usually experience heavy rainfall with relative humidity of about 80 %, although could be less than 50 % during the dry season (Federal Meteorological, 1982). Geologically, the highway is underlain by Precambrian Southwestern Basement Complex (Adelana et al., 2008; Rahaman, 1988) with migmatites, granite, quartz schist and gneiss being the major rocks observed within the highway alignment (Figs. 2 and 3), as they occur as range of hills of low - moderate altitude. The gneiss is banded with parallel alternation of light and coloured minerals. The migmatite gneiss is strongly foliated, composing of biotite, hornblende, quartz and feldspar. The highway falls within the Ondo (covering Akure–Uso axis) and Okemesi (Amurin–Owo) soil association types (Smith and Montgomery, 1962). The Ondo soil classification 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; while the Okemesi type are very coarse textured, generally pale grayish brown to brown, usually sand soils, often very shallow over quartz rubble, associated with a topography of steep sided elongated ridges. Along the highway, no noticeable side drainage was observed, but the area is characterized by dendritic and trellised drainage systems.

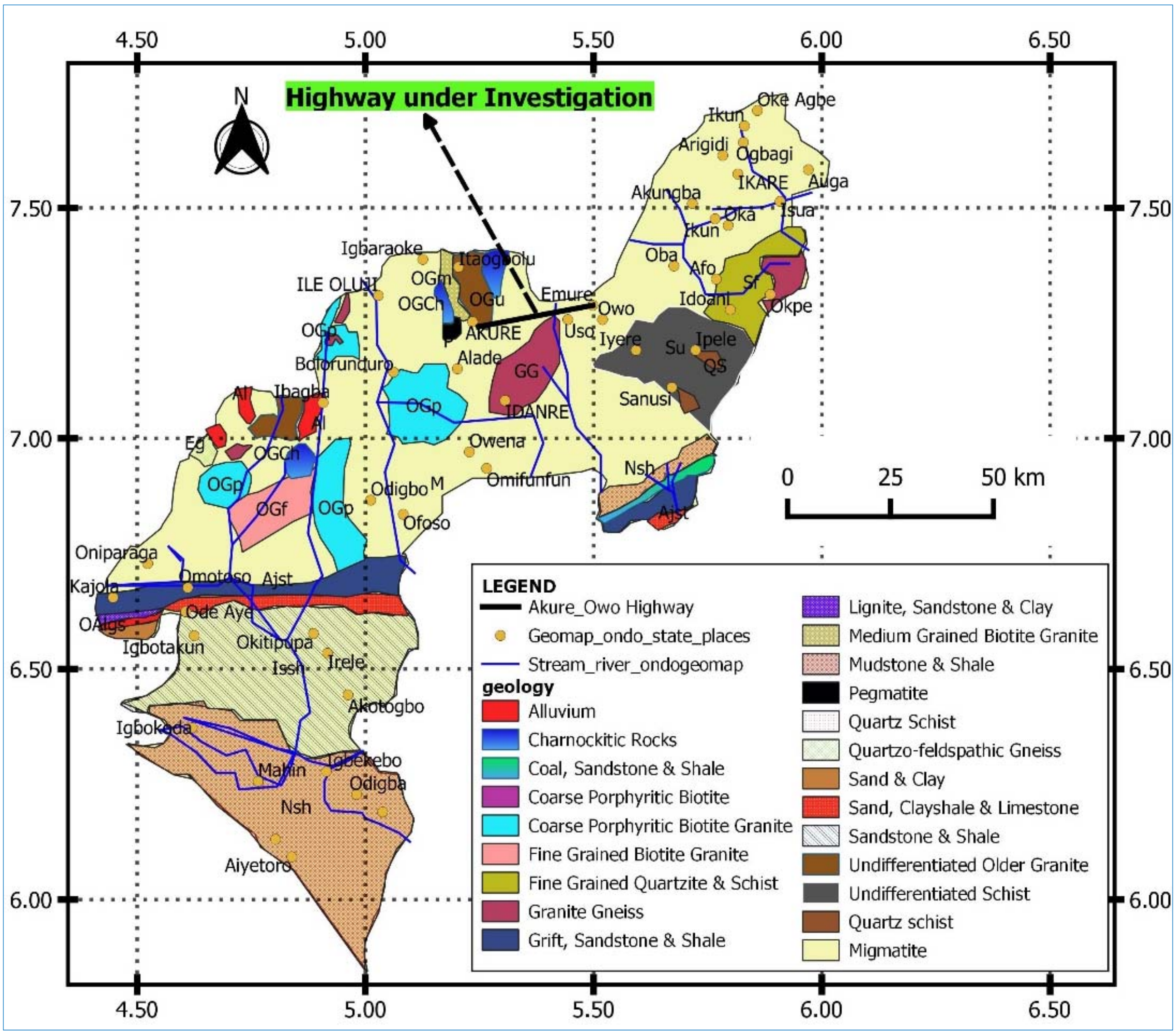

Fig. 3. Geological map of Ondo State showing the road under investigation straddling migmatite, and granite gneiss rock units (modified after Nigerian Geological Survey Agency, 2006)

2.2. Data Acquisition and Analysis

In this study, an integrated Dynamic Cone Penetration Test (DCPT), geophysical survey involving VES; laboratory geotechnical and geochemical analysis; trial pit excavation (depth of 1.0 m); and groundwater level determination (in order to ascertain any spring or artesian well situation along the highway) (Fig. 4). The DCPT was taken along the highway at about 1.0 to 5.0 m offset away from the edge of the highway (Fig. 5).

The DCP is a simple mechanical device used for rapid in-situ strength determination of highway structural material, especially the subgrade and other unbound layers (Vazirani and Chandola, 2009; Scala, 1956; Vandre et al., 1998), and is capable of delivering 45.5 Joules of energy. It measures the penetration of a standard cone when driven by a standard force (Chen et al., 2005). The DCP penetrative index in mm per blows of the standard hammer is recorded together with number of blows and depth of penetration. In this study, the standard steel cone with an angle of 60° and a diameter of 20 mm was used. The standard 8 kg hammer was also utilized which slides over a 16 mm diameter steel rod with a fall height of 575 mm that strikes the anvil to cause penetration (Done and Samuel, 2006).

The test was conducted at ten (10) locations along the highway. This limited test number was due to insecurity that usually characterized failed highways. The UK DCP 3.1 software was used for the analysis and interpretation of the data collected. In calculating the CBR using the Transport and Road Research Laboratory, 1990) relationship, the data recorded at each of the site was corrected for moisture content as shown in Table 1. All the test sites were numbered serially from Test No. 1 to test. No. 10. The hammer factor or coefficient is 1.0 based on the weight of the hammer used (Done and Samuel, 2006). The strength coefficient of the test sites was calculated by the UK DCP 3.1, by converting the penetration rate to CBR value and then to strength coefficient and finally to structural number. The Transport and Road Research Laboratory, 1990) 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 *Equation 2* (for base) and *Equation 3* (for subbase).

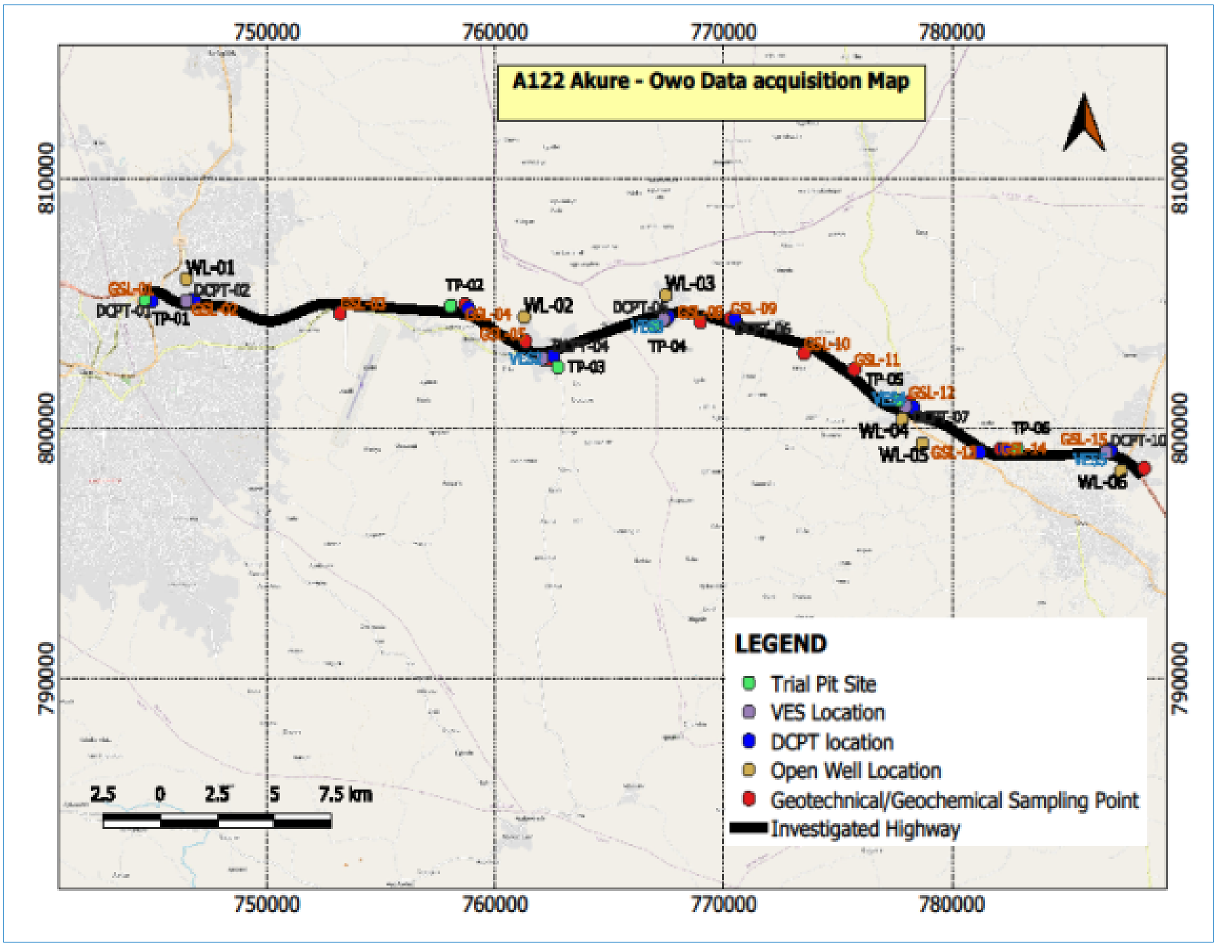

Fig. 4. Data acquisition map for the study showing the geotechnical/geochemical sampling points, geophysical locations, and trial pit points

$$Log_{10}^{(CBR)} = 2.48 - 1.057\, Log_{10}^{(pen\ rate)} \quad 1$$

$$a = 0.0001[29.14\,(CBR) - 0.1977\,(CBR)^2 + 0.00045\,(CBR)^3 \quad 2$$

$$a = 0.0001[29.14\,(CBR) - 0.1977\,(CBR)^2 + 0.00045\,(CBR)^3 \quad 3$$

The subgrade structural number (SNG) which is the contribution of the subsoil as subgrade material to structural number of a pavement (Done and 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*.

$$SNG = 3.51\, Log_{10}^{(CBR)} - 0.85\, Log_{10}^{(CBR)^2} - 1.43 \quad 4$$

The relative densities of each subsoil layering were derived using DIN 4094 model (DIN, 1980) in *Equation 5*, where n_{10} is the number of blows for every 10 cm. The resilient modulus (M_R) using Lockwood et al. (1992), George and Uddin (2000) and Jianzhou et al. (1999) models, as shown in *Equations 6–8* respectively) and Young modulus (E_R) 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 M_R and E_R,

M_R and CBR, dynamic cone penetration index (DCPI) and relative density, and CBR and relative density.

The geophysical investigation helps to detect zone of anomalies by measuring variation in subsurface condition (Kearey et al., 2002; Loke, 2000). They are used to determine the geological sequence and structure of subsurface rocks/soils by the measurement of certain physical properties (Loke, 2004). The properties that are made most use of in geophysical exploration are density, elasticity, electrical conductivity, magnetic susceptibility and gravitational attraction (Sudha et al., 2009; Telford et al., 1991).

In this study, electrical resistivity (vertical electrical sounding) was utilized at five locations along the highway. In this method an electric current is introduced into the ground by means of two current electrodes and the potential difference between two potential electrodes is measured. For this study, the resist-meter used was able to measure the apparent resistance directly in ohms rather than observing both current and voltage. The schlumberger array was used at half current spacing of 65 m. The data obtained (in terms of resistivity and thickness) was plotted as a graph of apparent resistivity against half the current electrode separation. Consequently, the electrode separation at which inflection points occur in the graph gives an idea of the depth/thickness of interphases of the layers and their resistivity. The WinResist software was used for the data analysis involving curve fitting and modeling. The result of the modeling was used to develop the geoelectric section along the highway.

Fig.5. DCPT Field Survey carried out along Akure–Owo Highway at different locations

Table 1. CBR Adjustment Factor (Done and 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

Six trial pits were dug along the highway to study the ground conditions (disturbed/undisturbed), as it gives opportunity to assess directly the weathered rocks. 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, fifteen soil samples were taken at different chainage along the study highway as shown in Fig. 4. They were subjected to geotechnical tests and geochemical tests. The geotechnical tests were conducted using American Standard of Material Testing Methods/Procedures (ASTM, 2006), and these included the CBR (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 diffraction technique. Subsequently, the silica/sesquioxides (se) ratio (Charman, 1988) was calculated to know the type of the soil and classified if laterite (se < 1.33), lateritic (1.33<se>2.0) and non-laterite (se>2.0).

3. Results and Discussion

3.1. Trial Pits

Trial pits can be used for all soil types irrespective of texture, grain size, and mineralogy. It is the cheapest way of site exploration, and do not require any specialized equipment. In this method a pit is manually excavated and soil is inspected in the natural condition. The pit sections (Fig. 6) depict five geologic units across the pavement alignment, consisting of clay-sand mixture, lateritic soil, clay-sand hardpan, sandy clay, and highly cemented clayey soil. At the Akure–Ogbese axis (Trial pits 01 and 02), it is made up of clay sand mixture, sandy clay, laterite and clay sand hardpan. However, the configuration changed when approaching Owo and Ikare junction (Trial pits 04–06) consisting of sandy clay, stiff clayey soil and laterite. Therefore, the soil on which the highway is founded is dominantly clay-sand/sandy clay, which is a fair competent soil.

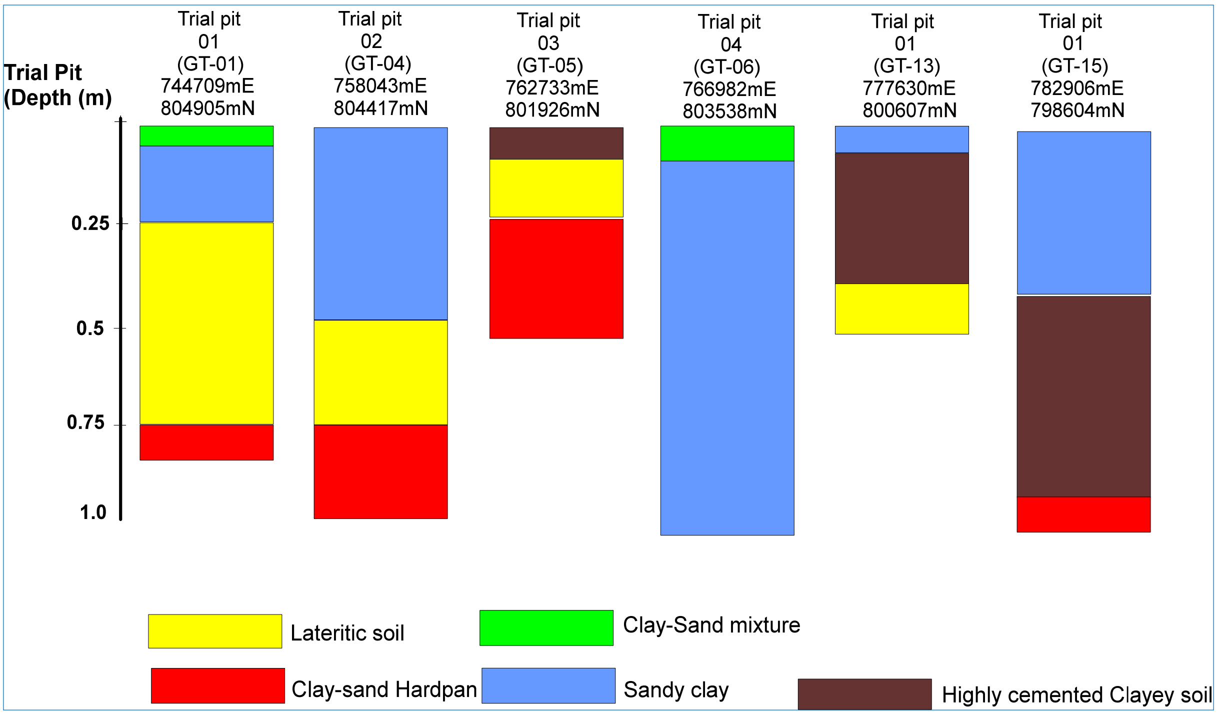

Fig. 6. Trial pit of the three sites investigated along the Highway showing the geologic section

Table 2. Summary of VES results

East	North	Elevation (m)	VES No	Resistivity (Ohmns-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	
746418	804512	344	1	205	658	102	2130		1.0	2.6	31.3		1.0	3.6	34.9		KH
745051	804759	318	2	95	289	53	887		1.2	6.8	22.4		1.2	8.0	30.4		KH
762098	802463	327	3	277	665	110	1242		1.1	5.3	26.4		1.1	6.4	32.8		KH
767324	804026	312	4	321	654	152	3232		1.0	4.8	36.4		1.0	5.8	42.2		KH
777923	800363	320	5	145	26	998			0.8	35.5			0.8	36.3			H
786862	798604	331	6	236	68	1274			0.6	23.4			0.6	24.0			H

3.2. Electrical Resistivity Geophysical Survey

The summary of the VES is presented in Table 2, while the geoelectric along the highway is shown in Fig. 7. The curve types obtained from the highway alignment varies from three layer curves (H) and four layer curves (KH). The KH-curve type is the most preponderant (66.6 %), while H-curve constituted 33.4 %. Geologically, the soil underneath the pavement consist of topsoil, subsoil, weathered layer, fracture basement and fresh basement rock. The H curve is composed of relatively high resistivity topsoil, underlain by very low subsoil/weathered layer, and bedrock; while the KH has a configuration of low resistivity overlain relatively high resistivity subsoil, followed by weathered layer and fresh or fracture or partly weathered basement. The topsoil has resistivity ranging from 26–321 ohm-m and thickness varying from 0.6–1.2 m and composed of clay and sandy clay (using interpretation Table 3). The subsoil delineated under VES 1 - 4 is characterized with resistivity ranging from 289–665 ohm-m composing sandy clay and clay sand. The thickness of this layer ranged from 2.6 to 6.8 m. The weathered layer is clayey and has resistivity ranging between 26 ohm-m and 152 ohm-m. The fracture basement was only observed under VES 2 with resistivity of 887 ohm-m. The fresh basement has resistivity ranging from 998–3232 ohm-m, depths to basement rock varied from 30.4–42.2 m, indicating thick weathering profile.

Consequently, the topsoil, and subsoil are generally composed of clay/sandy clay/clay sand soil material, which can be regarded as incompetent/fairly competent soil material to support the pavement structure. It is observed that the basement relief is valley-like between VES 3 and 5. Consequently this zone will aid groundwater and impoundment of water due to its configuration, hence the high magnitude of failures observed along this zone.

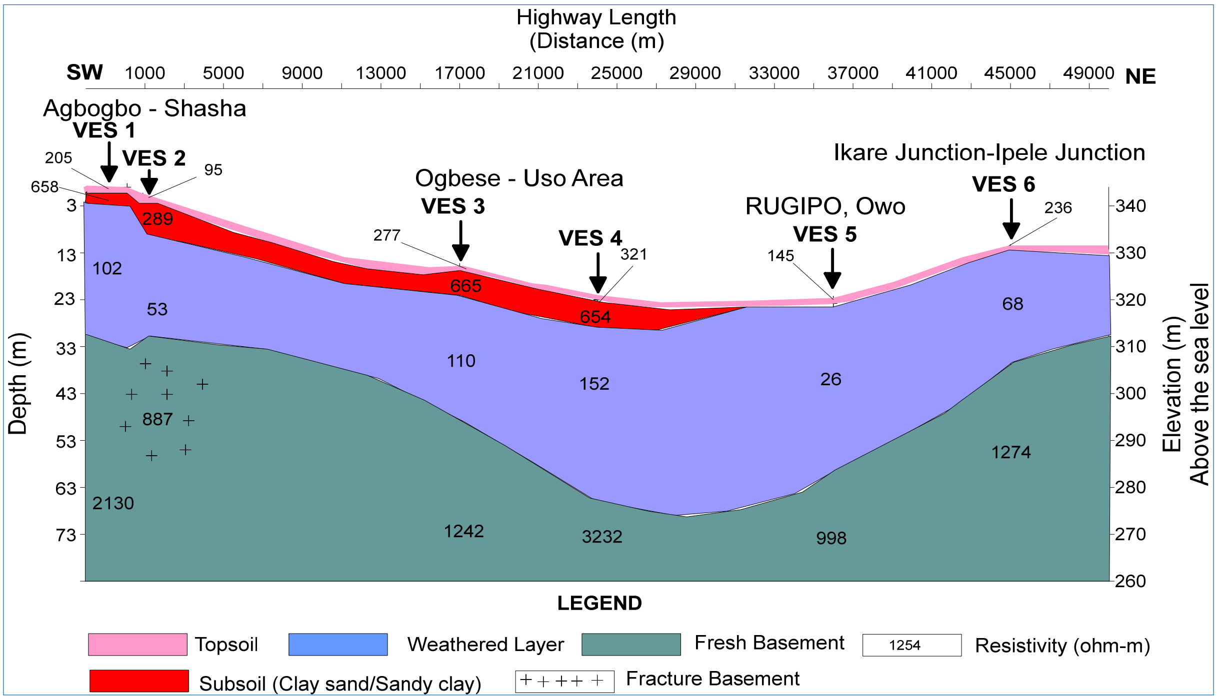

Fig. 7. Geoelectric Section along the Highway Alignment

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

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

Mineral oxide	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10	S11	S12	S13	S14	S15
SiO_2	53.2	55.5	54.36	58.7	60.3	62.5	57.9	56.4	57.2	52.5	56	59.3	60.1	58.8	55.2
Al_2O_3	18.21	16.52	16.22	15.36	17.8	16.5	17.2	17.5	18.22	15.6	18.9	15.5	18.74	17.22	17.9
Fe_2O_3	20.32	20.21	21.56	19.95	18.54	16.32	15.87	20.5	23.72	19.41	18.33	20.59	20.47	15.15	18.6
Sesquioxide ratio	1.38	1.51	1.44	1.66	1.66	1.90	1.75	1.48	1.36	1.50	1.50	1.64	1.53	1.82	1.51
Soil Type	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic

3.3. Geochemical Analysis

The stability and serviceability performance depends on the mineralogical make-up of the soil (Blyth and de Freitas. 1986). The result of chemical analysis (oxides) of the major elements (SiO_2, Fe_2O_3, and Al_2O_3) contained in the soil samples (Table 4), and silica-sesquioxide (S-S) ratio is presented in Table 4. The samples are well dominated by SiO_2 - Fe_2O_3 - Al_2O_3, ranging from 52.5–62.5 % (avg. 57.2 %), 15.2–23.7 % (avg. 19.3 %), and 15.4–18.9 % (avg. 17.2 %) respectively. S-S ratio of the samples ranged from 1.36 to 1.90 (avg. of 1.58). Accordingly, soils with S-S ratio between 1.33 and 2.0 are categorized as lateritic soil type. This corroborate the lateritic soil observed from the trial pit sections.

3.4. Geotechnical Analysis

Table 5 presents the summary of the geotechnical results. The natural moisture content varied from 5.8 to 19.2 % (avg. 12.9 %), this range is within the 5 – 15 % acceptable range for civil engineering uses or construction. 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 – 1.2 % (avg. 0.57 %) and 46.6 – 56.4 % (avg. 51.2 %) respectively. The % silt and clay contents ranged from 15.3 to 24.3 % (avg. 20.0 %) and 25.3 to 32.5 % (avg. 28.2%).

The % fines ranged from 42.6 to 53.4 (avg. 48.2). The composition of the soil is dominated (in order of magnitude) by sand, clay, and silt (SC-SM). The amount of % fines recorded is more than 35% specification of Federal Ministry of Works and Housing (1997) for highway subgrade. The plasticity chart (Fig. 8a) shows that the fines in the samples is dominated by clay of low to intermediate plasticity/ compressibility. All the soil samples plotted above the A-line. In terms of clay mineralogy, the soil samples are plotted within the range of llite and montmorillonite clay mineralogy group, majorly 60% belong to illite group (Fig. 8b). Montmorillonite is made up of two silica sheets and one gibbsite sheet and bonded by vander wall forces between the tops of silica sheets is weak and there's negative charge deficiency, water and exchangeable ions can enter and separate the layers (Bell, 2007). Hence montmorillonite has a very strong attraction for water and swells on absorption of water. Illite has a similar structure similar to montmorillonite, however in illite the interlayers are bonded together with a potassium ion linkage, making it to have relatively less attraction for water (Blyth and de Freitas, 1984). Therefore, it is expected that the soil will exhibits more of illite characteristics. The activity ranged from 0.61 to 0.75 (avg. 0.70) signifying inactive clay type.

Table 5. Summary of the geotechnical properties of the investigated soil

Parameters	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10	S11	S12	S13	S14	15
East	744806	746711	753305	758678	761267	762342	767568	768985	770352	773576	775774	778070	781147	782124	786813
North	804710	804417	804368	804270	802854	802316	803977	804005	803684	802658	801877	800411	798604	798555	798507
NMC	12.2	10.5	5.8	15.3	8.2	13.4	9.7	15.2	16.5	9.3	14.3	19.2	15.5	18.7	9.6
%Gravel	1.2	0	1	0	1.2	1.2	1	0	0	0	1	0	0	1	1
%Sand	48.5	50.1	49.6	48.4	53.3	53.5	49.7	50.6	55.2	53.6	47.9	46.6	51.1	53.8	56.4
%Silt	17.8	24.3	20.6	21.5	17.7	19	20.9	22.8	15.3	21.1	20.2	24.3	20.7	18.6	15.5
%Clay	32.5	25.6	28.8	30.1	27.8	26.3	28.4	26.6	29.5	25.3	30.9	29.1	28.2	26.6	27.1
%Fines	50.3	49.9	49.4	51.6	45.5	45.3	49.3	49.4	44.8	46.4	51.1	53.4	48.9	45.2	42.6
SG	2.665	2.7	2.689	2.688	2.695	2.692	2.684	2.697	2.705	2.699	2.684	2.686	2.691	2.695	2.699
LL (%)	42.2	45.6	44.8	46.3	39.8	40.7	48.3	39.2	37.8	36.6	42.1	40.8	38.5	37.2	38.4
PL (%)	20.1	26.3	24	23.8	20.3	22.1	27.8	19.6	19.4	18.3	23.2	21.1	20	18.4	19.2
PI (%)	22.1	19.3	20.8	22.5	19.5	18.6	20.5	19.6	18.4	18.3	18.9	19.7	18.5	18.8	19.2
SL	12.4	9.9	10.1	10.8	9.2	8.6	10.9	10.2	9.5	8.7	12.8	13.4	9.5	8.0	8.6
CBR soaked	5	15	15	32	44	13	15	26	28	30	25	48	42	33	40
In-situ CBR	7	20	-	45	51	-	-	34	-	40	-	52	48	38	44
MDD	1789	1987	1785	1853	2007	1998	1876	1890	1965	1936	1782	1811	1980	2013	2055
OMC	18.5	16.5	12.8	21.4	14.5	19.6	16.2	20.7	20.1	14.4	21.9	22.6	18.3	20.5	15.5
GI	7	7	7	8	5	5	7	6	4	5	7	7	6	5	4
GI Class	Fair	Fair	Fair	Fair	Fair	Fair	Fair	Fair	Good	Fair	Fair	Fair	Fair	Fair	Good
Rec. thickness (mm)	445	224	224	150	140	257	224	163	157	152	175	140	145	150	155
AASHTO	A-7-5	A-7-6	A-7-5	A-7-5	A-6	A-6	A-7-5	A-6	A-6	A-6	A-7-5	A-6	A-6	A-6	A-6
USCS	CL	ML	CL	CL	CL	ML-CL	ML-CL	ML-CL	CL	CL	CL	CL	CL	CL	CL
Subgrade Rating	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor	Fair/ poor
Activity	0.68	0.75	0.72	0.75	0.70	0.71	0.72	0.74	0.62	0.72	0.61	0.68	0.66	0.71	0.71
Clay type	Inactive	Normal	Inactive	Normal	Inactive	Inactive	Inactive	Inactive	Inactive	Inactive	Inactive	Inactive	Inactive	Inactive	Inactive
Clay mineralogy	I-M	I	I-M	I-M	I-M	I	I	I-M	I-M	I-M	I	I-M	I	I-M	I-M

The specific gravity (SG) is closely related with soil's mineralogy and/or chemical contents; the higher SG, the higher the degree of laterization (Bell, 2004).

In addition, the larger the clay fraction and alumina contents, the lower is the SG. The values of specific gravity of the samples ranged between 2.665–2.705 (avg. 2.691). The standard range of value of specific gravity of soils for civil engineering construction lies between 2.60 and 2.80, these values are considered normal; hence the soils are competent.

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 36.6 to 48.3 % (avg. 41.2 %), plastic limits (PL) ranged between 18.3 to 27.8 % (avg. 21.57 %) and plasticity index (PI) is between to 18.3 to 22.5 % (avg. 19.7 %).
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. Soil with high LL, PL, and PI are usually characterized with low bearing pressure. Hence the soils satisfied these requirements as subgrade material. The linear shrinkage ranged between 8 to 13 % (avg. 10.2 %), signifying a poor swelling potential, as SL greater than 8.0 tends to be active, of critical swelling potential.

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 (IRC, 2002; FHWA, 2006).

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 1782 and 2055 kg/m^3 (1915 kg/m^3) at standard proctor compaction energy while the optimum moisture content (OMC) ranged between 12.8 and 22.6 % (18.2%). An important part of the grading of the site often includes the compaction of fill. All the soil samples have high MDD at moderate low OMC. The 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 (Holtz and Kovacs, 1981). All compacted samples show soaked CBR values ranging between 5 and 48% (avg. 27%), with corresponding in-situ values obtained from DCPT ranging from 7 to 52% (avg. 38%).

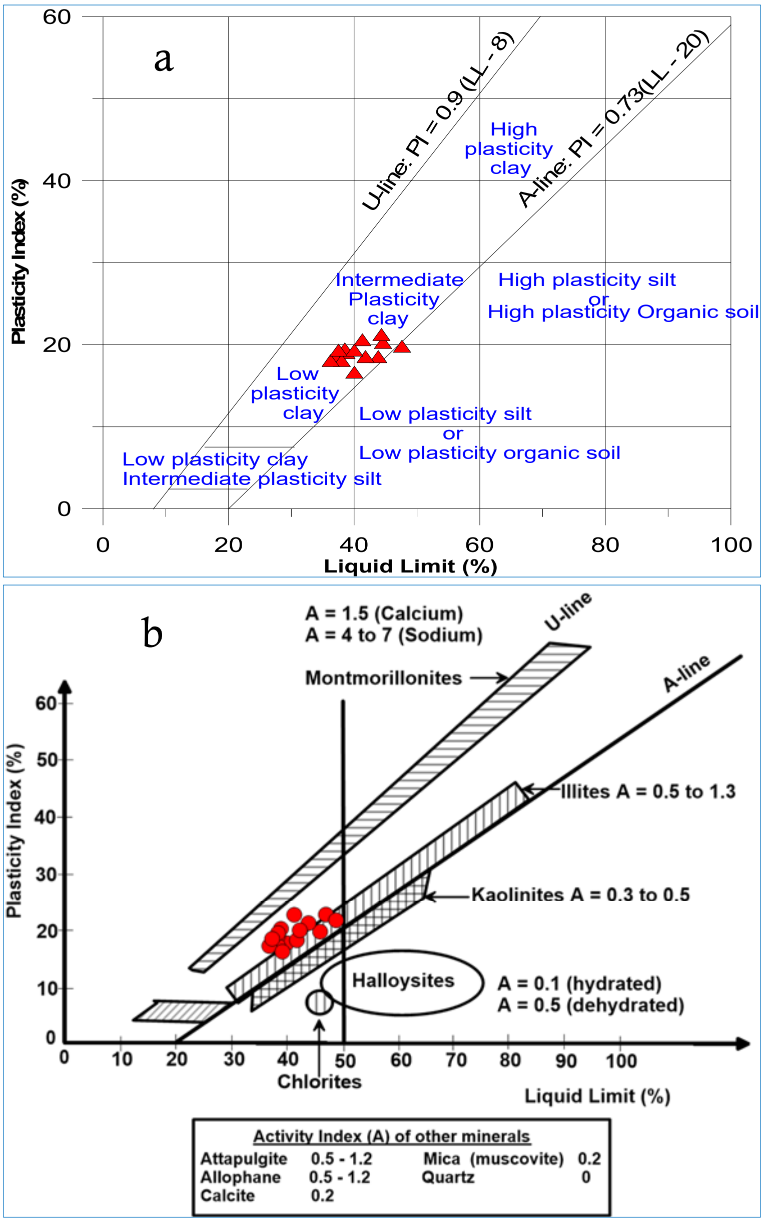

Fig. 8. (a) Plasticity Chart for Fine Contents of the soil samples (b) Clay mineralogy group of the soil samples with most within/or near the illite

Table 6. Subgrade strength classification for the studied highway (Carter and 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

The Federal Ministry of Works and Housing recommends a CBR of greater than 10% for subgrade materials. Therefore, using Table 6, the soils are rated as high (based on average value) as pavement subgrade material. The Group Index (GI) values obtained ranged from 4 to 8 (avg. 6) corresponding to fair subgrade soil. The result shows that the CBR values of the soils both in-situ and laboratory satisfied the 10 % minimum specification. 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 140 mm (good segment) to 445 mm (for weak segment) (avg. 193 mm) as shown in Fig. 9, which partly corresponds to 315 mm measured along the highway alignment during reconnaissance survey (Fig. 10).

This implies that the recommended design thickness of at least 193 mm was appropriately adopted during the design and construction of the highway.

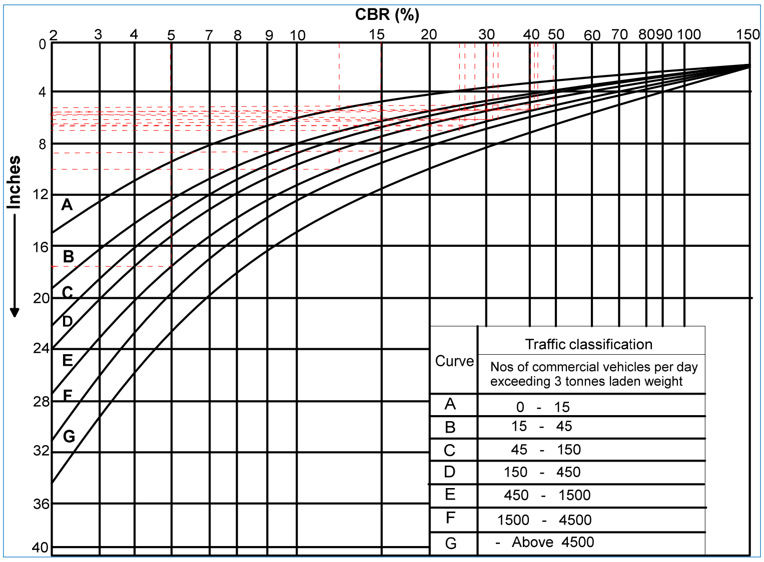

Fig. 9. The CBR Chart adopted for determine the recommended thickness across the highway alignment

Fig. 10. Section of the exposed highway structure at Ogbese, where existing design thickness was measured to be 315 mm

3.5. 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 is shown Figs. 11–12. In Table 7, the degree of penetration ranged from 519 mm (Test 7)–955 (Test 2) mm, with cumulative number of blows ranging from 49 (Test 7) to 102 (Tests 5 and 6). The penetrative index or rate ranged between 1.33 mm/blow (Test 9)–53.67 mm/blow (Test 1) with depth/penetration zone of 839/872 mm and 161/193 mm respectively. In terms of relative density (RD), all the tests are characterized by moderate–high cumulative number of blows in the upper 1 m investigated, signifying a medium/stiff/dense consistencies of relative densities of 0.371 to 0.509 (Table 8).

However, the upper 10 cm of Tests 1–6 is loose (RD of 0.320); while Tests 9 and 10 showed loose zones between 20 and 80 cm. With respect to layering, two layers (Tests 1, 3, 7 and 8), three layers (Tests 2, 4, 5, 9 and 10), and four layers (Tests 5 and 6) were delineated. The obtained CBR ranged from 4 to 54%. The most competent layers in terms of the obtained CBR are generally between 378 mm to 872 mm.

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

Point	Blow	Penetration (mm)	Cum. blows	Depth (mm)	Penetration rate (mm/blow)	Blow	Penetration (mm)	Cum. blows	Depth (mm)	Penetration rate (mm/blow)	Blow	Penetration (mm)	Cum. blows	Depth (mm)	Penetration rate (mm/blow)
	Test 1: 744953mE; 804808mN; CH 0 + 0.005 km RHS					Test 2: 746809mE; 804514mN; CH 0 + 2.55 km LHS					Test 3: 758825mE; 804270mN; CH 0 + 14.5 km LHS				
1	0	32	0	0	0	0	32	0	0	0	0	31	0	0	0
2	3	193	3	161	53.67	3	174	3	142	47.33	3	88	3	57	19.0
3	3	258	6	226	21.67	3	232	6	200	19.33	3	118	6	87	10.0
4	3	314	9	282	18.67	3	293	9	261	20.33	5	158	11	127	8.0
5	3	383	12	351	23.0	3	355	12	323	20.67	5	214	16	183	11.20
6	3	432	15	400	16.33	3	399	15	367	14.67	5	369	21	338	31.0
7	3	486	18	454	18.0	3	447	18	415	16.0	5	441	26	410	14.40
8	3	524	21	492	12.67	3	492	21	460	15.0	5	464	31	433	4.60
9	3	565	24	533	13.67	3	529	24	497	12.33	5	532	36	501	13.60
10	3	621	27	589	18.67	3	578	27	546	16.33	5	585	41	554	10.60
11	3	709	30	677	29.33	3	652	30	620	24.67	5	636	46	605	10.20
12	3	820	33	788	37.0	3	806	33	774	51.33	5	694	51	663	11.60
13	3	921	36	889	33.67	3	904	36	872	32.67	5	752	56	721	11.60
14	1	950	37	918	29.0	2	933	38	901	14.50	5	788	61	757	7.20
15	-	-	-	-	-	2	955	40	923	11.0	5	821	66	790	6.60
16	-	-	-	-	-	-	-				5	843	71	812	4.40
17	-	-	-	-	-	-	-	-	-	-	5	874	76	843	6.20
18	-	-	-	-	-	-	-	-	-	-	5	899	81	868	5.00
19	-	-	-	-	-	-	-	-	-	-	5	936	86	905	7.40
20	-	-	-	-	-	-	-	-	-	-	5	954	91	923	3.60
	Test 4: 762391mE; 802414mN; CH 0 + 19.0 km RHS					Test 5: 767422mE; 804124mN; CH 0 + 27.5 km RHS					Test 6: 770499mE; 803733mN; CH 0 + 30.6 km LHS				
1	0	33	0	0	0	0	22	0	0	0	0	21	0	0	0
2	3	96	3	63	21.0	3	91	3	69	23.0	3	100	3	79	26.33
3	3	121	6	88	8.33	3	228	6	206	45.67	3	241	6	220	47.0
4	5	172	11	139	10.20	3	285	9	263	19.0	3	273	9	252	10.67
5	5	236	16	203	12.80	3	319	12	297	11.33	3	322	12	301	16.33
6	5	401	21	368	33.0	5	368	17	346	9.80	5	380	17	359	11.60
7	5	478	26	445	15.40	5	402	22	380	6.80	5	426	22	405	9.20
8	5	503	31	470	5.0	5	445	27	423	8.60	5	473	27	452	9.40
9	5	583	36	550	16.0	5	485	32	463	8.0	5	509	32	488	7.20
10	5	638	41	605	11.0	5	525	37	503	8.0	5	541	37	520	6.40
11	5	690	46	657	10.40	5	573	42	551	9.60	5	597	42	576	11.20
12	5	752	51	719	12.40	5	605	47	583	6.40	5	633	47	612	7.20
13	5	809	56	776	11.40	5	628	52	606	4.60	5	664	52	643	6.20
14	5	831	61	798	4.40	5	643	57	621	3.0	5	691	57	670	5.40
15	5	854	66	821	4.60	5	656	62	634	2.60	5	704	62	683	2.60
16	5	883	71	850	5.80	5	689	67	667	6.60	5	732	67	711	5.60
17	5	912	76	879	5.80	5	709	72	687	4.0	5	753	72	732	4.20
18	5	934	81	901	4.40	5	752	77	730	8.60	5	798	77	777	9.0
19	5	963	86	930	5.80	5	801	82	779	9.80	5	841	82	820	8.60
20	-	-	-	-	-	5	833	87	811	6.40	5	866	87	845	5.0
21	-	-	-	-	-	5	874	92	852	8.20	5	895	92	874	5.80
22	-	-	-	-	-	5	901	97	879	5.40	5	922	97	901	5.40
23	-	-	-	-	-	5	933	102	911	6.40	5	946	102	925	4.80
	Test 7: 778265mE; 800118mN; CH 0 + 38.9 km LHS					Test 8: 781196mE; 798653mN; CH 0 + 41.0 km RHS					Test 9: 782320mE; 798360mN; CH 0 + 42.2 km LHS				
1	0	30	0	0	0	0	31	0	0	0	0	33	0	0	0
2	3	90	3	60	20.0	3	95	3	64	21.33	3	116	3	83	27.67
3	3	133	6	103	14.33	3	140	6	109	15.0	3	151	6	118	11.67
4	3	179	9	149	15.33	3	188	9	157	16.0	3	199	9	166	16.0
5	5	269	14	239	18.0	5	283	14	252	19.0	3	285	12	252	28.67
6	5	385	19	355	23.20	5	409	19	378	25.20	3	430	15	397	48.33
7	5	424	24	394	7.80	5	447	24	416	7.60	3	600	18	567	56.67
8	5	446	29	416	4.40	5	460	29	429	2.60	3	753	21	720	51.0
9	5	469	34	439	4.60	5	492	34	461	6.40	3	810	24	777	19.0
10	5	490	39	460	4.20	5	520	39	489	5.60	3	831	27	798	7.0
11	5	507	44	477	3.40	5	556	44	525	7.20	3	843	30	810	4.0
12	5	519	49	489	2.40	5	573	49	542	3.40	3	855	33	822	4.0
13	-	-	-	-	-	5	592	54	561	3.80	3	868	36	835	4.33
14	-	-	-	-	-	5	648	59	617	11.20	3	872	39	839	1.33
15	-	-	-	-	-	5	667	64	636	3.80	-	-	-	-	-
16	-	-	-	-	-	5	683	69	652	3.20	-	-	-	-	-
17	-	-	-	-	-	5	691	74	660	1.60	-	-	-	-	-
18	-	-	-	-	-	5	705	79	674	2.80	-	-	-	-	-
	Test 10: 786911mE; 798507mN; CH 0 + 46.8 km LHS														
1	0	33	0	0	0										
2	3	120	3	87	29.0										
3	3	158	6	125	12.67										
4	3	209	9	176	17.0										
5	3	299	12	266	30.0										
6	3	450	15	417	50.33										
7	3	582	18	549	44.0										
8	3	693	21	660	37.0										
9	3	777	24	744	28.0										
10	3	801	27	768	8.0										
11	3	836	30	803	11.67										

12	3	854	33	821	6.0
13	3	872	36	839	6.0
14	3	889	39	856	5.67
15	3	895	42	862	2.0
16	3	901	45	868	2.0

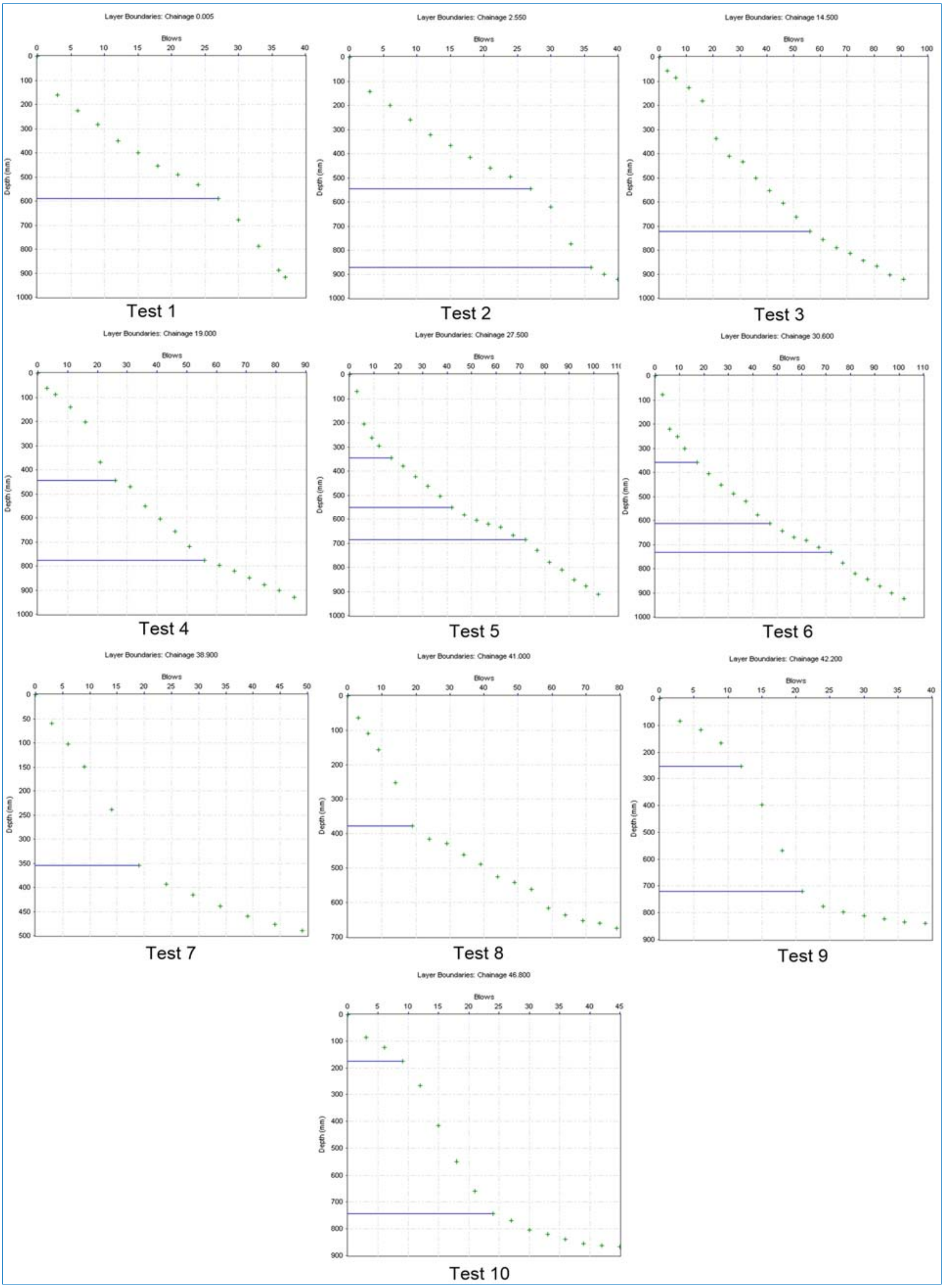

Fig. 11. The plot of cumulative blows against depth at Test points 1–10 showing the layering within the upper 1.0 m

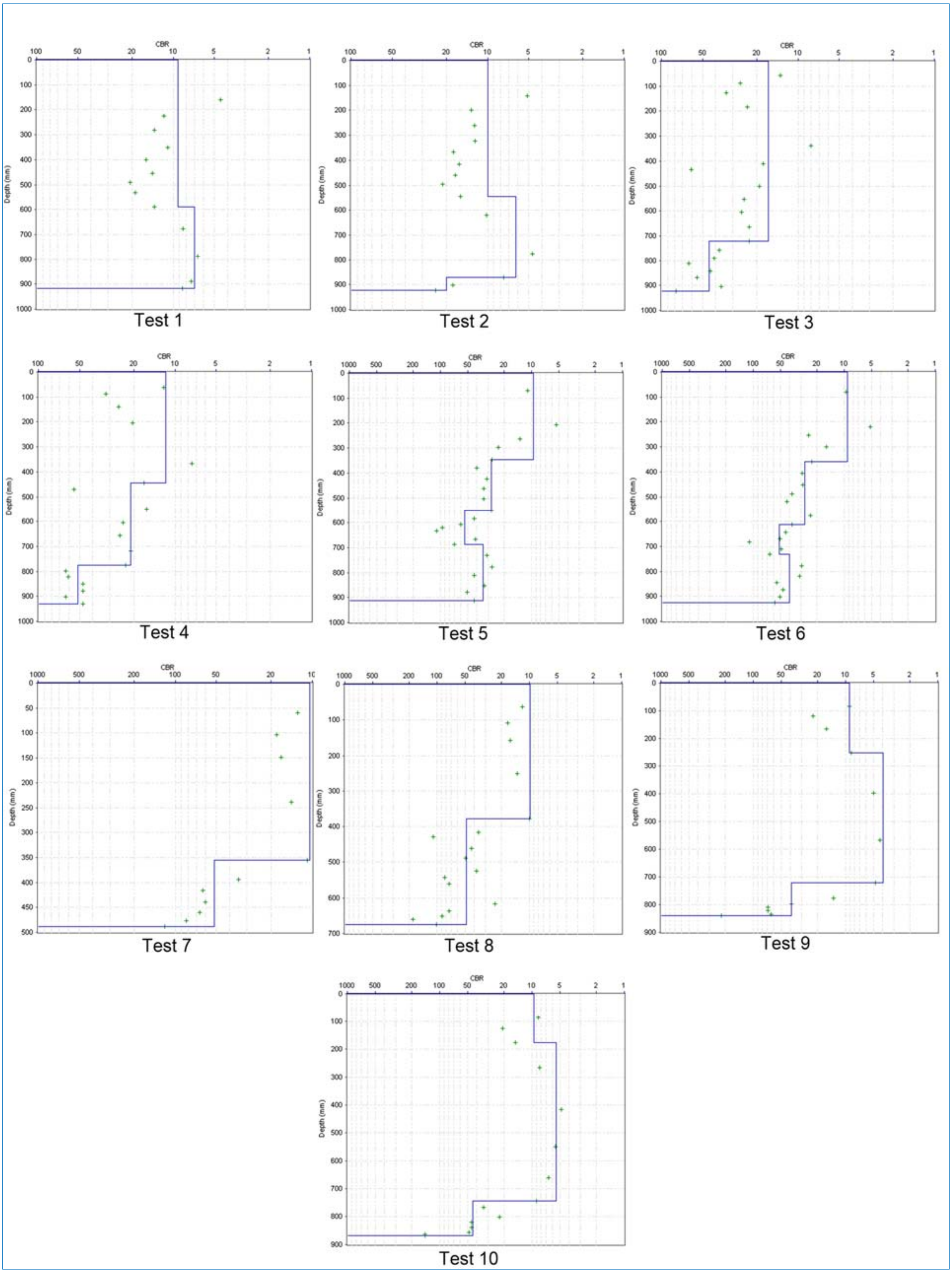

Fig. 12. The plot of CBR against depth at test points 1–10, showing the CBR of the layers

The estimated relative densities (RD) gives consistencies of the soil either very dense, dense, medium, loose or very loose, however showed layering not totally consistent with those observed from DCPI. The SNG contribution of the soil as subgrade material ranged from -0.16 (Test 10) to 1.71 (Test 5). This range of values is fairly above 0.5 SNG strength coefficient for subgrade pavement layer. Consequently, relating the CBR and SNG, the appropriate depths for Tests 1 and 2 are 589 mm and 872 mm respectively, 721 mm (Test 3), 776 mm (Test 4), 551 mm (Test 5), 612 mm (Test 6), 355

(Test 7), 378 mm (Test 8), 720 mm (Test 9) and 744 (Test 10). The strength coefficient of the soil as subbase and base is less than 0.5 and ranged from 0.02–0.11, and 0.01–0.11, with Structural Number (SN)/SNC and SNP ranging from 1.44 to 3.22 and 0.98 to 2.02; and 0.84 to 2.16 and 0.84 to 2.16, respectively (Table 9). From the values, the strength coefficient is generally low for subbase and base material.

The Young Modulus (E_R) and Resilient Modulus (M_R) was estimated from Lockwood et al. (1992), Jianzhou et al. (1999), and George and Uddin (2000); and the E_R varied between 34.93–762.09 (avg. 295.032), 89.43–272.05 (avg. 178.955), and 43.1–181.03 (avg. 108.196); the M_R ranged from 49.89 to 824.31 (avg. 326.898), 107.94 to 302.42 (avg. 203.278), and 58.59 to 205.49 (avg. 127.919) respectively. The Lockwood et al. (1992) and George and Uddin (2000) showed closely overlapping values, while Jianzhou et al. (1999) showed a wide variation (Table 10) in the values of E_R and M_R. Resilient modulus (M_R) is a measure of subgrade material stiffness. It is a means of estimating modulus of elasticity (E_R) of rapidly applied loads as against slowly applied load used for E_R.

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

Parameters	Values								
Test 1									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	3	3	6	6	6	3	3	3	4
Relative density	0.320	0.429	0.429	0.429	0.429	0.320	0.320	0.320	0.348
Soil consistency	Loose	Loose	Medium	Medium	Medium	Loose	Loose	Loose	Loose
Penetration rate (mm/blow)	-	-	18.67	16.33	12.67	-	-	-	30.67
Test 2									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	3	6	6	6	6	3	3	3	7
Relative density	0.320	0.429	0.429	0.429	0.429	0.320	0.320	0.320	0.404
Soil consistency	Loose	Medium	Medium	Medium	Medium	Loose	Loose	Loose	Medium
Penetration rate (mm/blow)	-	19.33	-	-	12.33	-	-	-	14.5
Test 3									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	3	5	5	10	10	10	10	20	20
Relative density	0.320	0.371	0.371	0.440	0.440	0.440	0.440	0.509	0.509
Soil consistency	Loose	Medium	Medium	Medium	Medium	Medium	Medium	Stiff	Stiff
Penetration rate (mm/blow)	-	-	-	13.4	13.6	10.2	-	5.6	7.4
Test 4									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	3	5	5	10	10	10	5	20	20
Relative density	0.320	0.371	0.371	0.440	0.440	0.440	0.371	0.509	0.509
Soil consistency	Loose	Medium	Medium	Medium	Medium	Medium	Medium	Stiff	Stiff
Penetration rate (mm/blow)	-	12.8	-	-	-	11.0	-	4.4	4.4
Test 5									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	3	6	6	15	10	25	10	15	20
Relative density	0.320	0.429	0.429	0.481	0.440	0.532	0.440	0.481	0.509
Soil consistency	Loose	Medium	Medium	Medium	Medium	Stiff	Medium	Medium	Stiff
Penetration rate (mm/blow)	-	45.67	11.33	-	18.0	4.6	-	-	6.4
Test 6									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	3	6	8	10	15	15	20	15	20
Relative density	0.320	0.429	0.418	0.440	0.481	0.481	0.509	0.481	0.509
Soil consistency	Loose	Medium	Medium	Medium	Medium	Medium	Stiff	Medium	Stiff
Penetration rate (mm/blow)	-	-	16.33	9.20	6.0	-	4.6	-	5.4
Test 7									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	6	5	5	15	20	-	-	-	-
Relative density	0.429	0.371	0.371	0.481	0.509	-	-	-	-
Soil consistency	Medium	Medium	Medium	Medium	Stiff	-	-	-	-
Penetration rate (mm/blow)	14.33	-	-	7.8	2.4	-	-	-	-
Test 8									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	6	5	5	15	20	20	20	-	-
Relative density	0.429	0.371	0.371	0.481	0.509	0.509	0.509	-	-
Soil consistency	Medium	Medium	Medium	Medium	Stiff	Stiff	Stiff	-	-
Penetration rate (mm/blow)	15	-	-	9.6	-	6.4	-	-	-
Test 9									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	9	3	3	3	3	3	3	18	-
Relative density	0.429	0.320	0.320	0.320	0.320	0.320	0.320	0.499	-
Soil consistency	Medium	Loose	Loose	Loose	Loose	Loose	Loose	Medium	-
Penetration rate (mm/blow)	-	-	-	48.33	-	-	51.0	7.0	-
Test 10									
Depth (cm)	10	20	30	40	50	60	70	80	90
Blows per 10 cm	6	6	3	3	3	3	3	15	18
Relative density	0.429	0.429	0.320	0.320	0.320	0.320	0.320	0.481	0.499
Soil consistency	Medium	Medium	Loose	Loose	Loose	Loose	Loose	Medium	Medium
Penetration rate (mm/blow)	12.67	23.33	-	46.33	-	37.67	-	11.67	-

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 SNG	Position	Strength coefficient	Pavement Strength/Layer contribution SN	SNC	SNP	Position	Strength coefficient	Pavement Strength/Layer contribution SN	SNC	SNP
1	1	9	589	589	0.30	Sub-Base	0.06	2.05	2.05	1.39	Base	0.03	0.84	0.84	0.84
	2	7	329	918	0.10	Sub-Base	0.05				Base	0.02			
2	1	10	846	546	0.38	Sub-Base	0.06	2.11	2.11	1.43	Base	0.03	0.91	0.91	0.91
	2	6	326	872	-0.02	Sub-Base	0.04				Base	0.02			
	3	20	51	923	0.92	Sub-Base	0.09				Base	0.05			
3	1	17	721	721	0.80	Sub-Base	0.08	3.22	3.22	2.02	Base	0.04	1.97	1.97	1.97
	2	45	202	923	1.56	Sub-Base	0.11				Base	0.10			
4	1	12	445	445	0.52	Sub-Base	0.07	3.09	3.09	1.86	Base	0.03	1.87	1.87	1.87
	2	21	331	776	0.96	Sub-Base	0.09				Base	0.05			
	3	51	154	930	1.66	Sub-Base	0.11				Base	0.10			
5	1	9	346	346	0.30	Sub-Base	0.06	3.14	3.14	1.87	Base	0.03	2.15	2.15	2.15
	2	27	205	551	1.16	Sub-Base	0.10				Base	0.07			
	3	54	136	687	1.71	Sub-Base	0.11				Base	0.11			
	4	34	224	911	1.34	Sub-Base	0.10				Base				
6	1	9	359	359	0.30	Sub-Base	0.06	3.16	3.16	1.84	Base	0.03	2.16	2.16	2.16
	2	27	253	612	1.16	Sub-Base	0.10				Base	0.07			
	3	52	120	732	1.68	Sub-Base	0.11				Base	0.10			
	4	40	193	925	1.47	Sub-Base	0.11				Base	0.09			
7	1	10	355	355	0.38	Sub-Base	0.07	1.51	1.51	1.28	Base	0.03	0.94	0.94	0.94
	2	52	134	489	1.68	Sub-Base	0.11				Base	0.10			
8	1	10	378	378	0.38	Sub-Base	0.06	2.22	2.22	1.60	Base	0.03	1.56	1.56	1.56
	2	48	296	674	1.61	Sub-Base	0.11				Base	0.10			
9	1	9	252	252	0.30	Sub-Base	0.06	1.44	1.44	0.98	Base	0.02	0.85	0.85	0.85
	2	4	468	720	-0.34	Sub-Base	0.02				Base	0.01			
	3	38	119	839	1.43	Sub-Base	0.11				Base	0.09			
10	1	10	176	176	0.38	Sub-Base	0.06	1.78	1.78	1.14	Base	0.03	0.98	0.98	0.98
	2	5	568	744	-0.16	Sub-Base	0.04				Base	0.02			
	3	44	124	868	1.54	Sub-Base	0.11				Base	0.09			

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 using Lockwood et al. (1992) E_R values	Young modulus using Jianzhou et al. (1999) E_R values	Young modulus using George and Uddin (2000) E_R values	Resilient modulus using Lockwood et al. (1992)	Resilient modulus using Jianzhou et al. (1999)	Resilient modulus using George and Uddin (2000)
1	CH. 0 + 0.005 RHS	9	0.30	34.93	89.43	43.10	49.89	107.94	58.59
2	CH. 0 + 2.550 LHS	20	0.92	70.24	112.66	58.82	87.49	132.67	75.33
3	CH. 0 + 14.5 LHS	45	1.56	257.01	180.66	108.32	286.40	205.10	128.05
4	CH. 0 + 19.0 RHS	51	1.66	150.16	147.98	83.95	172.62	170.29	102.09
5	CH. 0 + 27.5 RHS	54	1.71	228.55	172.91	102.45	256.09	196.84	121.80
6	CH. 0 + 30.6 LHS	52	1.68	216.41	169.43	99.83	243.17	193.13	119.01
7	CH. 0 + 38.9 LHS	52	1.68	401.68	213.66	133.86	440.48	240.23	155.25
8	CH. 0 + 41.0 RHS	48	1.61	339.24	200.49	123.56	373.98	226.22	144.29
9	CH. 0 + 42.2 LHS	38	1.43	762.09	272.05	181.03	824.31	302.42	205.49
10	CH. 0 + 46.8 LHS	44	1.54	490.01	230.28	147.04	534.55	257.94	169.29

3.6. 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 strong positive correlation coefficient (R^2) of 0.9507 (Fig. 13a), and linear regression model (*Equation 10*):

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

In this relationship, x = CBR (soaked)

The relative density (RD) values obtained from "DIN 4094" equation was plotted against in-situ CBR and DCPI. This gives a regression model of *Equation 11* and *Equation 12*, with very weak positive correlations (R^2) of 0.0023 and 0.0259 respectively (Figs. 13b and c).

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

$$DCPI = 10.34\ln(x) - 10.77 \quad (12)$$

In these relationships, x = relative density

The relationship between E_R derived from "DIN 4094" and average M_R calculated from expressions proposed by Lockwood et al. (1992), Jianzhou et al. (1999), and George and Uddin (2000) is shown by the regression model in *Equation 13*, with R^2 of 0.9612 (Fig. 13d).

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

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), Jianzhou et al. (1999) and George and Uddin (2000) to give *Equation 14*, with weak positive correlation coefficient of 0.0796 (Fig. 13e); while the plots of the in-situ CBR against each of this authors give R^2 of 0.0848, 0.0739 and 0.0838 (Fig. 13f). All the models follow the same trend. The variation in the coefficients is marginal as all showed weak positive correlations. The model expressions for these relationships are presented in *Equations 15–17*.

$$MR = 1.6789x + 59.112 \quad 14$$

$$MR = 1.5435x + 94.269 \text{ (Lockwood et al., 1992)} \quad 15$$

$$MR = 2.4465x + 33.442 \text{ (Jianzhou et al., 1999)} \quad 16$$

$$MR = 1.0466x + 49.625 \text{ (George and Uddin, 2000)} \quad 17$$

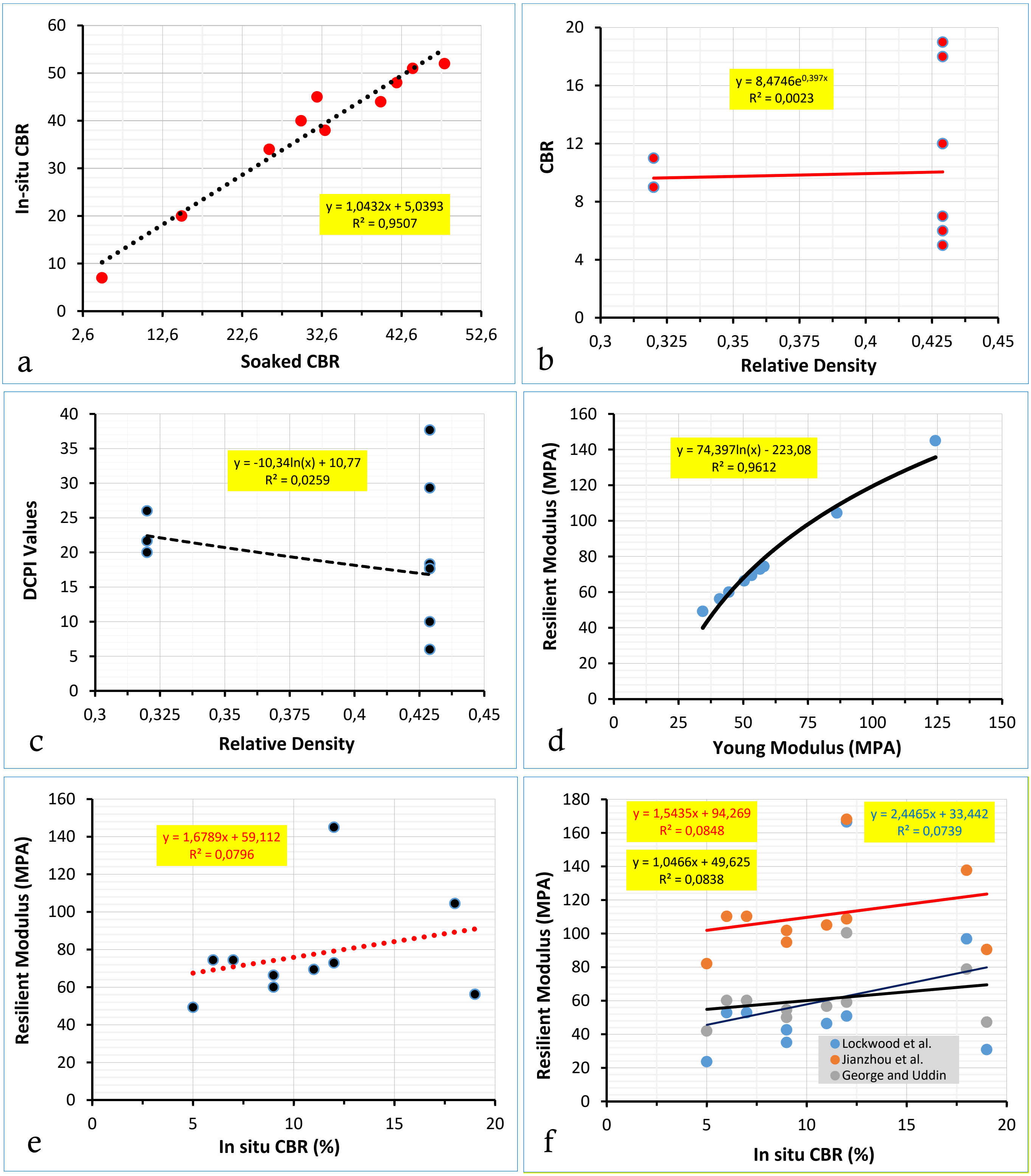

Fig. 13. Regression models for (a) CBR lab and in-situ CBR (b) RD and in-situ CBR (c) RD and DCPI (d) ER and MR (e) in-situ CBR and MR (f) in-situ and MR for Lockwood et al. (1992), Jianzhou et al. (1999) and George and Uddin (2000)

Table 11. Hydrogeological measurement of wells in close proximity to the pavement

East (m)	North (m)	Well. No	Elevation (m)	Total depth (m)	SWL (m)	Water column (m)	Hydraulic head (m)	Geology
746369	805540	W-1	291	15.0	4.5	10.5	286.5	Granite
761218	804026	W-2	285	13.5	6.1	7.4	278.9	Migmatite-Gneiss
767519	804466	W-3	298	9.8	4.8	5.0	293.2	Granite
778656	798946	W-4	310	12.5	3.5	9.0	306.5	Quartzite
787546	797872	W-5	287	10.2	5.3	4.9	281.7	Quartzite

3.7. Hydrogeological Measurement

Static water level (SWL) measured from open wells along the highway varies from 3.5 m (quartzite) to 6.1 m (migmatite gneiss) with an average of 4.8 m. The hydraulic head measured with respect to sea level ranges between 278.9 m to 306.5 m (avg. 289.4 m) (Table 11). The total depth of the well investigated in close proximity to the highway alignment ranged from 9.8–15 m (avg. 12.2 m).

Consequently, the SWL in the area is moderately low, therefore it may not seriously affect the subgrade. However excessive cut into the subsoil during reconstruction would lead to high water level situation which could compromise the integrity of the pavement structures.

4. Conclusion

The study combined geophysical, geochemical, hydro-geological, and geotechnical methods in probing the soil within A-122 Akure–Owo highway structure in Ondo State, southwestern Nigeria. The geophysical and trial pit sections revealed that the topsoil/subsoil on which the soil is constructed is composed of incompetent/fairly competent clay, sandy clay, clay sand, and laterite. The depth to basement rock ranged between 30.4–42.2 m. It is observed that the basement relief is valley-like between VES 3 and 5. Consequently, this zone will aid groundwater and impoundment of water due to its configuration, hence the high magnitude of failures observed along this zone. S-S ratio of the samples ranged from 1.36 to 1.90 (avg. of 1.58), and are categorized as lateritic soil type.

The clay mineralogy is within the illite (60%)–illite/ montmorillonite group (40%). The geotechnical correlated well with the geophysical/trial pit results, and showed the soil to be SC-SM of low–intermediate plasticity and compressibility with avg. PI of 19.7 % of moderate to high specific gravity. However, the % fines in the sample (48.2) is greater than 35 % specification, but with GI of 6, it can be adjudged as fair subgrade soil material. Subsequently, the in-situ CBR (avg. 27%) and soaked CBR (avg. 38%) satisfied the 10 % minimum specification for subgrade. The DCPT indicated the soil to be generally of medium/stiff/dense consistencies with penetrative index of 1.33–53.67 mm/blow. It also showed that 378–872 mm depths are the suitable layer to host the road structure based on the CBR and SNG with relative densities of 0.371–0.509.

The strength coefficient, SNG, SN, and SNP contributions of the soil are good for subgrade but low for subbase and base material. The regression models of all parameters gave strong positive correlations for soaked CBR and in-situ CBR, and E_R and M_R; while weak positive correlation for in-situ CBR and M_R, RD and DCPI, RD and in-situ CBR. The Static water level (SWL) measured from open wells along the highway varies from 3.5 m (quartzite) to 6.1 m (migmatite gneiss) with an average of 4.8 m which fairly deep, therefore it may not seriously affect the subgrade. Based on the GI and CBR values, and the traffic count carried out which placed the highway, the recommended thickness of the highway structure should range from 140 mm (good segment) to 445 mm (for weak segment) (avg. 193 mm) which partly corresponds to 315 mm measured along the highway alignment during reconnaissance survey.

This implies that the design thickness of the highway corresponds very well with recommended thickness emanating from this study. Thus the failures in some portions along the highway can be attributed to lack of drainage facility at the shoulders of the highway, topography/ basement relief, and usage.

Conflict of Interest

The authors declare no conflict of interest exists in this publication.

References

Adelana, S.M.A., Olasehinde, P.I., Bale, R.B., Vrbka, P., Goni, I.B., Edet, A.E., 2008. An overview of the geology and hydrogeology of Nigeria. Conference theme: Applied Groundwater Studies in Africa. In: Adelana, S.M.A., MacDonald, A.M. (ed). CRC Press Balkema, pp. 71-197.

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.

Akintayo, F.O., Osasona, T.D., 2022. Design of Rigid Pavement for Oke- Omi Road, Ibadan, Nigeria. FUOYE Journal of Engineering and Technology 7 (3), 382-386. https://doiorg/1046792/fuoyejetv7i3837.

Amosun, J.O., Olayanju, G.M., Sanuade, O.A., Fagbemigun, T., 2018. Preliminary geophysical investigation for road construction using integrated methods. RMZ–M&G 65, 199-206. https://doiorg/102478/rmzmag-2018-0017.

Anon, 1952. Soil Mechanics for Road Engineers. Transport and Road Research Laboratory, Her Majesty's Stationery Office, London, UK.

ASTM, 2006. Annual Book of ASTM Standards–Sections 4.02, 4.08, 4.09 and 4.13. ASTM International, West Conshohocken, PA.

Bell, F.G., 2004. Engineering Geology and Construction. Spon Press, London.

Bell, F.G., 2007. Engineering Geology. Second Edition, Elsevier Limited.

Blyth, F.G.H., de Freitas, M.H., 1984. A Geology for Engineers Seventh Edition, Elsevier Butterworth Heinemann, Oxford 365 pp. https://doiorg/101038/153476b0.

Brink, A.B.A., Parridge, J.C., Williams, A.A.B., 1992. Soil Survey for Engineering, Claredon, Oxford, UK.

Brown, S.F., 1996. Soil mechanics in pavement engineering. Geotechnique 46, 383-426.

Carter, M., Bentley, S.P., 1991. Correlations of soil properties, Pentech Press Publishers, London, 129 pp.

Cekerevac, C., Cekerevac, C., Baltzer,S., Charlier, R., Chazallon, C., Erlingsson, S., Gajewska, B., Hornych, P., Kraszewski, C., Pavšič, P., 2009. Water influence on mechanical behaviour of pavements: Experimental investigation. In: Dawson (Ed), Water in road structures (pp. 217-242) Springer.

Chapman, R.E., 1981. Geology and Water, Martinus Nijhoff/Dr W Junk Publishers, London. https://doiorg/101007/978-94-009-8244-4.

Charman, J.H., 1988. Laterite in Road Pavements Special Publication 47, Construction Industry Research and Information Association (CIRIA), London.

Chen, D.H., Lin, D.F., Liau, P-H., Bilyeu, J., 2005. A correlation between Dynamic Cone Penetrometer values and pavement layer moduli. Geotechnical Testing Journal 38 (1), 42-49.

Chen, F.H., 2000. Soil Engineering: Testing, Design, and Remediation, Edited by Morris, PE, CRC Press LLC, Corporate Blvd, NW, Boca Raton, Florida 33431, 288 pp.

DIN, 1980. DIN 4094 Part 2. Dynamic and Static Penetrometer.

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. Available: www.transport-linksorg/ukdcp/docs/Manual/manualhtml.

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. Federal Highway Department, Lagos, 2: 145-284.

FHWA, 2006. NHI-05-037, 2006, Geotechnical aspects of pavement. US Department of Transportation Federal Highway Administration pp 4-17.

George, K.P., Uddin, W., 2000. Subgrade characterization for highway pavement design, final report Jackson, MS: Mississippi Department of Transportation.

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 (3), 44-50.

Iloeje, 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 (LRS 10), in southeastern Nigeria. International Journal of Geo-Engineering 6 (7), 28 pp. http://dxdoiorg/101186/s40703-015-0007-2.

IRC, 2002. Guidelines for the design of flexible pavements. Indian roads congress Number 37 New Delhi, India.

Jianzhou, C., Hossain, M., Latorella, T.M., Record, T.R., 1999. Use of Falling Weight Deflectometer and Dynamic Cone Penetrometer in Pavement Evaluation. Paper Presented in the Transportation Research Board, Washington DC, USA.

Holtz, W.G., Kovacs, W.D., 1981. An Introduction to Geotechnical Engineering, Prentice-Hall Publishers, 733 pp.

Kadyali, L.R., Lal, N.B., 2008. Principles and Practices of Highway Engineering (Including Expressways and Airport Engineering), Fifth Edition, Romesh Chander Khanna, New Delhi, India, 858 pp.

Kearey, P., Brooks, M., Hill, I., 2002. An Introduction to Geophysical Exploration. Blackwell Science Limited, 262 pp.

Kodicherla, S.P.K., Nandyala, D.K., 2016. Use of CPT and DCP based correlations in characterization of subgrade of a highway in Southern Ethiopia. Region International Journal of Geo-Engineering 7 (11), 1-15. http://dxdoiorg/101186/s40703-016-0025-8.

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 Pretoria, South Africa: CSIR Transportek.

Loke, M.H., 2000. Electrical Imaging Surveys for Environmental and Engineering Studies: A Practical Guide to 2D and 3D Surveys, 59 pp. www.terrajp.co.Jp/lokenote.pdf,2004.

Loke, M.H., 2004. Tutorial: 2-D and 3-D Electrical Imaging Surveys, Revised Edition, 136 pp.

Nigeria 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, 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.

Powell, W.D., Potter, J.F., Mayhew, H.C., Nunn, M.E., 1984. The structural design of bituminous roads. ME TRRL Report LR pp 62, 1132.

Rahaman, M.A., 1988. Recent Advances in the study of the Basement Complex of Nigeria. In: Precambrian Geology of Nigeria. Geological Survey of Nigeria Publication, 11-43.

Scala, A.J., 1956. Simple Methods of Flexible Pavement Design Using Cone Penetrometer. New Zealand Engineer 11 (2), 34-44.

Smith, A.J., Montgomery, R.F., 1962. Soils and land use in Central Western Nigeria. The Government of Western Nigeria, Ibadan Publication.

Sudha, K., Israil, M., Mittal, S., Rai, J., 2009. Soil characterization using electrical resistivity tomography and geotechnical investigations, Journal of Applied Geophysics 67, 74-79.

Telford, W.M., Geldart, L.P., Sheriff, R.E., 1991. Applied Geophysics, Cambridge University Press, 792 pp.

Transport and Road Research Laboratory, 1990. A user's manual for a program to analyze 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), 19 pp. https://doiorg/101186/s40677-020-00174-8.

Uz, V.E., Saltan, M., Gökalp, I., 2015. Comparison of DCP, CBR, and RLT Test Results for Granular Pavement Materials and Subgrade with Structural Perspective. International Symposium Non-Destructive Testing in Civil Engineering (NDT-CE) September 15 - 17, Berlin, Germany, 10 pp.

Vandre, B., Budge, A., Nussbaum, S., 1998. DCP- A Useful Tool for Characterizing Engineering Properties of Soils at Shallow Depths. Proceedings of 34th Symposium on Engineering Geology and Geotechnical Engineering, Utah State University, Logan, UT.

Vazirani, V.N., Chandola, S.P., 2009. Concise Handbook of Civil Engineering. Revised Edition, Rajendra Ravindra Printers Pvt Ltd, New Delhi, India, 1318 pp.

Wright, P.H., 1986. Highway Engineering, Sixth Edition, John Willey and Sons, New York, USA.

Yoder, E.J., Witczak, M.W., 1975. Principles of Pavement Design. Second Edition, John Wiley and Sons, Inc New York, USA.