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INVESTIGATION OF SOME UNPAVED HIGHWAYS IN AWKA METROPOLIS, SOUTHEASTERN NIGERIA USING 2D ELECTRICAL RESISTIVITY TOMOGRAPHY AND GEOTECHNICAL METHODS

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ABSTRACT: *Shallow subsurface two-dimensional Electrical Resistivity Tomography (2DERT) and geotechnical surveys were carried out at some unpaved highways in Awka, Anambra State, Nigeria. The surveys were aimed at investigating and analyzing shallow subsurface formations along the roads on which future construction of pavement is expected. The 2DERT was carried out from five survey lines-oriented W-E, N-S and SE-NW directions respectively. Also, soil samples were collected from depths of sub-grade evaluation of maximum dry density (MDD), California bearing ratio (CBR), plastic index (PI), particle size distribution (PSD) and specific gravity of (G_s) respectively. Results of the 2DERT inverse models were characterized by apparent resistivity range of 0.200 to 5601 Ωm. Results from the geotechnical analyses showed ranges of MDD (1.94-2.09 mg/m^3), CBR (48.30-51.7%), PI (14-19%), PSD (34-38%) and G_s (2.57-2.61 mg/m^3) respectively. Interpretation of the results from the two methods showed that at shallow depths along the unpaved roads; sand, clayey-sand and clay are predominant. Non-homogeneity of the top soils suggests impending danger of expansive soil, non-homogenous subsurface formation, dipping and fractured layers at sub-grade depths in the area. Hence, the findings suggest the need for embankment of the unpaved roads at some portions before construction of durable highway pavements could be attained in the area.*

KEYWORDS: Resistivity, Tomography, Geotechnical, Pavement, Embankment, 2DERT

INTRODUCTION

The basis of development in any given society is anchored on well paved road network and its facilities for transportation of goods and services. In societies where accessible roads were either not paved at all or poorly paved during construction, socio-economic activities would either dwindle or totally collapse. Road failure is inability of a paved road to carry out its expected functional services by not providing a smooth-running surface for transiting vehicles. It is also a discontinuity in road pavement due to occurrences of cracks, potholes, bulges, rutting and depression (Aigbedion (2007); Onuoha and Onwuka (2014)). Sequential arrangement of a typical vertical section of a flexible road pavement underlain by sub-grade (foundation level) includes the sub-base which is lowermost, overlain by the base course, surface course and finally the surface dressing which is the finishing of the pavement (Benson and Lay, 2017). Defects on roads could be attributed to many factors among which are either geomorphologic or anthropogenic (Adegoke-Anthony and Agada, 1980; Ajayi, 1987). In one hand, the geomorphologic factors are due to rugged topography, poor surface drainage systems, instability of intact rocks and soils at shallow subsurface. On the other hand, the anthropogenic factor is related to the failures due to human activities such as dumping of refuse at drainage channels, poor or bad design, faulty construction, overloading of vehicles beyond the bearing capacity of sub-grade and the use of quarks instead of qualified engineering expertise for highway constructions. Highway pavements are of two types namely the flexible and the rigid types. The flexible pavement type consists of asphaltic or bituminous material and aggregates placed on a bed of compacted granular material of appropriate quality in layers over the sub-grade. The rigid pavement is constructed from cement concrete or reinforced concrete slabs (Benson and Lay, 2017). The flexible pavement involves the layers of sub-grade, sub-base, base-course and surface-course for vehicle highway construction whereas the rigid pavement requires only the rigid concrete for air fields.

The sub-grade of highways is the natural formation or foundation level of a road which has great importance in road construction. It is typically characterized by their resistance to deformation when the road pavement is under load based on its resistance to stress, breakages and rupture. In general, the more resistant a sub-grade is the more loads it can support before reaching its deformation value. The deformation value could be measured using some geotechnical parameters such as California Bearing Ratio (CBR) or Elastic (resilient) modulus. Other criteria for measuring deformation values include Optimum Moisture Content (OMC), maximum dry density (MDD), plastic index (PI), plastic limit (PL), liquid limit (LL), particle size distribution (PSD) and specific gravity (G_s). If the strength of the sub-grade material is weak or not properly evaluated prior to road construction, it will pave way to road dilapidation. Therefore, it is imperative to choose appropriate method(s) in characterizing the subsurface material as part of pre-design phases of road pavement construction.

Reconnaissance observation of the roads at Awka south of Awka metropolis, capital of Anambra state, Nigeria showed that some of the paved roads in the recent past have failed. The road pavements failures examined were mostly of are two types namely flexible pavement used for vehicle highways or rigid pavement used for airfields. Plate I show the pictorial views of some portions of road failures in Awka metropolitan. It was noted that the failures of the roads in the area were not caused by the type of pavement (the flexible) but in the standard of construction. Pavement constructions were expected to involve the preparation of the sub-grade, embankment and the layering from the sub-base to surface dressing of it. Particularly, it was observed that some failures were due to failures due to rutting and raveling (Plate Ia) owing to poor engineering construction. Other road failures in the city are suspected to have come from the subsurface. Careful observations showed that pot holes resulting to various kinds of cracks are rampant. Also, apart from alligator crack (Plate Ib) which was found most common; block, slippage, edge, transverse and longitudinal cracks also occur on the paved roads in Awka city. Upheaval and Subsidence of the subsurface suspected to be from the depths of the sub-grade were also observed on the existing roads in the study area (Plates Ic and Id).

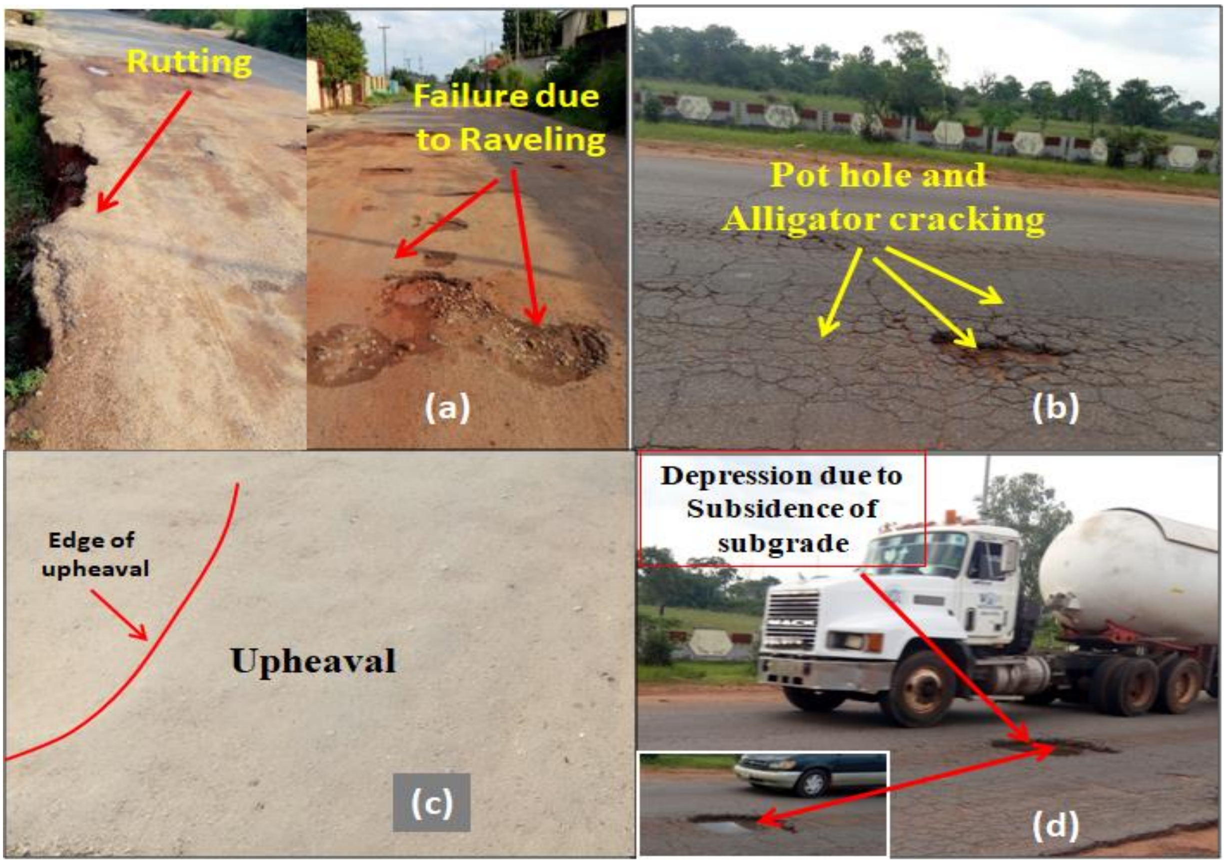

Plate I: The pictorial views of some portions of road failures in Awka metropolitan.

Responses to enquiries made from indigenous highway/civil engineers during the pilot onservation of failed roads in Awka revealed that inadequate pre-construction tests have contributed to road pavement failures in the area. Particularly, inadequate geotechnical tests and total negligence of geophysical survey prior to the constructions are the principal causes. The road pavement failures and unpaved zones along the roads in the area have led to accidents, kidnapping and armed robbery during transportation. Other aftermaths include gross delay in transportation of goods and services, increase in wears and tears on vehicle hence leading to socio-economic recession in the area. Therefore, in bid to reduce or completely avert this menace and its accompanying tragic aftermaths, this study is embarked upon for adequate understanding of the subsurface materials upon which these roads were paved. Thus, two complementary viable methods namely geotechnical and geophysical methods were applied for assessment of the subsurface particularly the sub-grade of the roads intended for pavement in the area. Geotechnical method is useful in the assessment of vertical column of shallow subsurface. However, it is very limited in characterizing the spatial structure of the subsurface due to its one-dimensional design. It is also expensive if several test pits must be sinking for area coverage. The geophysical method is therefore applied to augment geotechnical test results by providing two-dimensional cross-sectional images of the subsurface materials. Particularly, a fast, accurate, non-invasive and cost-effective geophysical method namely, Electrical Resistivity Tomogram (2DERT) is a modern geophysical technique for imaging shallow subsurface is usually applied. The technique is capable of analyzing the spatial models of highway subsurface in a given area. Therefore, the aim of this study is to use 2DERT and geotechnical surveys for assessment of sub-grade prior to construction of flexible pavement is Awka south of southeastern Nigeria. The results and recommendation from the surveys would serve as reference materials for civil engineering road works in future for designs and construction of durable road pavements in the study area.

The Study Area

The study area is part of Awka-South Local Government Area in Awka metropolis, Anmbra state, Nigeria. is bounded by latitudes $6°14'2.95''$ N and $6°13'18.48''$ N and, longitudes $7°4'54.29''$ E and $7°6'10.24''$ E at about 157.0 m above the mean sea level. It covers an area extent of about 1.45 km^2 and a perimeter of about 4.99 km. Awka area lies within the tropical rain forest belt of Nigeria. Its vegetation is mainly shrubs, grasses and perennial trees as occasioned by settlement and other human activities (Iloeje, 1981). Two main seasons characterize the study area namely the rainy season which is associated with moist maritime southwesterly trend wind from Atlantic Ocean and dry season which is associated with Atlantic continental northwesterly wind from the Sahara Desert respectively. According to Riheb *et al.* (2014), the climatic conditions of the area has major influence on the susceptibility of shrinking and swelling phenomenon of clay materials in the subsurface. Geologically, the study area is in Niger Delta Basin of Nigeria located in the Gulf of Guinea of equatorial West Africa lying between latitudes 3^oN and 6^oN and longitudes 5^oE and 8^oE (Reijers *et al.,* 1996). The Delta is characterized by a prograding depositional complex found within the Cenozoic Formation of the Southern Nigeria. It is bounded in the west by the Benin flank, in the east by Calabar flank, in the North by Anambra basin and by the Atlantic Ocean in the south (Murat, 1972). Major outcropping units of the Niger Delta include the Imo formation and Ameki group of outcrops. Particularly, the study area is characterized by *Imo* shale which consists of thick clayey shale, fine textured, colour range from dark grey to bluish grey with occasional admixture of clay, iron stones and thin sand stone bands (Reyment, 1965). The Ameki group comprises of Ameki, Nanka, Nsugbe, and Ogwashi-Asaba formations. The Niger Delta is composed of three classes of stratigraphic units which range from Akata formation; which is the oldest, through Agbada Formation to Benin Formation which is the youngest (Short and Stauble, 1967). The Akata Formation $\leq$7,000 m (Doust and Omatsola, 1990) is characterized by marine shales, sandy beds and silty beds considered as turbidites and continental slope channel fills. The Agbada Formation, generally $>$ 3,700 m thick is the major petroleum-bearing unit in the Niger Delta. It consists of sand and shale (Ajaegwu *et al.,* 2012). The Benin Formation which is the youngest consists of continental sands and gravels having thickness in the range of about 280 m to 2,100 m (Whiteman, 1982). Figure 1 shows the maps of the study area and, the location and pictorial views of the unpaved roads for survey.

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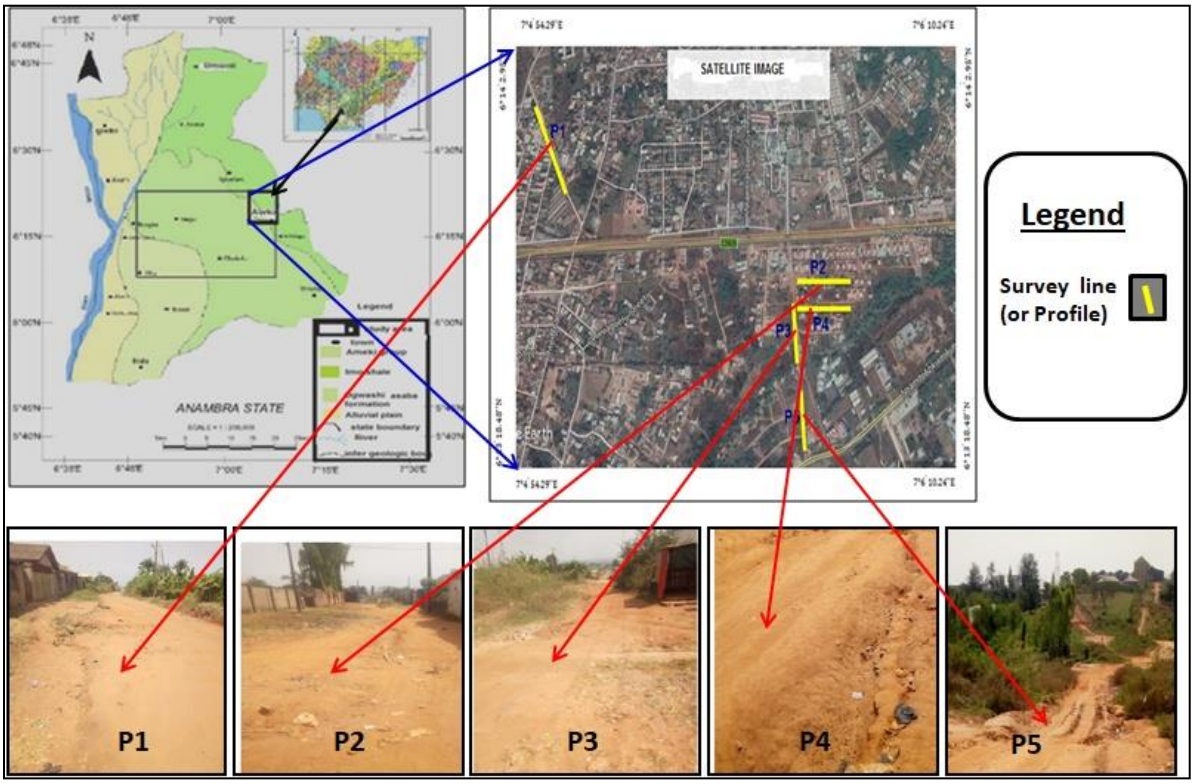

Figure 1: Map of the study area, five proposed roads and the surrounding (modified from Onyekwelu, 2014; Google earth map, 2018).

METHODOLOGY

Methodology I: GEOPHYSICAL

A state-of-the art geophysical instrument namely; *ABEM LUND* multi-electrode Imaging System was used for the 2DERT survey (ABEM Manual, 2010). The system, with a network of 42 electrodes was located at the centre of the road layout mapped for each profile. The system comprises Terrameter SAS1000/4000 aided by an electrode selector model ES10-64. The Terrameter was meant for setting the choice of array for measurement while the electrode selector automatically activates four electrodes (two as current and two as potential) for each measurement. A 12 V D.C. battery was used to power the entire system during measurement however, a backup power source of same capacity was provided to ensure continuous measurement throughout the field work.

Reconnaissance survey was first embarked upon during which the survey lines were mapped and the points along the lines whereupon soil samples would be collected for geotechnical tests were selected and marked. Five profile lines namely; P1, P2, P3, P4 and P5 (Figure 1) along the roads proposed for highway construction were marked for the survey. Profile P1 is oriented in SE-NW, P2 and P4 were oriented in E-W direction while P3 and P4 were in N-S direction in the geographical grid. Wenner-Short (*Wenner-S*) array technique was used for the 2DERT survey with minimum equal electrode spacing$'a'$ of 2 m. The technique was chosen because of its vertical and horizontal sensitivity and its high signal to noise ratio which makes it more viable in electrically noisy environment. Using the Wenner array configuration, the electrode spacing was automatically varied by the ES10-64 for the measurement of apparent resistivity $'\rho_a'$ (Equation1). For each registered datum, current $'I'$ was injected into the subsurface via two outer electrodes while potential difference $'\Delta U'$ was consequentially generated between the other two inner potential electrodes.

$$\rho_a = 2\pi a \frac{\Delta U}{I} \tag{1}$$

Prior to data acquisition along each survey line, various settings for each measurement were ensured done based on the measurement protocol (the *Wenner-S*) selected. The Terrameter was first of all set to LUND resistivity mode (*LR-mode*) before the commencement of the measurement. For each profile line, the 42 electrodes were aligned and the Lund cables were connected to all the spaced electrodes using the cables jumpers respectively. Afterwards, electrode test for electrical continuity between the system and the subsurface via the electrodes was carried out. Electrodes identified by their numbers after the test as having no continuity through them were identified by the field operator. Hence, there was wetting of the contact points of the electrodes with the ground surface with the aid of salt water (NaCl solution). Finally, after all the required settings and tests, data collection for the imaging was carried out for at all the survey lines. While collecting the data, the positions of reference points vis-à-vis the latitude, longitude and height above mean sea level along each survey line were noted and recorded for reference with aid of Geo-Positioning System (GPS).

The 2DERT data registered in the Terrameter from the five survey lines were downloaded into laptop in which SAS1000/4000 utility software meant for data conversion has been installed. Afterwards, the raw data in*.s4k* format was converted to data format *.dat* with the aid of the utility software installed to make it readable by the interpretation software. Each of the files of the converted data were named to represent the details of data collected from a profile line which has been imported into an interpretation software namely *Res2DINV* version 3.8 (Loke, 2000). Consequently, the software was used to process and model the data iteratively. Although care was taken during the survey, inherent bad data were registered and these were identified by their unusual relatively high or low values when compared with other data points at its neighborhood. The bad data points being as a result of unforeseen poor ground contact by electrodes such that sufficient current was not injected into the ground. Therefore, prior to the modeling of the data, bad datum points were exterminated in order to obtain a more reliable model, Inversion of the input data set for each profile was carried out iteratively after satisfactorily editing the data with least square inversion routine. Finally, the software automatically models a two-dimensional resistivity tomography (or imaging) after a minimum of 3 iterations and minimum absolute error in range of 3.3 % and 12.2 %. The final outputs of the models were displayed as two-dimensional tomograms which showed first the model of the measured data, then that of the calculated data and finally the tomography model of the inverted apparent resistivity data required for interpretation (Figures 2-6).

Methodology II: Geotechnical

Various instruments and the methods are involved in the geotechnical aspect of the investigation depending on the test required. Five geotechnical tests were carried out namely; Soil Compaction Test, California Bearing Ratio (CBR), Atterberg limits for determination of liquid limit (LL), Particle size distribution for wet and dry sieving tests and Particle density test for estimation of the specific gravity (G_s). While the compaction test, CBR, Atterberg limits and Particle size distribution were carried out **a**ccording to British standard test procedure (BS1377, 1990), the particle density was according to American Society of the International Association for Testing and Materials (ASTM 128). The instruments used for measurement in geotechnical tests include Proctor/Compaction Mould, California bearing tester, Atterberg Limit Machine, Auger and Sieves shaker. Table 1 shows the instruments' names, their uses and the standard used for geotechnical tests.

Table 1: Use of Instruments for Geotechnical Tests and their Standard

Instrument's Name	**Use of the Instrument**	**Remarks/ Standard**
Proctor/ Compaction Mould	For determining the maximum dry density achievable for the construction materials in the field.	British Standard (BS1377, 1990) mould
California bearing tester	For evaluation of the bearing capacity or elastic modulus of the sub-grade soil meant for design of flexible pavement. It also used in	British Standard (BS1377, 1990) mould
Atterberg Limit Machine	Used for determination of Plastic Index by finding the difference between the liquid limit and the plastic limit	British Standard (BS1377, 1990) mould
Auger	For distinguishing between silty and clayey soils. It is also used as a guild in determining how a soil is likely to settle or consolidate under load.	1.5 m length and above.
Sieves shaker	For determining the Particle size distribution of fine and coarse aggregate present in the soil.	British Standard (BS1377, 1990) mould

In the geotechnical method of investigation, samples were collected with the aid of Auger from the depth range of about 0.5-1.0 m at selected spots along each of survey lines. The depth range was guided by the depth limits of existing natural sub-grade in the study area. The samples were air-dried for 24 hours at room temperature. Afterwards, they were divided into five parts according to number of laboratory test intended. Four, out of the five geotechnical tests were carried out using the British standard test procedure (BS1377, 1990). These are the soil compaction test, the California bearing ratio (CBR), the Atterberg limits and the particle size distribution. The other, which is the particle density test, was carried out according to American Society of International Association for Testing and Materials (ASTM 128).

The soil compaction test was carried out in a rammer having 50 mm diameter face and 4.5 kg weight which was set to fall from a constant height of 300 mm based on a British Standard (BS) mould. While the guide tube was held vertically, each soil sample was compacted by 27 blows from the rammer to mould a layer of soil respectfully. A second, third, fourth and fifth approximately equal layer of the soil were placed in the mould and the procedure was repeated respectfully so that at least 5 compactions were made for each soil sample. Where, and were the weight of the mould M_1, weight of the mould and soil M_2 and the volume of the mould used X (1000 cm^3 for BS Mould) were measured. These were done such that water increment range encompasses the optimum moisture content W (%) Hence, using the sample moulds, the bulk density (ρ_b) and, the dry densities $and\ \rho_d$ respectively both in (mg/m^3) were estimated using equations 2 and 3 respectively.

$$\rho_b = \frac{M_2 - M_1}{X} \tag{2}$$

$$\rho_d = \frac{\rho_b}{1+W} \tag{3}$$

The *CBR* test was also carried out using the standard CBR mould, fittings and tools. Particles with >20 mm diameter were sieved out and 4.5 kg rammer was used for soil sample compaction. The samples were loaded in three layers and subjected to 62 blows from the rammer dropped from a height of about 300 mm. With the aid of a compression machine having its zero-error corrected, readings were taken at 0.25 mm displacement interval such that after 7.5 mm penetration, the machine was stopped. Afterwards, the moisture contents were determined for top and bottom of the specimen. The CBR was estimated by calculating the percentage ratio of the pressure for each sample P (N/mm^2) to the pressure of equal penetration on standard soil P_s (N/mm^2) for each sample based on equation 4 respectively.

$$CBR = \frac{P}{P_s} \times 100 \tag{4}$$

For the Atterberg limit tests, about 250 g of each soil sample were thoroughly mixed and passed through a 0.425 mm mesh placed in a porcelain dish. Each soil was sufficiently mixed with 15ml to 20 ml of distilled water, stirred, kneaded and chopped with aid of spatula. Using some quantity of the mix, the liquid limit (LL) of the sample was taken as the moisture content corresponding to 25 blows. Also, the plastic limit (PL) was determined using about 20 g of the sample which was passed through 0.425 mm sieve. The sample was thoroughly mixed with distilled water, kneaded to form a plastic ball, then rolled on a glass plate using steady pressure to about 3 mm, the pressure was maintained until the thread crumbled. This crumbling point is the plastic limit. Thus, the Plastic Index (PI) of the sample is given by equation 5 in the foregoing.

$$PI\ = LL - PL \tag{5}$$

Wet and dry sieving tests of the soil samples were carried out for particle size distribution. A 2 mm British Standard (BS) sieve was nested in a 63-micron sieve without its lid. With a jet of clean water, the samples in each sieve were drained of water and dried in an oven tray between 105°C and 110°C for about 12 hours. Finally, on a mechanical sieve shaker, the dry soil samples were passed through a nest of the complete range of sieves having minimum of 63µmm. Percentage weight of the sample retained and that passing in the sieves were determined using the equations 6a and 6b in the fore going respectively.

$$\%weight\ retained = \frac{weight\ retained}{initial\ weight} \times 100 \tag{6a}$$

$$\%passing\ = 100 - \ \%weight\ retained \tag{6b}$$

Finally, on the geotechnical survey, the particle density test, otherwise known as the specific gravity (G_s) was determined to estimate the strength of the soil against impact load (or toughness), impact of seasonal weathering (or soundness) and its extent of water absorption. First, three density bottles were washed, dried, cooled and weighed to the nearest 0.001g and recorded as (W_1). About 50 to 150 g of each sample was passed through a 2 mm sieve, put into each of the bottles, weighed and recorded (W_2). Distilled water was added to each bottle such that the soil was covered and the bottle half-filled. Hence, the soil, bottle and water were weighed and recorded as (W_3). The bottles were cleaned, filled completely with distilled water and placed in constant temperature bath until attainment of bath temperature. Finally, the bottle and distilled water were weighed and recorded as (W_4). Based on the measurements, the specific gravity, G_s was calculated using equation 7.

$$G_s = \frac{(W_2 - W_1)}{(W_4 - W_1) - (W_3 - W_2)} \tag{7}$$

RESULTS

Results I: Geophysical Investigation (2DERT)

The inverted apparent resistivity pseudosections obtained from the study area for the five survey lines were considered as 2DERT results for interpretation respectively. Profile 1 (Figure 2) oriented SE-NW in the study area shows apparent resistivity range of about 40-1,300 Ωm. It is characterized by relatively heterogeneous resistivity values at its topsoil ($0.5 - 1.5\, m$) in the range of 200 to 800 Ωm. Below the top soil ($5.4 - 15.0\, m$), the tomogram is layered with decreasing resistivity value with depth.

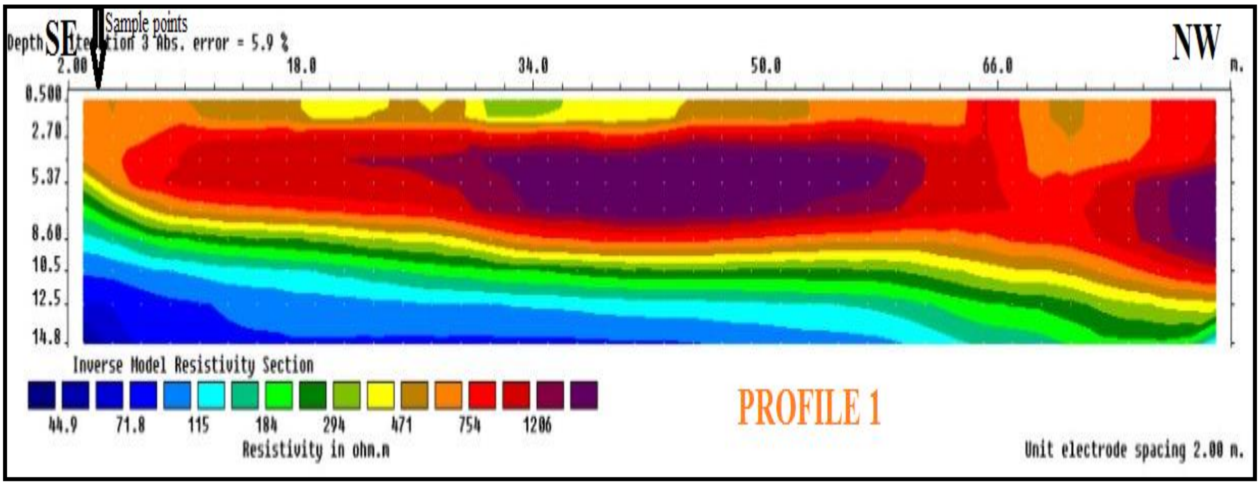

Fig. 2: Result of 2D Inversion of the Wenner Array Along Profile 1

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Profile 2 (Figure 3) oriented W-E in the study area shows apparent resistivity range of about 70-500 Ωm. It is also characterized by relatively heterogeneous resistivity values at its topsoil $(0.5 - 2.5\ m)$ in the range of 80 to 400 Ωm. Below the topsoil, there is a gradual vertical alignment of resistivity increase between lateral distances of about 40 to 50 m. resistivity. This shows a sign of developing fracture. Secondly, while at depths (>5.0 m) there is layered horizons between lateral below lateral distance of about 4.0 m, beyond 5.0 m, is occurrence of homogeneous and relatively high resistive zone (≥ 490 Ωm).

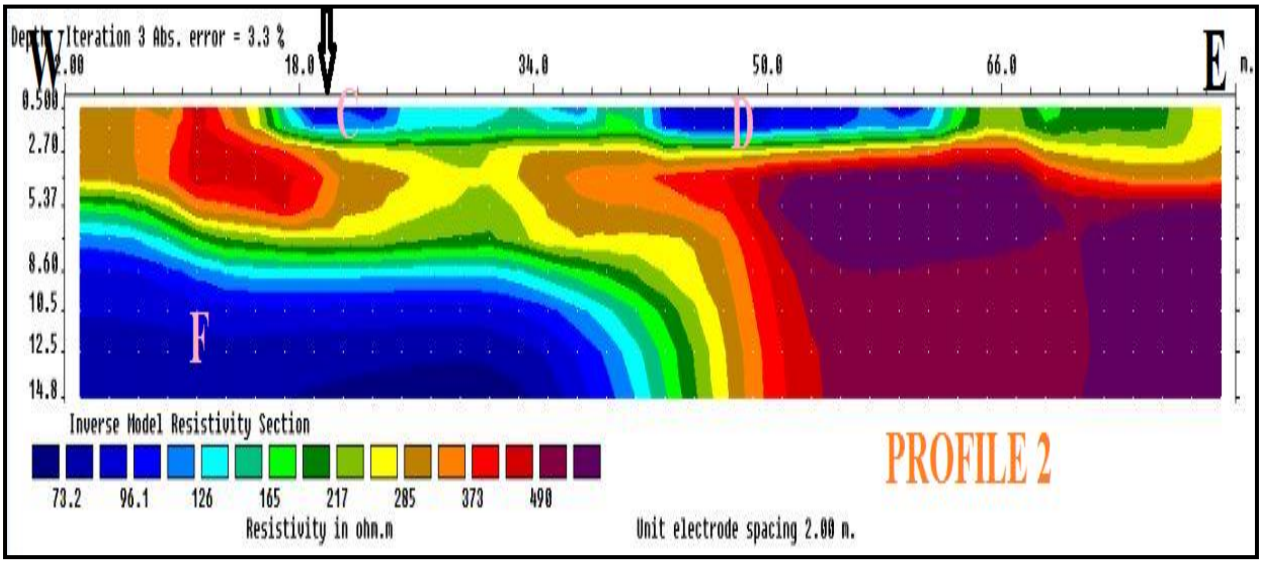

Fig.3: Result of 2D inversion of the Wenner Array Along Profile 2

Profile 3 (Figure 4) oriented N-S in the study area shows apparent resistivity range of about 1-5,700 Ωm. The topsoil $(0.5 - 2.7\ m)$, Generally but significantly shows vertical alignment of various resistivity values in the range of 1- 700 Ωm. At depths from about 5 to 15 m and lateral distance range of 0 to about 55 m, there is occurrence of undulating layers characterized by resistivity range of 70 to 700 Ωm.

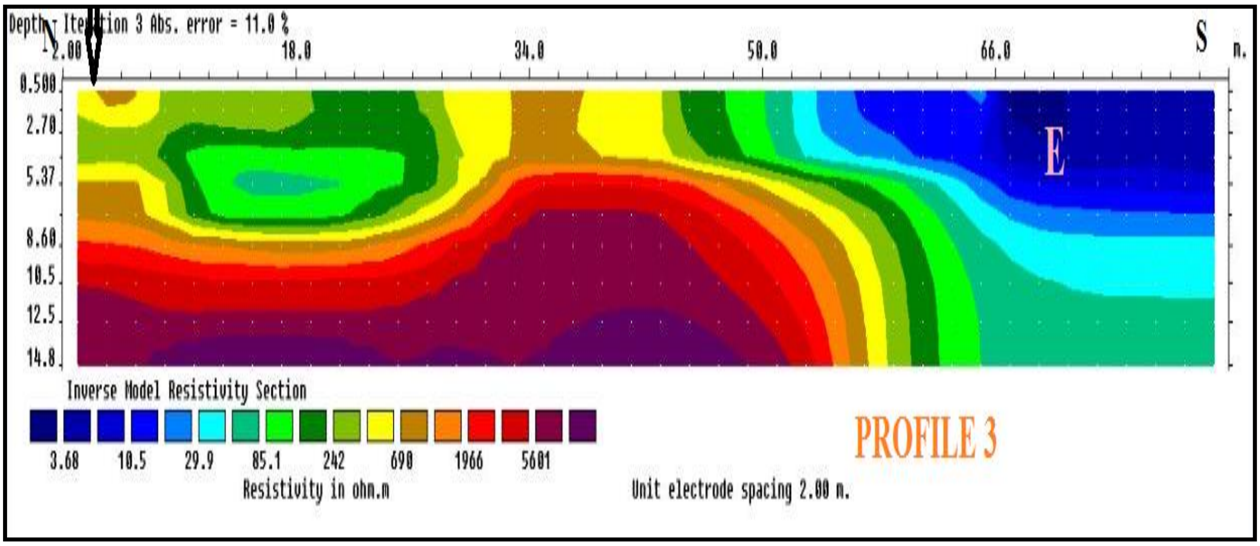

Fig.4: Result of 2D Inversion of the Wenner Array Along Profile 3

Profile 4 (Figure 5) oriented W-E in the study area apparent resistivity range of about 70-1,200 Ωm. Its topsoil is characterized by vertically aligned resistivity contrasts within the range of about 70 to 1,200 Ωm with exception of zones A and B. At depths, there is significantly faint variation in low resistivity of range 70- 550 Ωm.

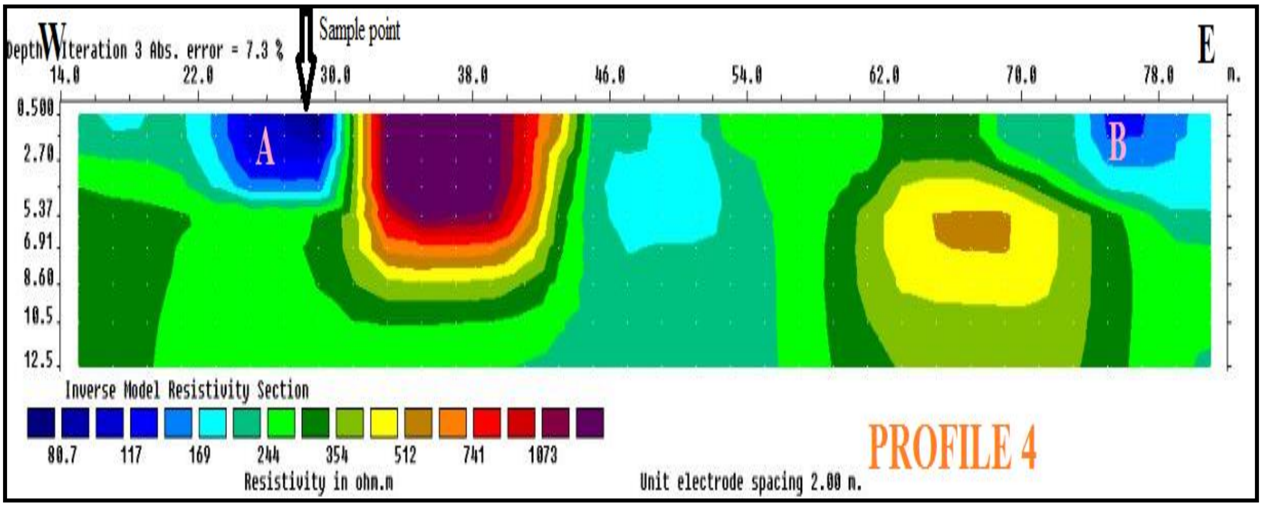

Fig.5: Result of 2D Inversion of the Wenner Array Along Profile 4

Profile 5 (Figure 6) also oriented W-E in the study area shows very apparent resistivity range of about 1-200 Ωm. It is characterized by relatively high resistivity values (30-2900 Ωm) at its topsoil compared with depths at its topsoil. At lateral distance between 38 and 40 m within the topsoil, there is evidence of a developing fracture. At depths below the top soil from about 5.5 m to 15.0 m), the tomogram shows faint vertical variations of low resistivity values with depth.

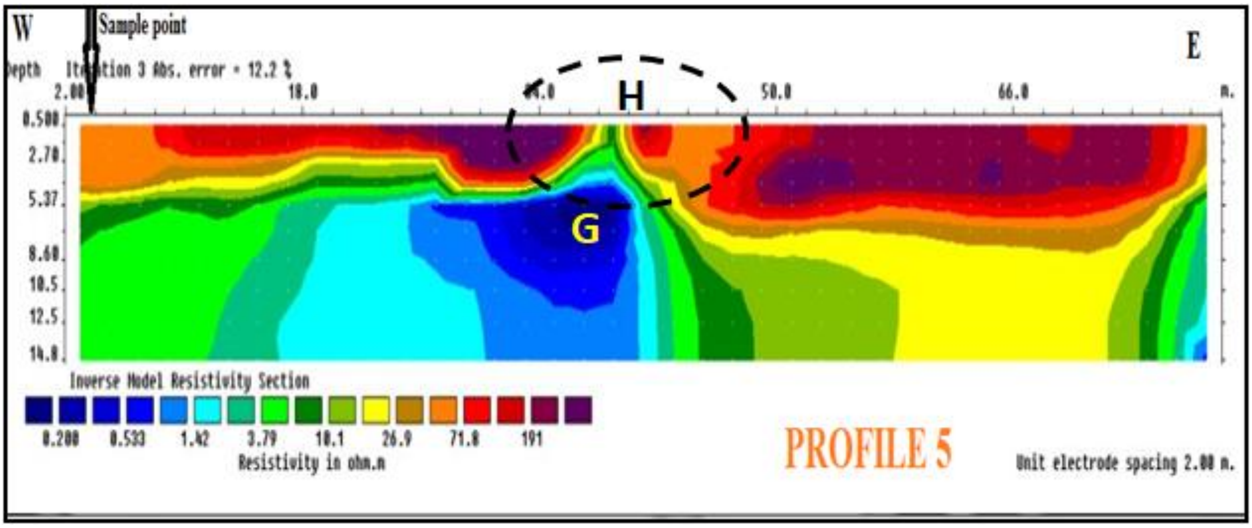

Fig.6: Result of 2D Inversion of the Wenner Array Along Profile 5

Results II: Geotechnical Evaluation

The geotechnical results obtained were evaluated in comparison with the standard of Nigeria Specification for Road and Bridge Material namely; Federal Ministry of Works and Housing of 1997 (FMWH, 1997). All the results of the five geotechnical tests were obtained using equations 2-7 respectively. From the results, samples with high value of MDD and low value of OMC were considered as the best suitable for sub-grade materials. Particularly, for a material to be suitable for construction, it should have MDD > 0.047mg/m^3 and OMC < 18% (FMWH, 1997). The result of the compaction test (Table 2) indicates that the optimum moisture content (OMC) of samples range from 6.61% to 10.69% and that the maximum dry density (MDD) ranged from 1.94 mg/m^3 to 2.09 mg/m^3. Therefore, the compaction results show that the sub-grade is suitable for road construction. The Atterberg limit results (Table 2) showed that liquid limits (LL) ranged from 33.00 % to 37.00 %, Plastic limit (PL) ranged from 16.00 % to 21.00 % and Plasticity index (PI) ranged from 14.00 % to 19.00 % on average of 19.00 %. However, the recommended standard (FMWH, 1997) of LL and PI standard for sub-grade are 40% maximum and 20% maximum respectively. Therefore, the liquid limit (LL) and Plastic Index (PI) results of the sub-grade in the study area show suitability for road construction based on their values.

From the assessment and analysis of the specific gravity of the sub-grades in the study area, it is known that lower specific gravity indicates a coarse soil while higher specific gravity indicates a fine-grained soil. According to Wright (1986), the standard range of values of specific gravity of soil lies between 2.60 mg/m^3 and 2.80mg/m^3. The results of the specific gravity G_s of the samples from this survey range from 2.57 mg/m^3 to 2.61 mg/m^3. These values are higher than the minimum specific gravity of 2.2 mg/m^3 recommended by FMWH

(1997) for roads and bridges construction. This is indicative of fine grain soil which is significantly higher than that recommended hence, suitable for the road constructions. Table 2 shows the results of the all the geotechnical tests (OMC, MDD, CBR, PI and G_s) as compared with the Nigeria Specification for Road and Bridge Material (FMWH, 1997).

Table 2: Results of Geotechnical Tests and their Compared with the (FMWH, 1997) Standard.

	Survey Line	**P1**	**P2**	**P3**	**P4**	**P5**	**Mean Value**	**FMWH Standard for Road Construction**	**Competency for Road Construction**
Sample Points	**Latitude**	6°13′ 48″N	6°13′ 48″N	6°13′ 48″N	6°13′ 48″N	6°13′ 48″N			
	Latitude	7°05′ 07″E	7°05′ 07″E	7°05′ 07″E	7°05′ 07″E	7°05′ 07″E			
Geotechnical Test	Compaction Test								
	OMC (%)	8.70	8.71	6.61	8.71	10.69	8.70	< 18 %	competent
	MDD (mg/m^3)	2.06	1.94	2.09	2.08	2.07	2.06	> 0.04 mg/m^3	competent
	California Bearing Ratio (CBR)								
	CBR (%)	51.70	49.00	51.00	50.00	48.30	50.00	≤ 10%	competent
	Specific Gravity								
	G_s ($\mathbf{mg/m^3}$)	2.61	2.62	2.64	2.60	2.57	2.61	≥ 2.2 mg/m^3	competent
	Atterberg Limits								
	PI (%)	14.00	19.00	15.00	19.00	16.00	19.00	≤ 20 %	competent
	LL (%)	34.00	35.00	33.00	36.00	37.00	35.00	≤ 40 %	competent
	Particle Size Distribution								
	% of Fines	36.00	37.00	35.00	34.00	38.00	36.00	< 50%	Competent

DISCUSSION

The interpretation of the 2DERT model is based on the knowledge of the geology of the area, published standard resistivity range of values (IEEE (2011); Loke, (2000); Telford *et al.*, (1990); Reynolds (1997)) and previous published works (Ehirim and Ebeniro, 2010). According to the geology of the study area, sandy soil, clayey soil, shale, sandstones are predominant at shallow the depths (Reyment (1965); Ezenwa (1998) and Ekwe *et al.* (2006)). Hence, the 2DERT have delineated inverse apparent resistivity models of shallow subsurface materials in the study area. The models are generally characterized by resistivity range of about 1.00 Ωm to 5601 Ωm . The delineated range of resistivity encompasses clay (1-100 Ωm), sandy-clay (40-300 Ωm), sandstone (1-5000 Ωm), shale (20-2000 Ωm) and water saturated zone (0.2-5.0 Ωm). From the tomograms, it was observed that the topsoil at the study area (sandy-clay of various consolidations), features relatively heterogeneous and fractured zones. The topsoil also shows patches of low resistivity zones some of which were labeled *A*, *B*, *C*, *D* and *E* (Figure 5, 4 and 3). These low resistivity zones delineated and interpreted as clay/shale suggests that they are zones of low competency which are inimical to road construction. Obviously, tomograms P1, P2, P4 and P5 showed decrease of apparent resistivity with depths. Therefore, gradational decrease in consolidation of sandy clay soil and, conglomeration of various consolidation of shale were delineated. However, some zones of very low resistivity were interpreted as water-saturated zones such as zones A and B on Profile 4 (Figure 5), zones C, D and F on profile 2 (Figure 3) and zone E on profile 3 (Figure 4). On Profile 5 (Figure 6) is shown significantly a relatively consolidated top soil which is underlain by water saturated zone G.

Water saturated zones pose a very high degree of threat to the condition of the sub-grade material during preliminary stage of road construction. Hence, this could lead to road failure. Relatively high resistive zones sand and clayey sand delineated at the depths usable for road sub-grades P1, P2, P3 and P4 (Figure 2, 3, 4, and 5) suggest high soil competency against bearing pressure against during the road use. Conversely, the delineated low resistivity intercalates either on the surface or in topsoil horizons are which were clayey and water saturated suggest poor competency against expected bearing pressure during road use. The zones are causatives of potholes and breakages after before or during the tertiary settlements of road pavements. The weak zones identified as low resistivity zones are highly susceptible to weathering and deterioration of the sub-grade which could lead cracks to road failure. There are both faint and obvious evidence of inferred fracture in the delineated topsoil as shown in the tomograms P2-P5 (Figures 3-6). Particularly, within the topsoil of P5 (Figure 6), a fractured zone H is shown. Fractured ground surface and topsoil suggests impending danger to the sub-grade if not addressed before construction of road pavement.

The implication and dangers of clayey soil to roadways are manifest when they predominantly occur as topsoil used as sub-grade without excavation and embankment before construction of pavements. Clayey soil is noted for its seasonal swell and shrinkages which continually reduce the shear strength and incompressibility when left to serve as sub-grade for road pavement. The delineated clayey topsoil delineated from the 2DERT agrees well with the geology of the area which has informed in the foregoing that the area contains clayey materials at its near-surface even as its younger alluvium. The occurrence of clayey soil at shallow subsurface in the study area is a major factor in determination of the performance of the sub-grade during the use of the road. However, the extent to which sub-grades would bear the pressure during road use is dependent on the percentage or ratio of competent soils and clay which formed it. This is accentuated from the analyses of geotechnical test results for plausible conclusion.

Soil with amount of fines less than 50% are expected to possess better engineering properties when compared with those with amount fines greater than 50% which tends to pose field compaction problem when used as sub-grade material (Oyediran and Durojaye, 2011). Based on the results from the Particle size distribution (PSD) test, the samples have fines of range from 34.00 % to 38.00 %. Therefore, the sub-grade would not pose field compaction problem during the road use. The results of CBR test for soil samples presented in Table 2 show that the CBR obtained range from about 48.30 % to 51.7 % with an average value of about 50 %. The recommended minimum CBR for soil use as sub-grade material is ≤10% for unsoaked soil (FMWH, 1997). This implies that the minimum strength for sub-grade/fill shall not be less than 10% after at least 48 hours soaking. Subjection of the samples to at least 48 hours during the test, the sub-grade samples in the area are inferred to be fit for the road construction. However, PI values of the samples indicated high proportion of clay which at high consolidation are shale, suggests high potential of swelling and shrinking in the sub-grade, thus would promote pavement defect after construction (Egwuonwu *et al*., 2011).

The geotechnical results of the sub-grade samples shown in Table 2 were evaluated using American Association for State and Highway Transportation (AASHTO) official classification system (3). The system comprises seven groups of inorganic soils, A-1 to A-7 based on particle size distribution, liquid limit, plasticity index respectively (Braja, 2010). The classification encompasses clayey-sand, sand and clay which can be rated from excellent to poor material for road construction (AASHTO, 1982). Based on the range of values of LL and PI from the geotechnical tests, the sub-grade fell under the group classification of A-2 and A-6 of the AASHTO classification system. Although the Plastic Index (PI) of the sample met the FMWH required standard, its high percentage of clay (Table 4) is capable of reducing the shear strength the sub-grade by its susceptibility to permeability. This would pose a threat to the sub-grade if proper measure is not taken to address it.

Table 3: Revised AASHTO System of Soil Classification (Braja, 2010)

General Classification	**General Materials (35% or less passing 0,075 mm)**							**Silt-Clay materials (more than 35% passing 0.075 mm)**			
Group Classification	A-1			A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-a	A-1-3	A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis % passing											
2.00 mm (No.10)	50 max										
0.425 (No. 40)	30 max	50 max	51 min								
0.725 (No. 200)	15 max	25 max	10 max	35 mas	35 max	35 max	35 max	36 max	36 max	36 max	36 max
Characteristics of Fraction Passing	6 max										
Liquid Limit			N.P.	40 max	41 min	40 max	41 min	40 max	41 min	40 max	40 min
Plastic Index				10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of Significance	Stone Fragment		Fine	Silty or Gravel and sand				Silty Soils		Clayey Soils	
Constituent Material	Gravel and Sand		Sand								
General Rating	Excellent to Good							Fair to Poor			

Table 4: Type of soils based on plasticity Index (Prakash and Jain, 2002)

Plasticity index (%)	Soil type	Degree of plasticity	Degree of cohesiveness
0	Sand	Non-plastic	Non-cohesive
<7	Silt	Low plastic	Partly-cohesive
7-17	Silt clay	Medium-plastic	Cohesive
>17	Clay	High-plastic	Cohesive

Finally, comparing the interpreted results from both the 2DERT and the geotechnical tests in the consideration of the general geology of the study area, there are points of agreement. First of all, it can be observed that the PI tests results (14-19%) are within the classification of clayey materials in the AASHTO. This agrees with the geology of the area comprising Imo shale which is consolidated clay hence, the sub-grade classified as consolidated clay sample are inferred as shale. The consolidated clay (shale) and sandy-clay were observed to have occured in various formation aggregates at the depths of the sub-grades. The wide range of clayey materials obtained from the 2DERT (1-300 Ωm) and shale (20-2000 Ωm) which were predominantly interpreted topsoil lithology agree with the geotechnical tests' deduction of clayey soil. Also, in agreement with the geology of the study area, the delineated shale from both the geophysical and geotechnical surveys suggest the occurrence of the thick, clayey, fine-textured, dark grey to bluish grey coloured *Imo* (Reyment, 1965). Since the climate of the study area is characterized by distinct seasonal wet and dry seasons, extreme variation in moisture content of topsoil lithology in the area is commonly obtainable. Hence, based on the fact that electrical resistivity of the near-surface lithology varies with variations in water content and the dissolved ions in them in soil formation, the 2DERT interpretation justifiable. Consequently, the seasonal variation in water content of the subsurface soil (the sub-grade) would be susceptible to seasonal shrinking and swelling depending on the concentration of clay in them (Egwuonwu *et al.*, 2011). The inferred agreements between the interpreted geophysical and the geotechnical investigations and their justification of the underlying geology of the study area are considered plausible for reliable recommendations.

CONCLUSIONS

The findings from the interpretation of 2DERT have confirmed the analyses made with the geotechnical technique. Hence, it is concluded that the soil formations at the study area generally encompassed by sand, clayey-sand, clay and water saturated zones. Predominantly, sand, clayey-sand and clay characterize the topsoil along the surveyed parts of the roads proposed for pavement construction. The 2DERT survey has unveiled the extent to which the interpreted lithological materials of the study area's shallow depths were laterally distributed at shallow depths. Consequently, the geotechnical analyses showed that the competency of the predominant topsoil falls within the acceptable range of FMWH standard for road construction. Various proportions of clayey material in the sub-grade as indicated from the plastic index range indicate that that there is no lateral continuity of homogeneity in the formation of the competent soils in the area. Heterogeneity and the occurrence of incompetent and clayey soils shown in the 2DERT psuedosections suggest the danger of differential settlement during road use in Awka. Consequential to this is the occurrence of road defects and failures such as rutting due to lateral movement of sub-grade during traffic, upheaval, which is the swelling of the sub-grade due expansive (clayey) soils enrichment and subsidence due to sharp depressions leading to pot holes. The non-homogeneity of the sub-grade materials could also cause the occurrence various kinds of cracks on the road pavement especially edge and alligator cracks due to uneven bearing capacity envisaged. The laterally delineated irregularities in subsurface even at the depths of the expected sub-grade has unveiled the need for the practice of removal of incompetent soil and the piling of competent soil as sub-grade. Therefore, road embankment of the proposed sub-grade with competent soils at some portions along the surveyed roads in Awka-south metropolitan is imperative.

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