

**GEOELECTRICAL AND GEOTECHNICAL INVESTIGATION OF
FAILED SECTION OF ORSU-IHALA ROAD, IMO STATE SOUTH-
EASTERN NIGERIA**

BY

AJAH, NNAMDI JOSEPH (B.Sc EBSU)

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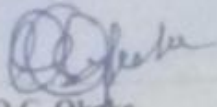
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CERTIFICATION

This is to certify that this Research work "Geoelectrical and Geotechnical investigations of Oru-Isala, southeastern Nigeria" was carried out by AJAH NNAMDI JOSEPH (20154241500) in partial fulfillment of the requirement for the award of the degree of Master of Science (M.Sc) in Geophysics in the department of Geology of Federal university of Technology, Owerri.

Prof. C.C.ZAKAOLISA
(Supervisor)

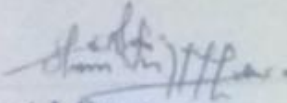
DATE



12/12/2019

Prof O.C. Okoko
(Co-Supervisor)

DATE



03/02/2019

Dr. A.I. Opara
(Head of Department)

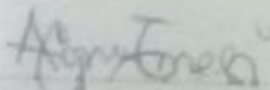
DATE

PROF. C.C.Z.Akaolisa
(Dean, School of Physical Science)

DATE

Prof. Mrs. Nnenna. N. Oti
(Dean, Postgraduate School)

DATE



12/12/2019

Prof. A.G. Onwuemesi
(External Examiner)

DATE

DEDICATION

This work is dedicated to Almighty God and to my wonderful family

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First I give praise to God Almighty, for the strength and his blessing in finalizing this research.

I owe my most sincere gratitude to my wonderful supervisor Prof. C.C.Z Akaolisa for his continuous support during the course of my research, his guidance, encouragement and patience.

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ABSTRACT

Geophysical and Geotechnical investigation of the causes of the consistent failure of Orsu-Ihiala road has been studied. The study area is part of the Anambra Basin consisting of Eocene Ameke formation, the Oligocene Ogwashi-Asaba Formation and the recent Meander belt. 2-D electrical resistivity tomography of the subsurface was carried out using Wenner method (for $a = 3, 6, 9$ and 12 respectively), 6VES was run along the failed and partially stable part of the road using Schlumberger method (with maximum $AB/2 = 55\text{m}$). Three (3) samples were collected at depth of 1.5m (subgrade) from the failed part of the road (collected from Amanachi, Orlu LGA headquarters and Amagriget) and were analyzed at Arab contractors limited. The 2-D pseudo resistivity section at the subgrade (less than 2m) shows that the area is underlain by moderate to low resistivity ($10\Omega\text{m} - 135\Omega\text{m}$) which is interpreted as clayey topsoil and clay. These zones coincide with the fairly stable and failed portion of the road. At the same depth resistivity was observed to be decreasing NW-SE, implying the clay content decrease along the NW -SE direction on the road. Analysis of VES suggested 3-4 geoelectric layers defined as clayey topsoil, clay, clayey sand and sand with resistivity of $96\Omega\text{m} - 136\Omega\text{m}$, $54\Omega\text{m} - 135\Omega\text{m}$, $288\Omega\text{m}$ to $980\Omega\text{m}$ and $2780\Omega\text{m} - 32800\Omega\text{m}$ respectively. The geotechnical results reveal that the soil sample is made up of granular material with highly compressible silty clay belongs to group A-2, subgrade A-2-6 (sample S01) and group A-7, subgroup A-7-5 (sample S02 and S03) as per AASHTO classification. The result of the Atterberg Limit showed that the plastic index (PI) is relatively low (16.9%, 7.1% and 14.9% for S01, S02 and S03 respectively). The LL for S01, S02 and S03 are 38.5%, 42.5% and 46% respectively and are referred to as intermediate plasticity. The maximum dry density (MDD) and optimum water content (OMC) are $1.776\text{g}/\text{cm}^3$ and 14.4% for S01, $1.713\text{g}/\text{cm}^3$ and 14.6% for S02 and $1.692\text{g}/\text{cm}^3$ and 14.4% for S03. Low MDD with high OMC is an indication that the soil is generally loose. The California Bearing Ratio (soaked) for sample S01, S02 and S03 are 2.1%, 3.4% and 16.2% respectively. S01 and S02 fall within FMW specification while S03 is above the specification. The specific gravity of studied samples is $2.401\text{g}/\text{cm}^3$, $2.586\text{g}/\text{cm}^3$ and $2.413\text{g}/\text{cm}^3$ for S01, S02 and S03 respectively. Which indicates moderate organic content and moderate porosity (especially sample S01). The moisture content of the samples is 18.2%, 24.2%, 25.4% for S01, S02 and S03 respectively. The material is generally rated as a fair to poor subgrade material (as per general specification Roads and Bridges, Revised 1997, FMW, Nigeria). 2-D pseudo resistivity section, VES and the geotechnical result shows that the road was founded on a subgrade (at depth 1.5m) clayey material. And that the clay content of the subgrade material decreases along the NW-SE profile resulting from transition from Recent sand rich meander belt to clay rich Ameke Formation. The road failed basically due to the clayey material found at the subgrade and poor drainage construction. This project therefore shows that geophysics complement Geotechnical analysis and used be carried out prior to construction of engineering structures.

Keywords: *Geoelectrical, Geotechnical, Geophysical, Resistivity, Orsu-Ihiala, South-Eastern Nigeria*

CHAPTER ONE

INTRODUCTION

1.1 Background Information

The foundation upon which engineering project (constructions) lies is the earth. The type and nature of this subsurface material is essential as regards to how the construction project would fair. Hence it is therefore of paramount importance that the nature and distribution (Heterogeneity/Homogeneity) of subsurface are studied prior to the construction of any engineering project(Dan Ruffing, 2016).

All engineering project rests directly on soils or rock as the case maybe. Olorunfemi et al., 2004 proposed that this soils and rock which serves as host for this construction project should be properly studied to enable good design and construction. There is record of failure of Majority of construction projects especially roads, due to no studies or inadequate studies prior to construction. These failures result from induced tectonic and anthropogenic activities causing inhomogeneties in the distribution of earth materials. Tectonic activity creates minifractures and fault which can alter the original distribution of earth materials in the subsurface.

Geoscientist have pointed out that geological features like nature of the soil (topsoil, subgrade, and sub base), pre-existing faults/fractures, composition/strength of the bedrock and presence of ancient stream channels can affect the durability of any engineering in Nigeria (Pazzi et al., 2016). Routinely it is very common to carry out Geotechnical studies prior to construction of any project but in Nigeria the above routine is neglected or in a best case scenario carried out partially(Pazzi et al., 2016).

The strength of geo-materials varies significantly geographically, this may be due to local geology (depth of geo-material, composition and nature of geo-material) and it is therefore important that strength, nature and composition of the subsurface geo-materials are studied using geotechnical and geophysical methods. Because the geotechnical investigation is very much time consuming and expensive, a surface geophysical method is sometimes used to complement the geotechnical method (Omowumi, 2014).

Geophysical techniques are the most cost-effective method adopted for subsurface investigation and studies as they offer pre-construction to after-construction studies for construction activities (Savvaidis, *et al.*, 1999 and Soupios *et al.*, 2007). This is possible because this method allows for cheaper, more convenient and non-destructive investigation. With the Rapid advances in compute iterative software geophysical has made surface geophysical method the prime method for investigating subsurface geo-material (Omowumi, 2014).

A good road pavement is a stretch of smooth laid down asphalt for a comfortable ride. A bad road or road failure is therefore any visible cracks or holes that may obstruct such smooth ride (Aigbedion, 2007). Road failures is said to be jointedness in a road network resulting in cracks, potholes and depressions.

Most Nigerian paved roads are known to fail soon after building and way before their life span (design age). It is however becoming a routine ritual for roads to be continuously repaired shortly after construction. A big reason for this is that no adequate knowledge of the surface is known prior to the construction of the road or the poor nature of the construction material or that the road rest on a geological difficult material (Aigbedion, 2007).

It is no news that, bad portions of road, may result from poor construction or as a result of been built on an incompetent materials. And these have been responsible for server deadly accidents, causing loss of lives and properties.

Geophysical/Geotechnical information, if properly constrained with supervised borehole derived lithological logs, will give out important understanding about the nature of the subsurface material or structured investigated. Geological derived materials/structures that can contribute to road failures includes:

- 1 Geology of the area (near-surface geology and presence of geological structures such as fractures and joints)
- 2 Placement of groundwater (shallow water table)
- 3 Nature of the geologic materials upon which the road rest directly like: limestone, chalk, gypsum and clay
- 4 Peat and unconsolidated recent deposit
- 5 Presence of old stream channels and failure zones,
- 6 Geomorphological factors, topography and surface/subsurface drainage system.

Investigation conducted by the department of Material Geotechnics and Quality control Federal Ministry of Works Nigeria, revealed that most premature failures experienced on our roads are attributed to weak subgrades. In order to address this situation it has become imperative to review the subgrade specification to ensure that highway pavement is constructed on sound foundation that can withstand harsh environmental conditions. The current subgrade specification is as follows;

1. Percentage passing sieve 200 not exceeding 35%

2. Liquid limit not greater than 50%
3. Plastic limit index no greater than 30%

The behavior of subgrade soil in a completed road structure is of paramount importance especially when the stability and performance of the structure is most critical. This would in most cases be at the rainy period when the degree of water saturation in the subgrade is increased. Underground water places a significant role in the stability of the subgrade. It is therefore important that subgrade materials should have some level of strength under saturated condition. The parameter to measure this strength is California Bearing Ratio. It is therefore important to obtain an accurate geotechnical data on subgrade as a precondition to accurate design of pavement that would lead to durable roads.

The use of near surface geophysical investigation methods to complement geotechnical studies in evaluating the causes of road/structural failures will never be overemphasized. This research therefore integrates electrical resistivity and geotechnical techniques to investigate the causes of the repetitive failure of Orsu-Ihiala road, Southeastern Nigeria.

1.2 Problem Statement

Consistent failure of some part of the Orsu-Ihiala road has attracted a lot of attention recently as the need to properly investigate and if possible, infer a solution has become an inevitable quest. The road is an important road linking Imo to Anambra (Orsu- Ihiala) and has failed repeatedly shortly after repair, causing discomfort drive, delay in trip time vehicle spoil, longer travel time, increased transport fee, accidents and even loss of lives (Plate 1.1). Approximately 100 persons lose their lives daily as a result of accidents according FRSC, 2017.

1.3 Objectives of Study

1.3.1 Main Objectives

This research contains geoelectrical and geotechnical investigation of failed section of Orsu-Ihiala road, Southeastern Nigeria.

1.3.2 Specific objectives

The specific objectives include:

1. Delineate the various lithologic units underlying the study area
2. Produce 2-D pseudo section of the failed, partially stable and stable sections of the road
3. Produce electrical resistivity curves of the failed and partially stable part of the road
4. Produce a correlation profile of the geoelectrical sections of the failed and partially stable part of the road
5. Classify the competency of the subsurface material based on resistivity values
6. Carry out geotechnical analysis (sieve analysis, natural moisture content, Aterberg limit, compaction test, California bearing ratio and specific gravity) and compare results with ASSTHO and FMWH standards.
7. Integrate 2-D, I-D resistivity and Geotechnical analysis to unravel the causes of Orsu-Ihiala road failure
8. Infer possible solution or remedy to the failed road

1.4 Justification of Study

This Research came up, due to the fair to poor function of some paved roads to human or society needs, in terms of causing stress upon discomfort ride, extending trip time, delay easy traveling and service delivery. Nigeria paved roads are always allowed to deteriorate and cause many travelers risks of accident, injury, permanent disability, damage or loss of properties and loss of life. Because the society depends heavily on efficient roads for its economic and social well-being, hence there is need to evaluate the causes of failure of this road and to employ an efficient way to mitigate the aforementioned problem. The choice of choosing this road (orsu-ihiala road) was because of its consistence failure shortly after repair.

1.5 Scope of study

This research involves to review and investigate Orsu-Ihiala road using geological, geophysical and geotechnical methods and hence evaluate the causes of the consistent failure of the supposed road (even when it has been repaired repeatedly).

This research contains Electrical resistivity (1D/2D) and geotechnical (sieve analysis, compaction test, Atterberg limit, specific gravity, natural moisture content and California Bearing Ratio) analysis.

This research is limited as the soil samples analyzed are representative of only the failed apart of Orlsu-Ihiala road. And it is important to note that XRD and Geochemical analysis were not carried out on the samples.



Plate 1.1: Severe Potholes and Ponding Water on Potholes at (A) Amanachi and (B) Isseke (Orsu-Ihiala road).

CHAPTER TWO

LITERATURE REVIEW

2.1 Previous study

The study area is located in Imo State, Southeastern Nigeria. The area fell within equatorial rain forest belt of Nigeria with mean annual rainfall is about 3,100mm (National Root Crop Research Institute, 2012). Several studies have been carried out to investigate near subsurface structures within the study, all in a bid to solve hydrological, environmental and geotechnical problems. Onu (1995) carried out a hydro geophysical study of the Njaba River sub-basin using electrical resistivity method.

Apakama (2010) studied the determination of the sub-surface deposit of the southern part of Orsu and its Environs using vertical electrical soundings. He found out that the major lithologic units underlying the area are sand, sandstone, clay and mixture of sand, slit and clay occurring at varying depths and with varying thickness.

Field observations and laboratory analysis carried out by Adegoke-Anthony (1980), Mesida (1981), and Ajayi (1987) showed that road failures does not necessarily results fromill-usage or poor design but can also result from ill knowledge of the behavior of soil on which the road rest directly upon.

Nelson and Haigh, 1990 suggested that hidden geologic structure influenced by joint, fracture or mineral leaching also contribute to the deterioration of paved roads. They also concluded that another key factor to consider is the geomorphological factors as this relates to topography, runofffs and sub-surface drainage systems.

Oladapo et al., 2008 explained that the reliability of the highway pavement can be dented by the presence of geological features and that ignorance of this fact has resulted to premature failure of well paved road.

Emujakporue (2012) suggested that most paved roads in Nigeria fails due to ill design/construction and they specified that high percentage of Nigerian road are built with poor drainage facility or design ignoring geological factors.

Momoh et al., 2008; Onuoha, Onwuka&Obienusi 2014. All explained that failures of high ways are due to due to insufficient knowledge of the geotechnical attributes and behaviour of residual soils that the paved road sitsdirectly upon.

Layade et al 2017 in their research Titled, Integrated geophysical investigation of causes of road pavement failure along Ibadan-Lagos dual carriage, Southwestern Nigeria. Used Very low frequency and electrical method to evaluate near surface earth material. They concluded that the failure of the road is caused mainly by geologic sequence and structures such as clayey subgradesoil under the road pavement and some suspected geological features such as faultsand fractures.

Adenika, et al, 2018. Applied geophysical approach to investigate highway pavement failure, using basement complex terrain southwestern Nigeria as a case study. They adopted both electrical resistivity (Schlumberger VES and 2-D dipole-dipole) and ground magnetic for their study. They concluded that the failed portions of the road are probably precipitated by very thick and low resistive substratum (clay) beneath the highway pavement and the identified suspected linear features are the major geological factors responsible for the highway pavement failure.

Osinach et al, 2018 deployed geotechnical and electrical resistivity method in investigating failures in engineering structures around Kogi, North Central Nigeria. In their study they evaluated cracks in buildings around Kogi state Polytechnic and their results indicate that the surroundings is composed of poorly graded soil with poor geotechnical properties. Electrical investigation of deeper soils indicates that shallow layers are made up of top soil and weathered soil which they classified as incompetent soil. They concluded that cracks which led to differential settlement was as a result of incompetent soil found at shallow depth.

2.2 Location and Accessibility

The project area is found within Orsu LGA Imo State Southeastern Nigeria It lies between Latitudes $5^{\circ}47'0'' - 5^{\circ}54'0''$ N and longitudes $6^{\circ}51'0'' - 7^{\circ}00''$ E (Fig 2.1)

The area is found at the northern section of Imo state. The area is bounded to the north-west by Anambra State, to the East by Okigwe and to the south by Owerri. The area is accessible from Orlu, Lili, Azia and Ihiala.

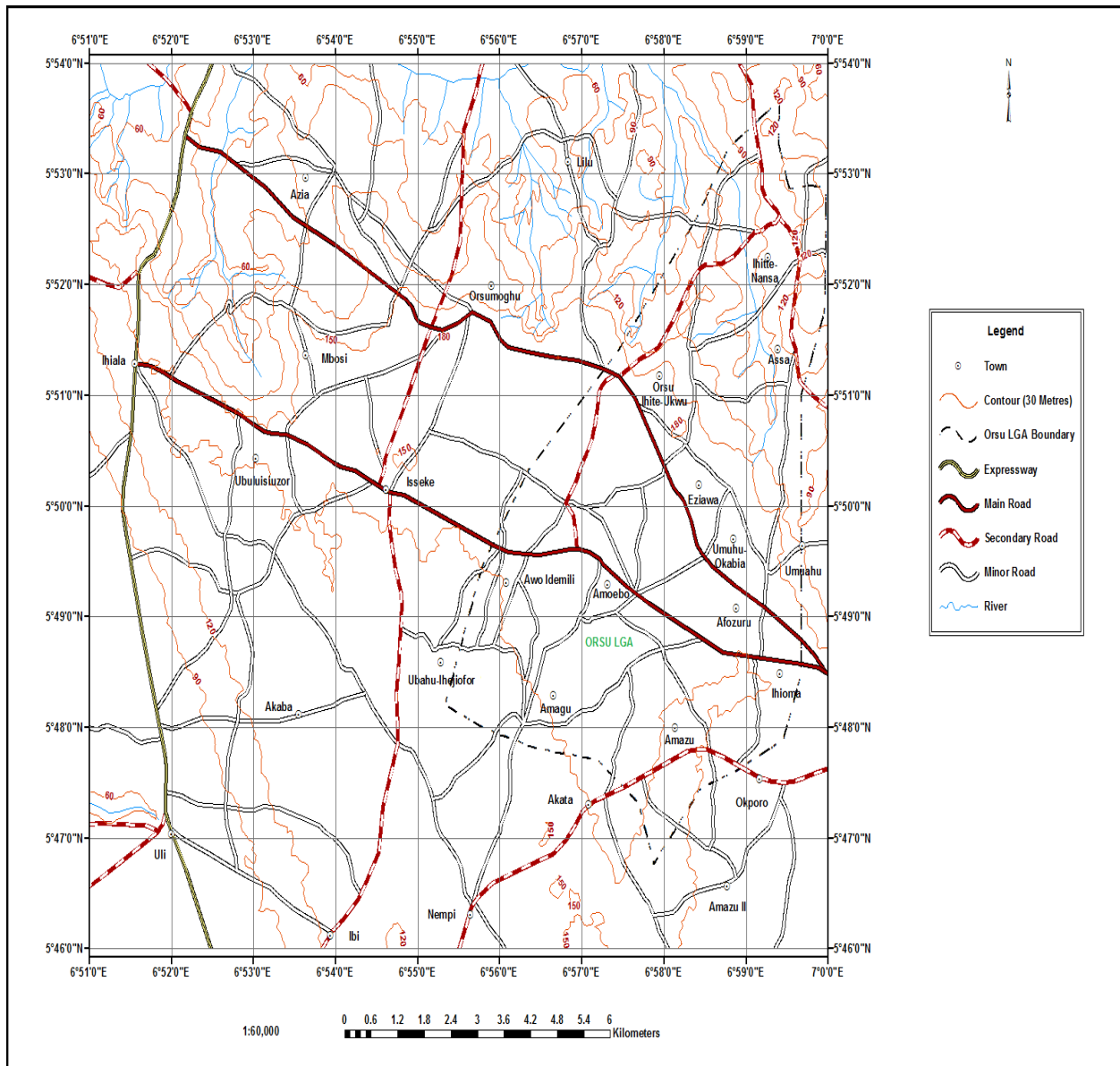


Fig 2.1. Location map of the study area

2.3 Geology

2.3.1 Regional Geology

The study area falls under Anambra basin. The Anambra basin resulted due to high degree Middle–Santonian deformation and magmatism in the Benue Trough which led to the westward displacement of depositional axis. The basin therefore is a product of the Post deformational sedimentation in the Lower Benue Trough. Sediment supply in this basin was observed within Campanian – Eocene of the Enugu/Nkporo Formations and Nanka Sand respectively.

The Enugu and Nkporo shale of a regressive marine background which is directly overlain by the Mamu Formation followed by the Ajali Sandstone then the Nsukka Formation and lastly the Nnaka and. The Miocene Ogwashi-Asaba Formation rest directly on the Nnaka sand.

The Eocene Nanka Sands indicates the reappearance to progradational conditions. Nanka Formation is overlain by sandstones, shales and lignite beds of the Oligocene Miocene Ogwashi–Asaba Formation. These Tertiary units constitute the proto- Niger Delta Eocene to Recent sequences in the subsurface (Fig 2.2).

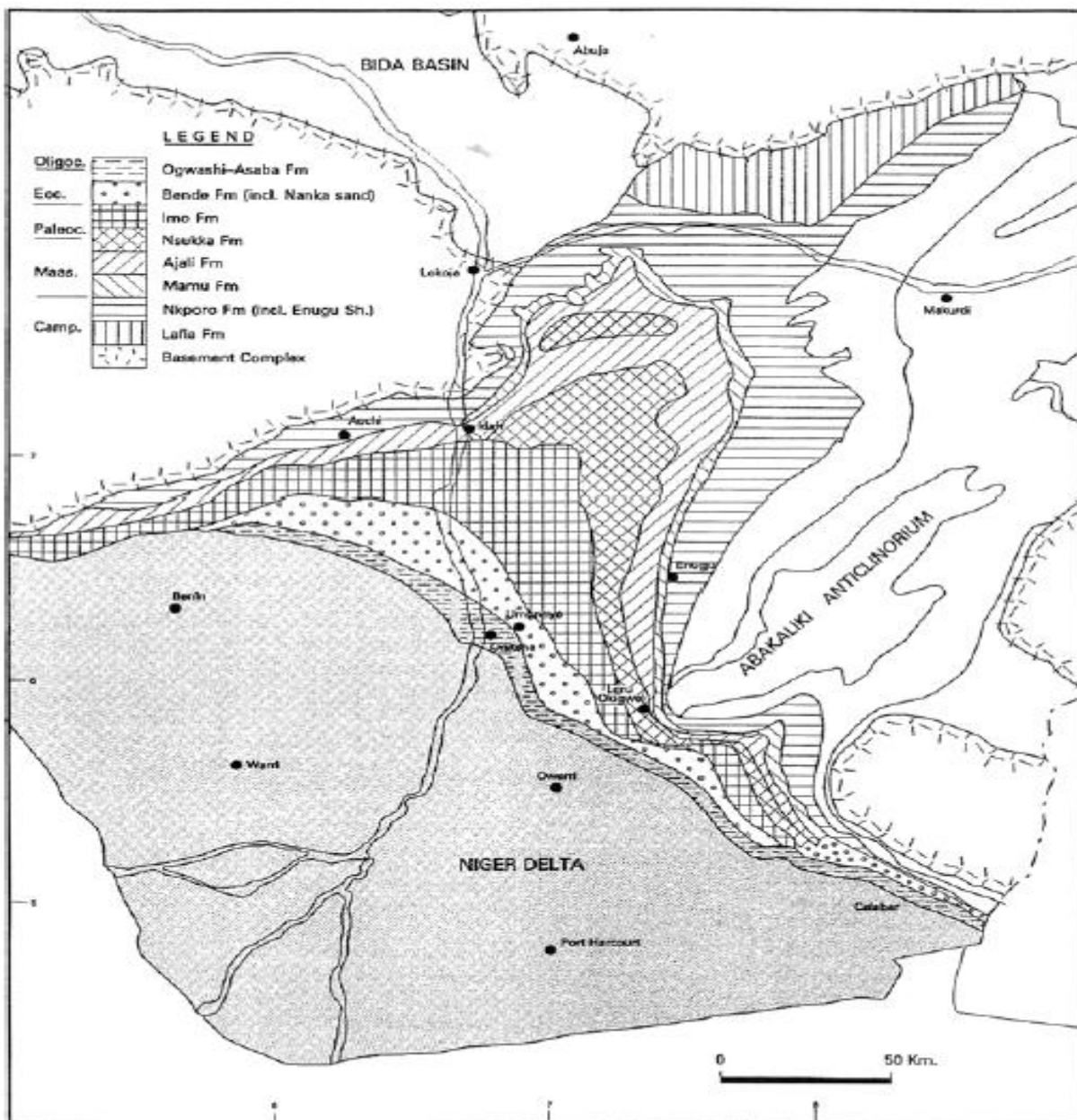


Fig 2.2: Geology map of Anambra and Niger Delta Basin (After Obaje, 2009)

2.3.2 Local Geology

The researched region composed Anambra basin (Ogwashi Asaba Formation and Ameke Formation). Which is made up of three major packages, with Ameke Formation as the oldest, the Ameke Formation underlain the Ogwashi Asaba Formation and this in turn underlain the recent Meander belt. Stratigraphic succession is shown in Table 1.1.

Table 2.1: Stratigraphy of the study area (modified after Uma and Egboka, 1985)

Period	Age	Formation	Lithology
Quaternary	Recent	Meander belts	Sand, gravel and clay
	Oligocene	Ogwashi- Asaba	Consolidated sands with lignite seams
	Eocene	Ameke	Clayey sandstones and sandy claystone

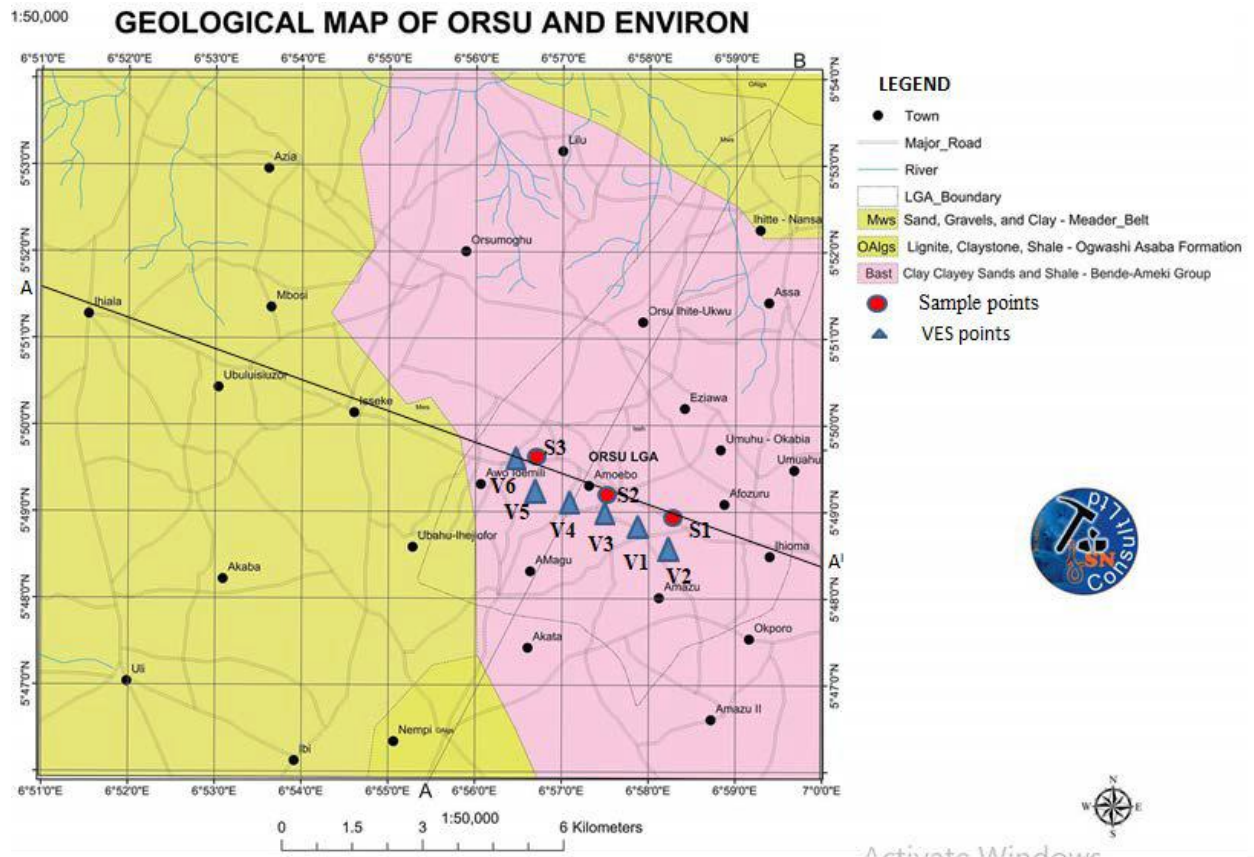


Fig 2.3: Geological map of the study area (Nigeria Geological survey)

Ameki Formation

The AmekeFormation is of the Eocene-Oligocene age and it consists of sandstone (coarse-medium), pebbles, with little silt, mottled clays and thin sheet of limestone. Considerable difference in it lateral lithologic extent has been identified. The bottommost part of the formation is made up of lenses of sandstone.

Ogwashi/Asaba Formation

The Ogwashi/Asaba Formation popularly known as the Lignite series is observed within the palaeocene Anambra basin (Oboh-Ikuenobe et al., 2005). The zone is composed of clays, sands, grits and lignites (Bassey & Eminue, 2012). The Ogwashi-Asaba Formation has the Agbada Formation as its equivalence (surface equivalent). The lignite seams within this Formation varies from locality to locality with an average thickness of 0.3km.

Recent meander belt

This consists of major sand deposit, gravel and little clay that varies in colour and thickness around the study area. This unit is the youngest as it is seen to overlay the thin lateritic layer in the study areas.

CHAPTER THREE

MATERIALS AND METHOD

3.1 Basic theory

The electrical resistivity method operates on the principle of Ohm's Law. The resistivity of any material is said to be its ability to produce some form resistance to conductivity across section of a unit area (A) and a unit length (L), see Fig 3.1. Explains the resistance of a cylindrical material

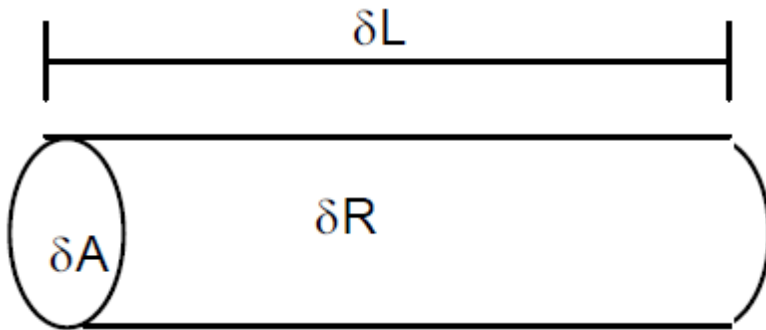


Fig 3.1: resistivity of a cylinder (after, Reynold, 1997)

$$\rho \propto A/L$$

$$\rho = RA/L \quad \text{hence,}$$

$$R = \rho L/A \tag{1}$$

$$\text{but } V = IR \quad \text{hence}$$

$$R = V/I \tag{2}$$

Equating (1) and (2)

$V/I = \rho L/A$, this implies

$$V/L = \rho I/A \quad (3)$$

R= electrical resistance,

ρ = resistivity (Ωm).

A reflection on a distinct point electrode situated at the margin of a semi-infinite electrically homogeneous medium that signifies a fictional homogeneous earth. The current (I) is sunk at a considerable large distance away from the electrode, current will flow radially away from the source electrode in such a way that it would be disseminated unvaryingly over the hemisphere shell centered at the surface. At a distance (r), the surface area is $2\pi r^2$ (see Fig 3.2).

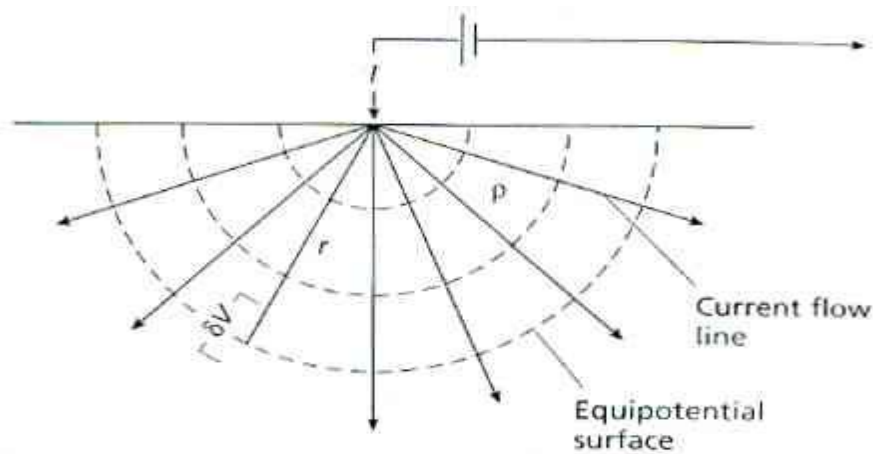


Fig 3.2: current flow for single surface electrode (modified after Kearey and Brookes 2002)

Hence (3) becomes,

$$V/L = \rho I/2\pi r^2 \quad (4)$$

Integrating (4) it becomes

$$\int \partial V = \frac{\rho I}{2\pi r^2} \int \partial r \quad (5)$$

$$V = \rho \frac{I}{2\pi r} \quad (6).$$

Let an electrode carry a current say (I), the resultant potential at every point on or below the surface of a homogeneous half -space is given by equation (6) above.

Where; V= potential

ρ = resistivity of a material,

r = distance away from the electrode

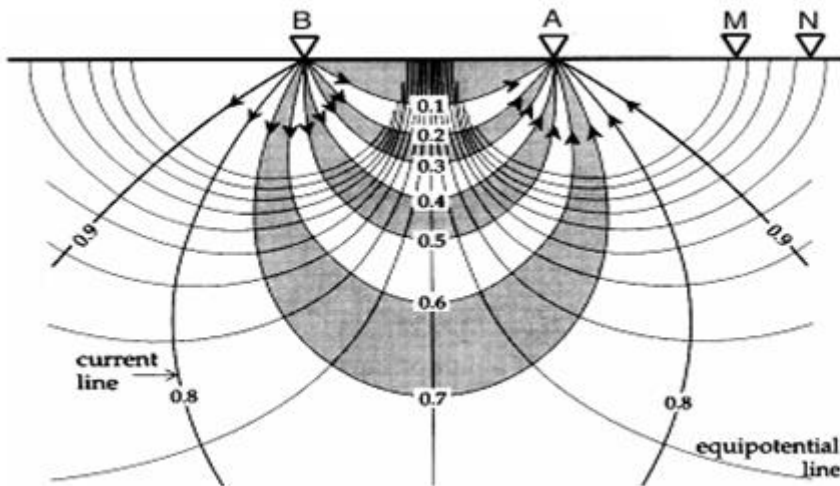


Fig 3.3. Distribution of current and potential lines for two current electrode at the surface homogenous half space (modified after Van Nostrand and Cook, 1966)

The mathematical estimation for the equation is explained in Kearey et al. (2002). For any electrode pair with current (I) at electrode A, and (-I) at electrode B (see Fig 3.4), the potential V_C at an electrode C is the cumulative of the entire electrode potential resulting from V_A and V_B as a result of the current source at A.

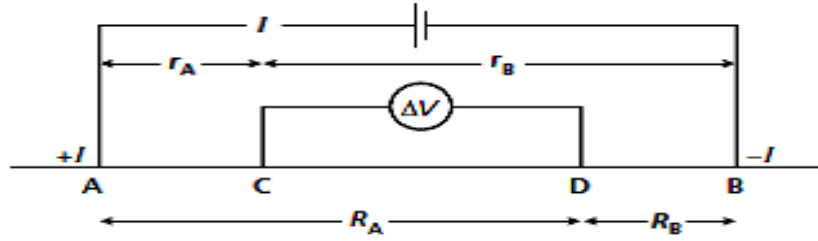


Fig 3.4: The general electrode configuration (modified after Kearey & Brookes 2002)

$$V_C = \frac{\rho I}{2\pi} \left[\frac{1}{r_A} - \frac{1}{r_B} \right] \quad (7)$$

R_A and R_B = distances from A and B electrode respectively

Similarly,

$$V_D = \frac{\rho I}{2\pi} \left[\frac{1}{R_A} - \frac{1}{R_B} \right] \quad (8)$$

Because absolute potentials are challenging to monitor, the potential difference ΔV in the middle of electrodes C and D can only be measured.

$$\Delta V = V_C - V_D = \frac{\rho I}{2\pi} \left[\frac{1}{r_A} - \frac{1}{r_B} + \frac{1}{R_A} - \frac{1}{R_B} \right] \quad (9)$$

Where; V_C and V_D = potentials at C and D respectively.

The above equation can be transformed into:

$$\rho = \frac{2\pi \Delta V}{GI} \quad (10)$$

where G = is defined as array geometric factor.

We then can obtain:

$$\rho = \frac{2\pi}{G} R \quad (11)$$

3.2 Fieldwork and Sample Collection

For this study, the adopted methods include geological, geophysical and geotechnical techniques. The Geological technique involves the identification of rock units and taken their associated geological measurements. The geology helped in constraining the interpretation of the geophysical results. After the geological mapping 2-D Wenner resistivity survey was carried out to reveal the nature of the near surface material across the failed portion, fairly stable and stable portion of the road. The 2-D pseudo-section guided the planning of the VES as sounding were done on zone of weak near surface material (as indicated by the 2-D Wenner Constant Separation Traversing). Subgrade samples were then collected from this weak zone and their geotechnical properties properly analyzed.

3.2.1 Geophysical survey

For the geophysical investigation, the use of electrical resistivity was employed; both VES (Vertical Electrical Traversing) and 2-D were carried out using Schlumberger and Wenner configuration respectively. The instrument used was Abemterrameter Signal Averaging System (SAS) 4000. As the name implies, the instrument is capable of taken up to ten or more consecutive readings and displaying their average. The continuously updated running average is persecuted automatically in digital form on the display unit in a cycle of 1-4 which can be selected by the operator. Accessories includes; four electrode, four wire rims, battery, marked ropes, measuring tape, hammer and a ranging pole. First 2-D Wenner was used to produce the pseudo-resistivity section across the road, and then 1D (VES) was run on the zone with poor near surface materials. This was done to see the nature of the deeper earth material.

A total of 6 VES and a 2-D electrical tomography were carried out for investigation of Orslu-Ihiala road failure. The choice of chosen Schlumberger and Wenner for this study are because, both field procedures are simple, field logistics are easy, penetration is high for Schlumberger , while data analysis is less wearisome and inexpensive (Ako & Olorunfemi, 1989). It is also associated with the competency to discriminate between saturated and unsaturated layers. Wenner configuration is known for its good signal to noise ratio especially for shallow depth and at such, it is best for carrying out 2-D at shallow depth.

2-D ERT (Electrical Resistivity Tomography) using Wenner array

Wenner method was used to carry out the 2-D resistivity variations in the near subsurface along the failed part and stable portion of the road.

A transverse was taken along the NW-SE trend on the road, the chosen transvers covers the failed, partially stable and stable portion of the road, to reveal near surface geoelectic view of the road. Wenner method was adopted for the near surface constant separation traversing (CST) with $a=3, 6, 9$ and $12m$. A total of $5.5km$ of the road was covered as CST was taken at every $500m$ (running from Amanchi – Mbosi junction). Surfer 13 software was used to produce the 2-D pseudo resistivty section of the road, this done by using the gridding method then applying krigging with the best linear unbiased estimator. This surfer 13 software allows for the creation of the 2-D resistivity view of the near surface using the already estimated resistivity values.

The electric current was introduced in the subsurface through the dual current electrodes, arranged on a straight track with the potential electrodes placed in between them at a $3m$ interval (say ‘ $a=3$ ’), the potential difference between the two potential electrode was picked and

measured by the tetrameter. After which the whole configuration were symmetrically (i.e, the 3m interval was still retained) moved to the point (measure 3m) and measurement taken again the. This procedure was continued till the profile was covered. The same procedure were repeated for a = 6, 9 and 12m respectively. 2-D data was inverted using interplex 1-D inverting software and suffer 13 (is a grid-based computer graphics program) was also used to produce pseudo sections of apparent resistivity.

Vertical Electrical Sounding using Schlumberger method

This technique allows for the supply of direct current into the ground through a duo of current known as the current electrodes and then measurement of the resulting potentials through another duo of electrodes called potential electrodes. Because the sunk-in current is known and the resultant potential are measured, apparent resistivity can therefore be estimated. The resistivity of any subsurface earth material is influenced by the magnitude of the current, the observed potential difference, and the geometry of the electrode a configuration adopted. Likewise, the depth of investigation is influenced by the geometry of the Schlumberger array used and the maximum spread of AB/2. For any electrode configuration adopted the apparent resistivity is given as. The basic equation is given as:

$$\rho_a = 2\pi G \left(\frac{\Delta V}{I} \right) \quad (12)$$

The intervals between the potentials and current electrode were increased at suitable paces in order to attain potential differences big enough to be recorded with adequate accuracy.

The recorded field data (resistance) was converted to apparent resistivity by multiplying with the Schlumberger geometric factor given as:

$$K = \pi \left(\frac{a^2}{b} - \frac{b}{4} \right) \quad (13)$$

Where “a” is the distance between the two current electrode and “b” is the distance between the two potential electrodes.

The obtained data was displayed on a graph of apparent resistivity against half current electrode spacing using computer iterative software (Zohdy and interplex 1D). The maximum current electrode spacing determines the extent of penetration. An approximation of two-third of the maximum current electrode spread is assumed to be the total vertical apparent depth of investigation (Flathe, 1970). The apparent resistivity and thickness gotten from partial curve matching was used as input data for computer iterative modeling (Ogilvy, 1970; Zohdy, 1976; Choudbry et al., 2001).

3.2.2 Sample collection

The investigations include field studies to collect samples from various points and depth. Soil augers were used in the collection of the samples from a depth of 1.5m at three (3) sample points along the failed and stable part of the road. A summary of adopted method is shown in Fig 3.5 below.

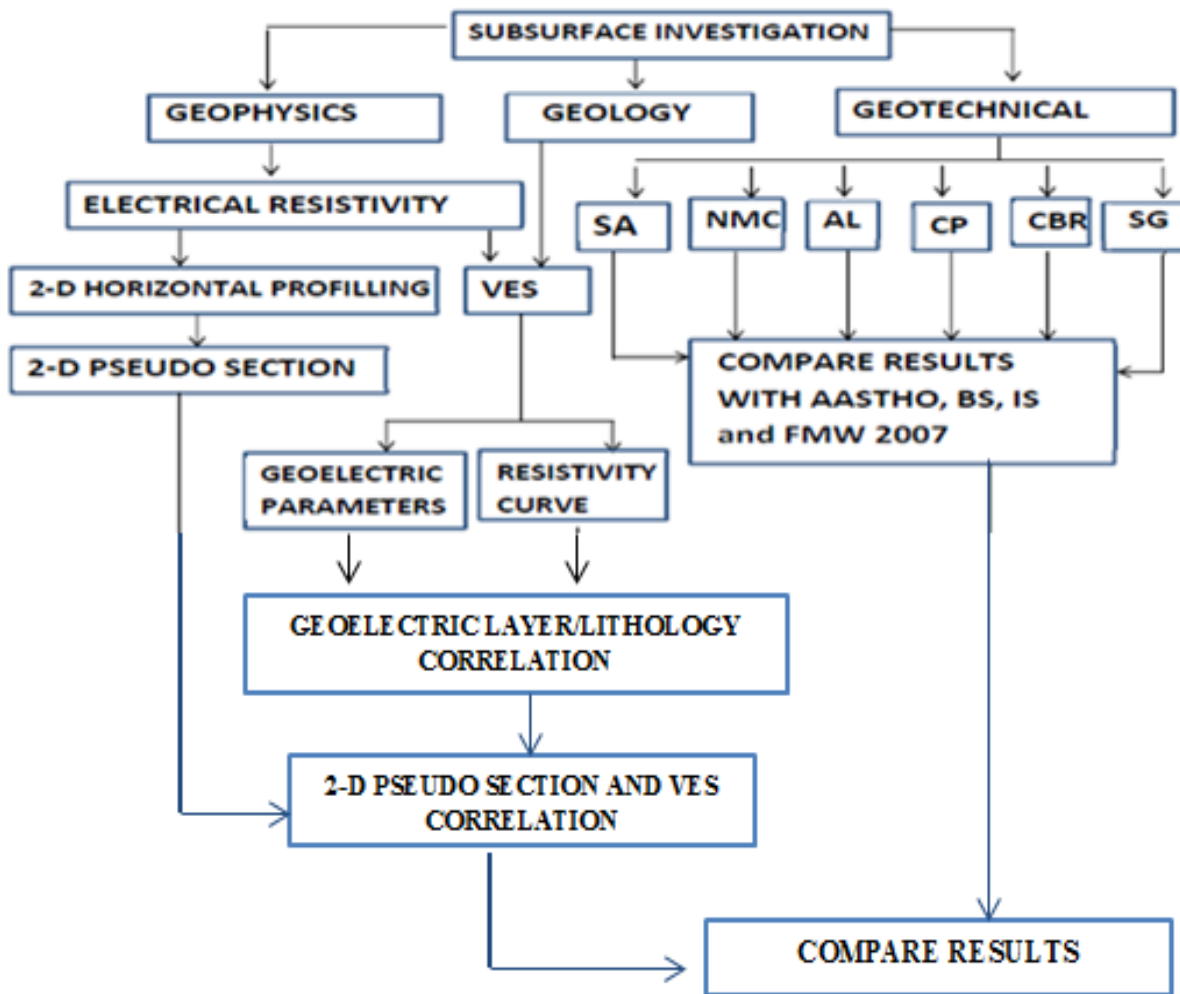


Fig 3.5: Flow chat of the adopted method

3.3 Laboratory Analysis

The samples were analyzed at Arab contractors A.O.O NIG .LTD. located at Alakwo, Owerri Imo state Nigeria. The laboratory parameters analyzed include: Sieve analysis, Atterberg limit, Linear shrinkage, dry density/moisture content compaction tests, laboratory CBR test, natural moisture content test (by speedy moisture tester) and specific gravity test (ASTM D 854). The laboratory analysis were done and compiled between 25th and 31st October, 2017.

These tests were analyzed in accordance with the general specification Road and Bridges, Volume II, revised 1997, Government of the Federal Republic of Nigeria and categorized employing the standard of American Association of State Highways and Transportation Officials (AASHTO) and Unified Soil Classification System. The result was meant to ascertain soil for engineering purposes: as applying necessary ASTM, British Standard and IS-methods for testing materials. The soil sample was duly air dried using drying tray (ASTM D421), samples greater than sieve 19mm (oversize) were removed using sieve 19.5mm and finally, quartered to suitable quantities required using riffle box (12.5mm slot). The laboratory tests were analyzed carefully to achieve accuracy. For CBR sample was soaked for 48hours. The tests conducted include:

3.3.1 Natural moisture content

Purpose: This test was carried out to estimate the original water (moisture) content of soils. The originally available in soil water, is defined as a percentage, of the mass of the water present in the pore” to the mass of the dry soil.

Standard Reference: ASHTO D 698 for the determination of water content in Soil, Rock, and Soil-Aggregate Mixtures

Significance: this is very important in understanding the relationship between the way soil behaves and its properties. A soil containing little water in its void especially if it's a sandstone will mean that the soil is well compacted and could have a good binding strength. The water content also helps depict the relationship between air, water and solid in the mass volume of a solid.

Equipment: oven, weighing Balance, Moisture can, Gloves and Spatula.

Procedure: About 50 grams of the sample was weighed and the sample was then oven dried to a constant weight at approximately 105⁰C-110⁰C for a specified period of 24 hours. It was then allowed to cool in a desiccator and then weighed. The weight of water was recorded as the loss in weight of the sample and recorded as a percentage of the dry sample to obtain the natural moisture content.

$$\text{Moisture content} = \frac{W_1 - W_2}{W_2 - W_3} \times \frac{100}{1} \quad (14)$$

W₁ = weight of wet soil

W₂ = weight of dry soil

W₃ = weight of moisture lost

3.3.2 Particle Size Distribution

Purpose: This is done to determine the percentage grain size makeup of a soil and to understand how these grains are distributed in the supposed sample.

Standard Reference: ASTM D 421

Significance: grain size distribution of particles has an influence on the engineering properties of soil and is required for the classification of soil base on engineering purposes.

Equipment: Balance, Set of sieves, Cleaning brush, Sieve shaker, Mixer (blender),

Procedure: A measure of the sieve and bottom pan was first done, then the weight dried sample. The sieves were arranged in the order of the largest to the smallest (#4 sieve situated at the uppermost and #200 sieve positioned at the bottom). The pan was positioned just below sieve #200. The sample was then emptied into the top sieve and a cap was used to cover it. The stack of sieves was then shook for 10 minutes using the mechanical shaker. After the elapse of the 10 minutes, the stack removed from the shaker and the residue on each sieve was carefully weighed.

3.3.3 Atterberg limit

Purpose: This test was conducted to be able to define the plastic and liquid limits of a fine grained soil. The liquid limit (LL) measure the water content, at which a fragment of soil cut by a groove of standard dimensions will flow collectively at the bottom of the groove along a distance of about 13 mm (1/2 in.) when subjected to 25 shocks from the cup dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, at which a given soil sample can no longer be deformed when rolled into 3.2 mm (1/8 in.) diameter threads without crumbling.

Standard Reference: ASTM D 4381

Significance: plastic limit measure the moisture content when the soil changes from a semi solid to a flexible (plastic) state where as the liquid limit defines change from plastic to viscous fluid

state. The shrinkage limit is the moisture content that points the limit of which the soil can no longer be reduced even with reduction in moisture content. Atterberg limit is used to classify engineering properties of soil. The Atterberg limits represent fundamental measures of the critical water contents of a fine-grained soil

Equipment: Evaporating dish, Flat grooving tool accompanied with gage, moisture cans, weighing Balance, Glass plate, Spatula, Wash bottle, distilled water, oven.

Procedure:

Liquid Limit: The Casagrande method was adopted for research but another method known as the cone penetrometer also exist.

200grams of each air dried soil sample was sieved through a 425um sieve. The soil was then positioned on a glass sheet and sterilized with little distilled water to form a thick paste and then left overnight. Thereafter the cup of the apparatus was half filled with the wet soil and the top leveled off. A 2mm groove was then made at the middle of the soil by the means of a grooving tool. The handle of the apparatus was rotated at 2 revolution per second and this caused the cup to lift 10mm and then to fall on to the rubber base continuously. The number of blows required to close the gap over 13mm was recorded and a portion of the soil just tested was removed for moisture content determination. The test was repeated five more times applying a little more water for each test. The liquid limit was obtained by plotting the variations of moisture content against the corresponding number of blow. The best fit curve was drawn across points and the moisture content equivalent to 25 blows estimated to be the liquid limit.

Plastic Limit: 20grams of soil prepared during the liquid limit test was used. The soil was mixed with water to produce a plastic state soil which was then rolled into a ball. The ball was then

rolled between the hands to form a thread. The soil is accepted to be at its plastic limit at the junction when it just begins to crumble at a thread diameter of 3mm. when this was achieved a portion of the thread-like soil was collected for moisture content estimation. The test was therefore repeated for three times and the average moisture content was taken as its plastic limit.

Plasticity Index: The plasticity index quantifies the water contents of a soil where the soil demonstrates plastic properties. The PI denote the arithmetic difference between the liquid limit and the plastic limit ($PI = LL - PL$).

Mathematically, it can be represented as:

$$P.I = L.L - P.L \quad (15)$$

P.I = plasticity index

L.L = liquid limit

P.L = Plastic limit

Linear shrinkage: This test delineates shrink/swell potential of geologic deposits. Shrinkage limit is the water content after just enough water is added to fill all the voids in soil. It is a ratio of the decrease in length to the original length of test sample expressed in percentage.

Equipment: Mould, Oven

Procedure: A sample of 150g from thoroughly mixed portion of bulk material passing 425 micron BS sieve was prepared. The mould was scrupulously cleaned and a thin film of grease was applied to its inner walls. The sample was then mixed with distilled water, until it attains a homogenous state. Soil-water paste was positioned in the mould. The sample was then dried in the oven between 105 to 110°C temperatures of for 24 hours. Cooling of the sample was achieved after a complete drying was attained and the length of the soil bar was then recorded.

$$\text{Linear shrinkage (LS)} = 1 - \left(\frac{\text{Length of oven dried specimen}}{\text{Initial length of specimen}} \right) \quad (16)$$

3.3.4 Compaction test

Purpose: This test was carried out to define the relationship between the moisture content and the dry density of a soil for a specified compact effort. A compact effort is the magnitude of mechanical energy that is subjected to the soil mass. Methods available includes tamping, kneading, vibration, and static load compaction but Proctor impact compaction method (Proctor test) was adopted for this research.

Standard Reference: ASTM D 698 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbs/ft³ (600 KN-m/m³)).

Significance: this is a cost effective method of achieving soil stabilization. This test is used to ascertain if the material has the adequate density in relation to its moisture content suitable to carry/hold a particular structure (engineering structure). Soil density has a direct relationship to soil strength, its stiffness and its resistance to shrinkage or swelling, therefore improving the soil density is indirectly improving the above mentioned properties. For this test the key factors are the maximum dry density and the optimum water content. The optimum water content is the water content that will give rise to highest density for a specified compactive effort. Compacting at water contents other than the optimum water content will result to degradation of the soil strength (the soil becomes weaker, more ductile, and highly susceptible to shrinkage when subjected to water content higher than the optimum content or causing flocculation of soil structure when subjected to water content lower than the optimum water content).

Equipment: Molds, Manual rammer, Extruder, Balance, Drying oven, Mixing pan, Trowel, #4 sieve, Moisture cans, Graduated cylinder, Straight Edge.

Procedure: The air dried sample was sifted using a 4.75mm sieve and 3kg of the sample was collected. The soil was mixed with about 6% of distilled water and put immediately into a cylindrical compaction mould and an extension (collar) was added. The mould has a diameter of 105mm. A hammer which falls freely from a height of 50cm was used to compact the soil in the mould in three (3) layers with every layer given twenty five (25) blows to achieve the proctor density. A calculated attempt was made to ensure that the last compacted layer was not below the collar joint and not more than 1cm above the collar joint. Thereafter the mould extension was formerly removed and the soil sample reduced to assume the same level with the mould. Sample of this soil was then collected for the determination of moisture content while the remnants of the compacted soil was fragmented and mixed together with the remnant of the original soil. An increment of 2% of water was now added and the test repeated until a weight drop of the soil was detected. The results of the tests were represented by a moisture-density curve in which the dried density for each determination was taken as the ordinate and plotted against the corresponding value of the moisture content which was taken as the abscissa. The apex of the curve is designated as the maximum dried density (MDD measured in g/cm^3) and its corresponding value of moisture content is designated as the optimum moisture content OMC measure (in %). The zero air voids for each compaction was determined using the formula:

$$\rho_d = \frac{G_s}{(1+wG_s)} \rho_w \quad (17)$$

Where, ρ_d = maximum dry density

G_s = soil specific gravity

ρ_w = water density

3.3.5 California Bearing Ratio (CBR) ASTM D698

Purpose: This is strength integrity test used to determine the relative strength of a soil material for its use in road and airfield pavements. It is used to calculate the required thickness of the pavement.

Equipments:

- 1) CBR test apparatus (compaction mould and collar spacer disk)
- 2) Loading compression machine with load capacity of at least 10.0001b and penetration rate of 0.05mm
- 3) Two dial gauge (with accuracy of 0.001in)
- 4) Standard compaction hammer
- 5) Miscellaneous equipment (mixing bowl, scales, soaking tank, oven)
- 6) Expansion measuring apparatus

Procedure: A minimum weight of 5.5kg of sample was weighed out and sieved with a 20mm B.S sieve. The sample was poured on a large tray and was added a specific amount of moisture content (about 14% of the total mass of the soil sample). It was then mixed meticulously and left to stay for 24hours, after which the sample was poured on a tray and re-mixed. The sample was divided into three layers, the mould was weighed with its base plate and recorded as MI. The mould was oiled lightly and fitted with the extension collar and then the filter paper was inserted into the mould. And was given 62 blows each, making sure that the blows are distributed evenly on the sample surface. Having compacted the whole layers, the collar was detached and the excess sample was rimmed off. The mould was weighed together with the compacted sample on

the base of the CBR machine, with the collar attached to the mould. And four surcharge weights were added.

The plunger was adjusted to touch the surface of the specimen, the specimen was run at each given penetration and the force was recorded. The bottom of the specimen was then run the same way. 350g of the sample was then collected from the point of penetration of the plunger for moisture content determination.

3.3.6 Specific gravity

Purpose: Specific gravity represents the ratio of the mass of unit volume of soil at a particular temperature to the mass of the same volume of gas-free distilled water at the same temperature. Soil comprises particles of different minerals with different specific gravities. Hence, the specific gravity represents the average value of the specific gravities of the constituent particles that make up the sample.

Standard Reference: ASTM D 854 – Standard using Pycnometer.

Significance: This is useful in understanding the phase connection of the different constituents of a soil (air, water, and solids).

Equipment: Pycnometer jar, Dropper, Mortar and Pestle, Spatula, Oven, Balance, 2 Circular Pans, # 10 Sieve with Pan and Cover.

Procedure: A pycnometer of a known weight (w_1) was filled with 500ml of water and was weighed (w_2), the pycnometer was then emptied and a known weight of a prepared sample ($w_3=50g$) was placed in it with distilled water carefully poured into it until the pycnometer is

filled (to a 500ml mark). The above was now weighted (water + soil sample) represented as w_4 .

The mathematical calculation for estimating the specific gravity is given below:

$$S.G = (W_2 - W_1) / W \quad (18)$$

Where $W = (W_4 - W_1) - (W_3 - W_2)$ (19)

W_1 = 'weight of bottle

W_2 = weight of bottle+ water

W_3 = weight of the soil

W_4 = weight of bottle + soil+ water

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Results

4.1.1 Geophysical results

2-D Horizontal Profiling

A transverse was taken along the NW-SE trend on the road, and the chosen transvers covers the failed, partially stable and stable portion of the road, to reveal near surface geoelectric view of the road. Wenner method was adopted for the near surface constant separation traversing (CST) with $a=3, 6, 9$ and 12m . A total of 5.5km of the road was covered as CST was taken at every 500m (running from Amanchi – Mbosi junction). Surfer 13 software was used to produce the 2-D pseudo resistivity section of the road, this done by using the gridding method then applying krigging with the best linear unbiased estimator. This surfer 13 software allows for the creation of the 2-D resistivity view of the near surface using the already estimated resistivity values. The Table 4.1 below shows the resistivity values at different depth while Fig 4.1 shows the 2-D pseudo-section of the road.

Table 4.1: Location of VES points and their resistivity measurement

S/N	Location	Longitude	Latitude	Elevation	Resistance			
					a=3	a= 6	a =9	a =12
1	Amanachi (former VC's wife house)	05°49.2'	06°58.2'	161	9.1900	9.0774	9.3234	6.8058
2	Amanachi (Opp all saints church)	05°48.6'	06°58.2'	165	5.4317	6.9698	8.4220	7.7256
3	Amaebu	05°49.2'	06°57.6'	168	9.9913	6.4984	5.0485	4.4428
4	Up Jesus block industry	05°49.8'	06°57.0'	161	0.0925	15.325	9.1007	29.404
5	EzeOkoligwe road	05°49.8'	06°57.3'	152	9.4915	6.8353	4.6320	3.6338
6	New generation church	05°49.5'	06°56.4'	150	8.5736	6.8681	6.3387	5.4748
7	Mouka foam shop	05°49.56'	06°56.4'	150	8.6171	7.9814	6.6710	4.9122
8	Afor market Awo-idemili	05°49.8'	06°55.8'	163	6.0556	4.5890	3.9779	3.0919
9	Recreational park Awo	05°49.2'	06°55.68'	152	44.479	20.878	12.443	10.541
10	Isseke	05°49.8'	06°54.78'	151	50.521	26.816	15.680	10.579
11	Mbosi junction	05°49.2'	06°53.4'	150	15.250	10.942	8.8450	8.5570

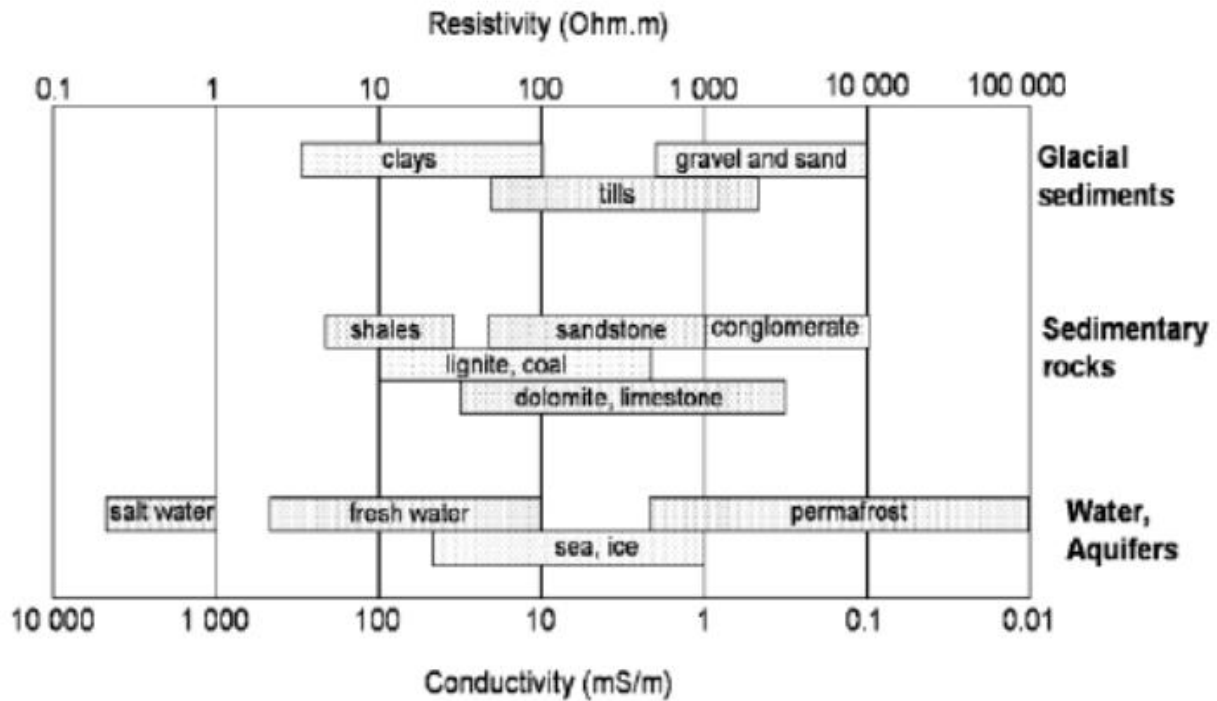


Fig4.1: Typical resistivity and conductivity values for geo-materials (Samouelian et al., 2005)

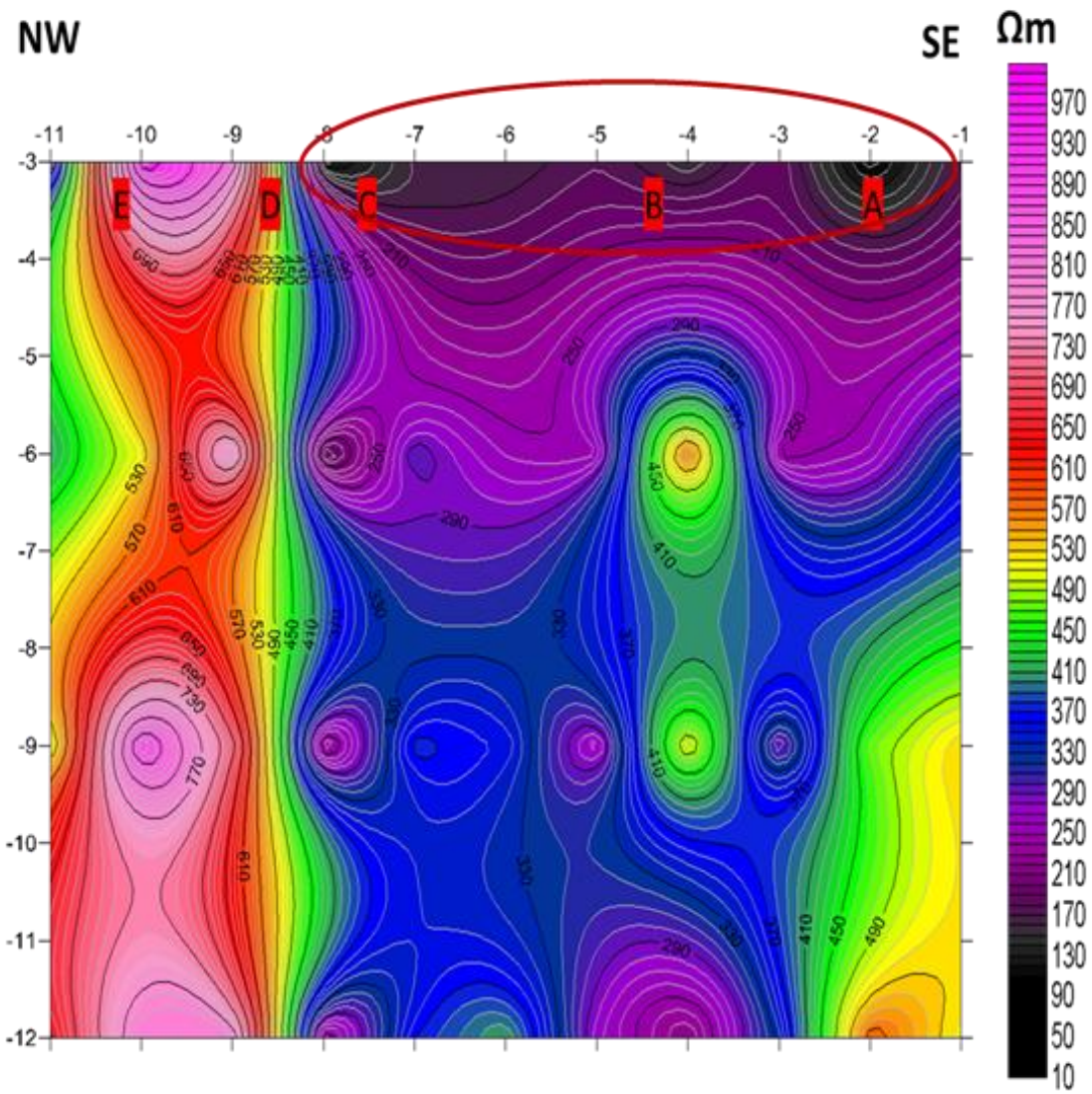


Fig 4.2: 2-D pseudo-resistivity section of Orsu-Ihiala road

Table 4.2: Rating of subsoil competence using resistivity values

Soil resistivity (Ωm)	Lithology	Competence rating
<100	Clay	Incompetent
100-350	Sandy clay	Moderately competent
350-750	Clayey sand	Competent
>750	Sand/Laterite/Crystalline rock	Highly competent

Analysis of the above Fig 4.2 shows that the studied depth is at 12m beneath the surface and the area consist of vary resistivities values at constant depth and this is an indication that the geo-material is unevenly distributed. It resistivities ranges from $10\Omega\text{m}$ - $970\Omega\text{m}$ corresponding to clay and clayey sand respectively (see Fig 4.2 and 4.3).

It could be observed at the near surface (0-4m) the resistivity at the Northwestern part of the road is relatively low compared to that at the high resistivity values observed at Southeastern end and this correspond to the failed and stable portion of the road. The area highlighted with red (A, B and C) represent the low resistivity (less than $130\Omega\text{m}$) materials and are foundon the SE are interpreted to be clay deposit and it can be observed that this clay deposit diminishes towards the Northwestern part of the road as a result of possible transitionfrom the clay rich Ameke Formation to the Sand rich recent Meander belt. It should be important to note that this recent Meander belt is a product of the weathered and eroded Benin formation coming from the upland to the downland. Within the failed section of the road it is worthy to note that the most failed portion of the road is found at Awo-Idemili and this represent the shallowest elevation (151m). Water through run-off is most likely to end in this section and this could mean intense volume change in clay, with a resultant effect of rapid swelling and shrinkage when water enter or leaves the clay (water variation during in the clay during wet and dry season).

Six (6) VES was carried out at the zone highlighted in red (zone of incompetent geomaterial). This was done to revalidate the classification of the geomaterials as incompetent and to further see deeper into the subsurface.

Vertical electrical sounding results

6VES was carried out at Zone A, B and C which coincides with the failed portion of the road, in order to further define the geomaterials in the deeper depths ($AB/2 = 55\text{m}$). The result of the VES was interpreted using Zohdy and Interplex1D software and guided by the simple analysis of field data. The characteristics of the VES curve were determined through the shape of the curve. By implication both the features of each layer and their maximum depth of penetration was determined by observing the VES curves. The observed curve type varies from HA- AH- AA with HA observed in VES 1 and 2, AH observed in VES 3 and 4 AH and AA observed in VES 5 and 6 (see Fig 4.3-4.8).

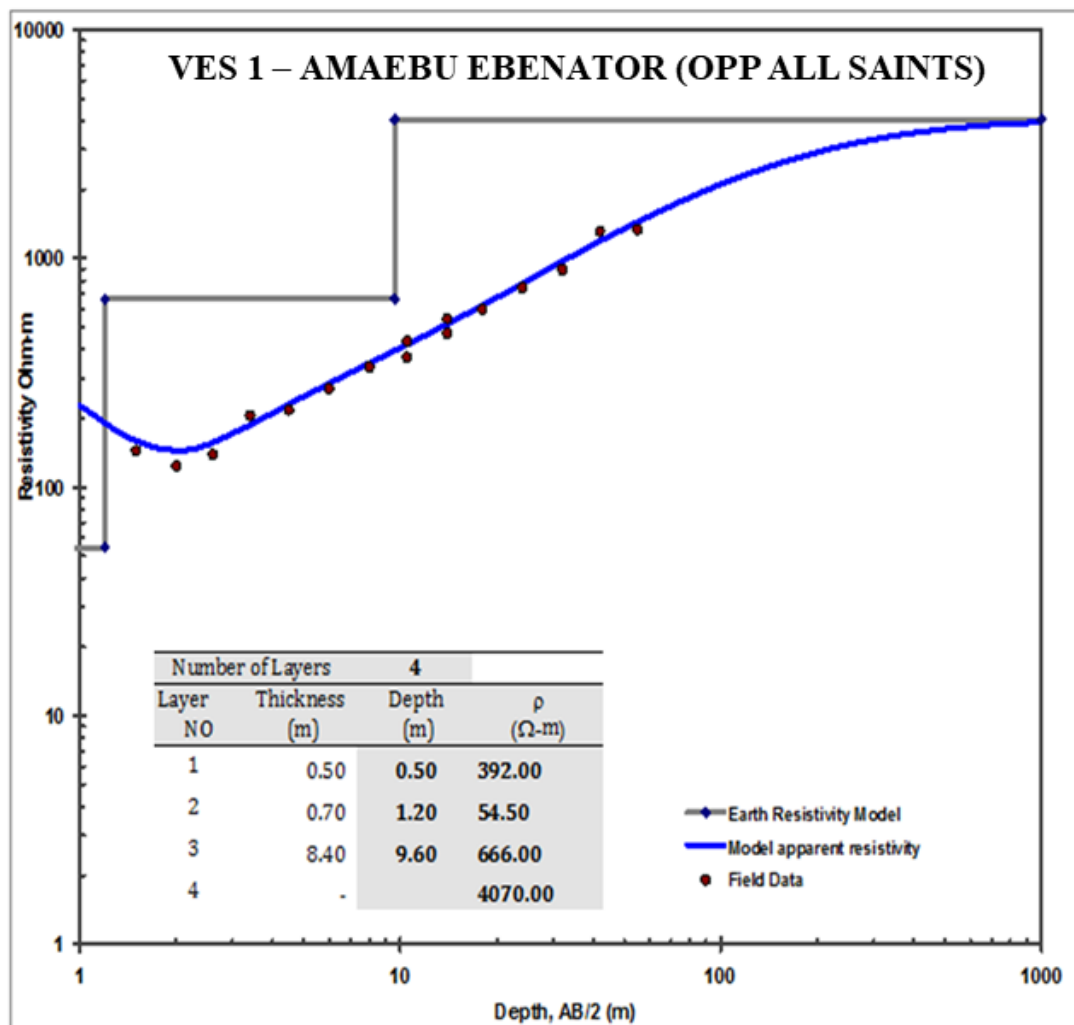


Fig 4.3: Graph curve of VES R1

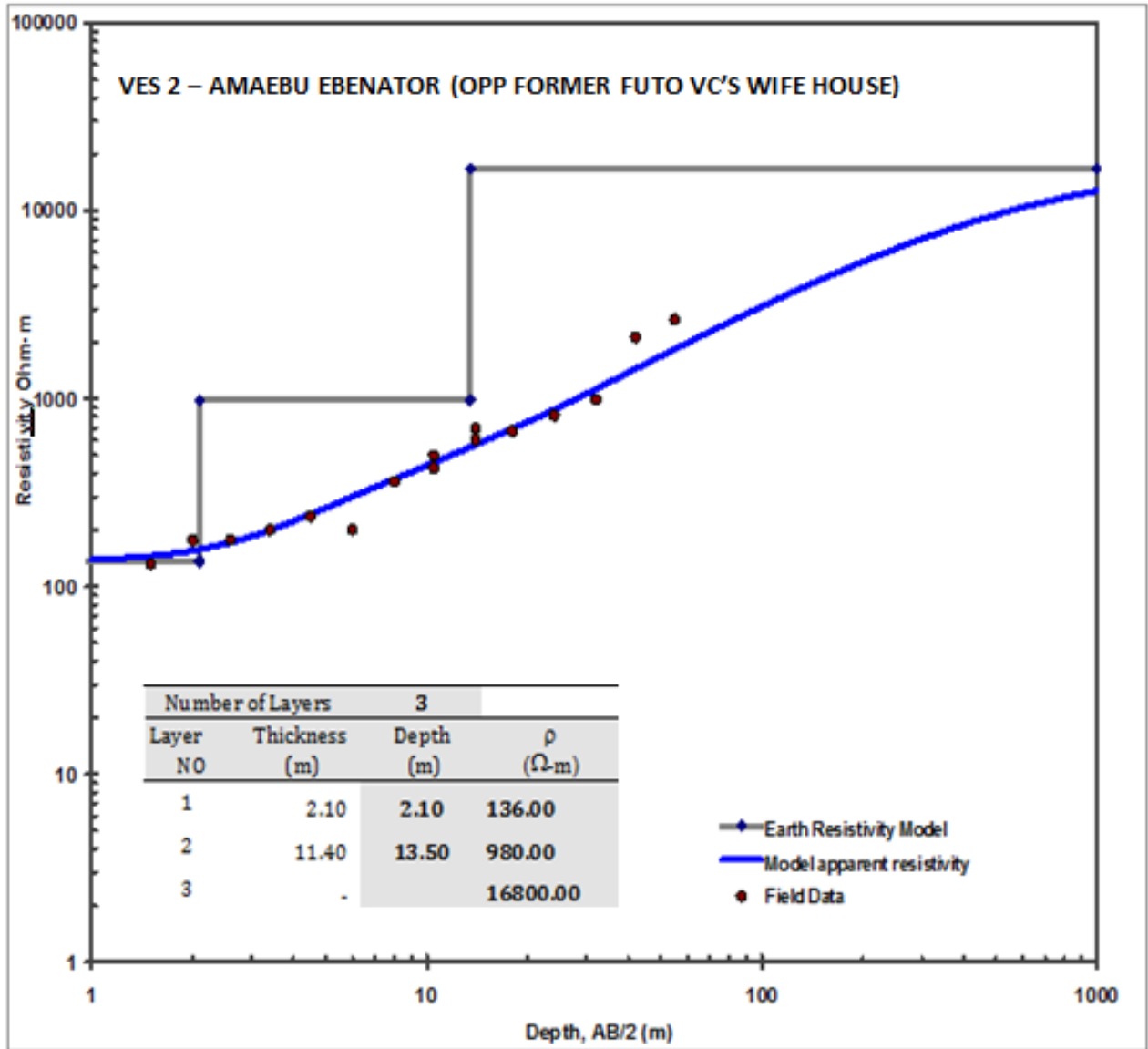


Fig 4.4: Graph curve of VES R2

VES 1 and 2 was carried out Amaebu just 200m away from each other (see Table 4.1), analysis of curve shows that both VES 1 is of curve HA while VES of curve type indicating from high-low-high resistivity trend typical of the $\rho_1 > \rho_2 < \rho_3 < \rho_4$ for VES I and $\rho_1 < \rho_2 < \rho_3 < \rho_4$. The resistivity of VES1 and 2 ranges from 54.5 Ω m - 16800 Ω m corresponding to clay and sand respectively. Three to four geoelectric layers was defined as clayey top soil, clay, clayey sand and sand. VES 1 showed a succession of Sand, clay and clayey top soil (from bottom to top). Top soil with resistivity of 392 Ω m is seen to be seated at a depth of 0.5m and has a thickness of 0.5m, while the second layer (clay) is seen at depth 1.2m with resistivity of 54.5 Ω m and thickness of 0.7m. The third layer (clayey sand) is seen at depth 8.4m with resistivity of 666 Ω m and thickness of 9.6 Ω m. The fourth layer was interpreted as sand with resistivity of 4070 Ω m (see Fig 4.5).

VES 2 was interpreted to be made of three layers on the succession of sand, clayey sand and clay topsoil (from bottom to top). The topsoil was observed at 2.10m with resistivity and thickness of 136 Ω m and 2.10m respectively. Second layer was interpreted as clayey sand with resistivity, depth and thickness of 980 Ω m, 13.5m and 11.4m respectively. Third layer was interpreted as dry sand with resistivity of 16800 Ω m.

It is obvious that layer1/2 for VES 1 and layer 1 for 2 VES which are below 2.2m are made of incompetent geology material defined as clay and this depth coincide with the depth of subgrade materials

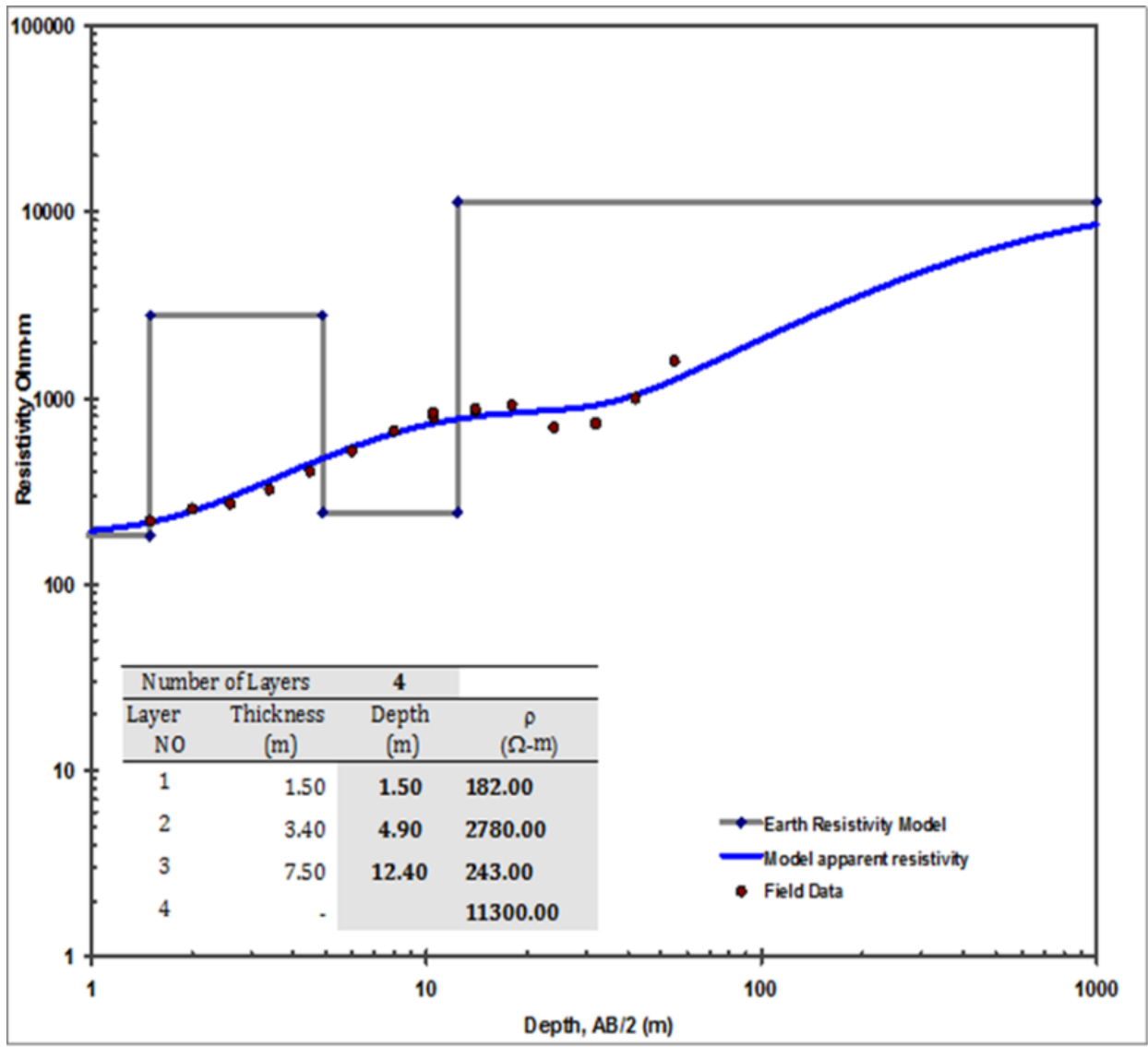


Fig 4.5: Graph curve of VES R3

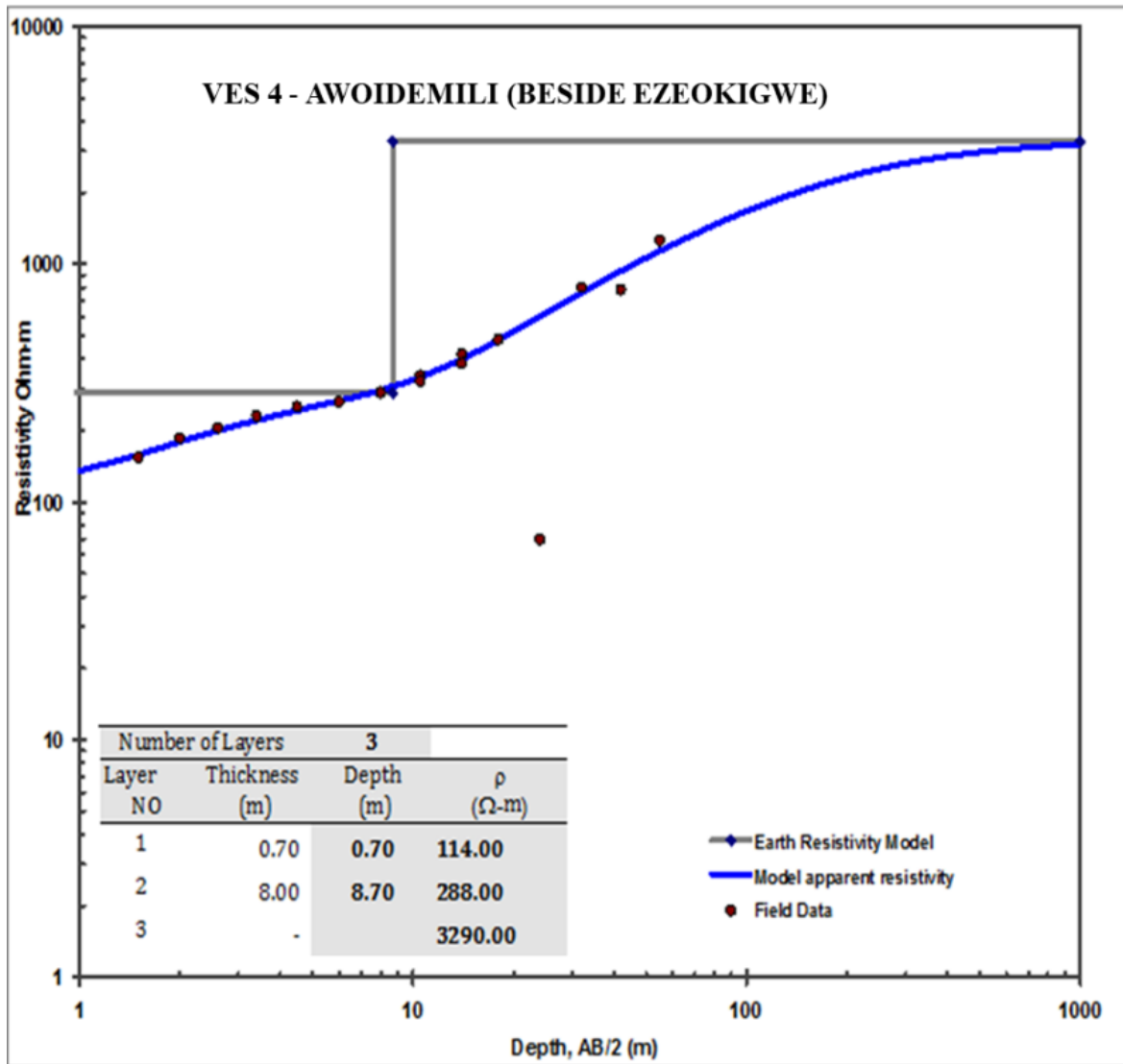


Fig 4.6 :Graph curve of VES R4

VES 3 and 4 was carried out Awo-Idemili with just 200m away from each other, analysis of curve shows that both VES 3 and 4 shows KA curve type trend, typical of the $\rho_1 < \rho_2 < \rho_3 < \rho_4$. The resistivity of VES3 and 4 ranges from 114 Ω m - 11300 Ω m corresponding to clay topsoil and sand respectively. Three to four geoelectric layers was defined as clayey top soil, clay, clayey sand and sand. VES 3 showed a succession of Sand, clayey sand, clay and clayey top soil (from bottom to top). Top soil with resistivity of 182 Ω m is seen to be seated at a depth of 1.5m and has a thickness of 1.5m, while the second layer (water filled clay) is seen at depth 4.9m with resistivity of 243 Ω m and thickness of 3.4m. The third layer (clayey sand) is seen at depth 12.4m with resistivity of 2780 Ω m and thickness of 7.5m. The fourth layer was interpreted as sand with resistivity of 11300 Ω m (see Fig16). VES 4 has three layers interpreted as topsoil, clayey sand and sand. First layer is topsoil found at 0.7m with resistivity and thickness of 114 Ω m and 0.7m respectively. The second layer was interpreted as clayey sand at depth 8.7m with resistivity and thickness of 2.88 Ω m and 8m respectively. The layer was interpreted as sand with resistivity of 3290 Ω m. Both VES 3 and 4 showed that at the subgrade level (less than 2m below) resistivity of earth materials is 200 Ω m.

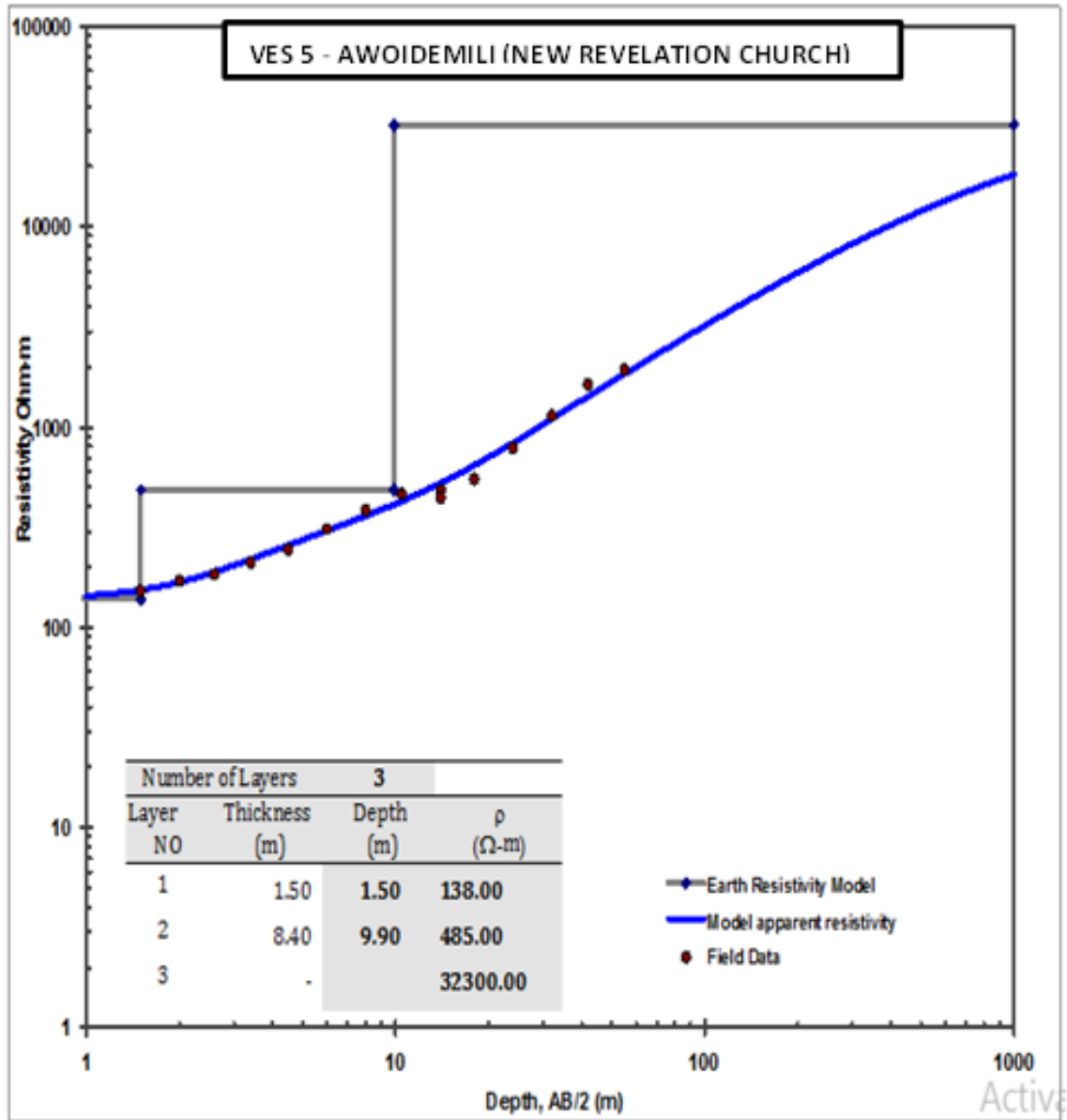


Fig 4.7 :Graph curve of VES R5

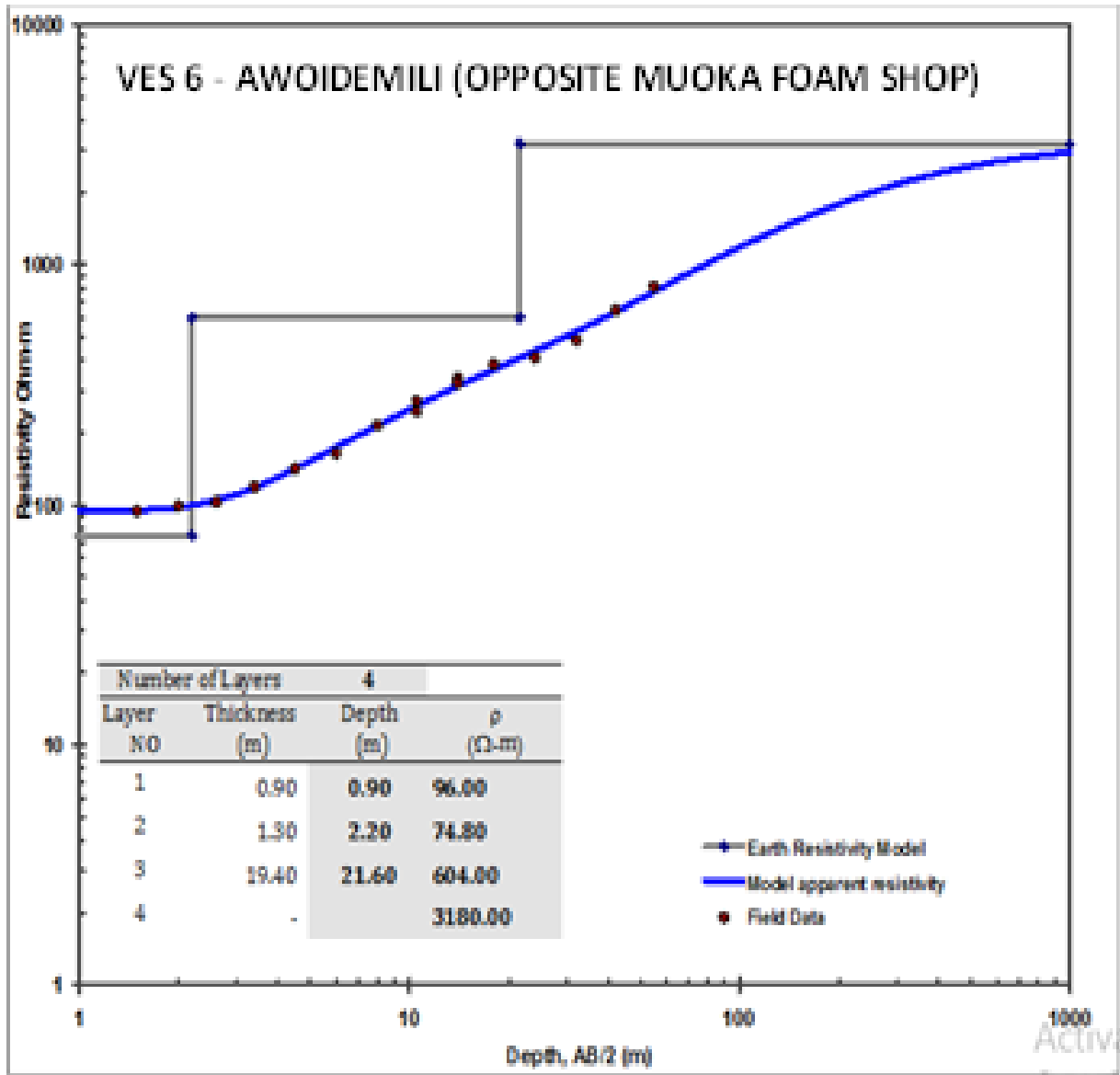


Fig 4.8: Graph curve of VES R6

VES 5 and 6 was carried out Awo-Idemili (Amagriget) just 200m away from each other, analysis of curve shows that both VES 5 and 6 is of curve AA. The resistivity of VES5 and 6 ranges from $96\Omega\text{m}$ - $323000\Omega\text{m}$ corresponding to clay and dry sand respectively. Three to four geoelectric layers was defined as clayey top soil, clay, clayey sand and sand. VES 5 showed a succession of Sand, clayey sand and top soil (from bottom to top). Top soil with resistivity of $138\Omega\text{m}$ is seen to be seated at a depth of 1.5m and has a thickness of 1.5m, while the second layer (clayey sand) is seen at depth 9.9m with resistivity of $485\Omega\text{m}$ and thickness of 8.4m. The third layer defined as dry sand is seen at depth 8.4m with resistivity of $32300\Omega\text{m}$.

VES 6 was interpreted to be made of four layers on the succession of sand, clayey sand and clay topsoil (from bottom to top). The topsoil was observed at 0.9m with resistivity and thickness of $96\Omega\text{m}$ and 0.9m respectively. Second layer was interpreted as clayey sand with resistivity, depth and thickness of $171\Omega\text{m}$, 2.2m and 1.3m respectively. Third layer was observed at 21.6m, interpreted as clayey sand with resistivity and thickness of $604\Omega\text{m}$ and 19.4m respectively. A summary of the 6VES is given in Table 4.3 below.

It is obvious that at near surface (below 2m) the resistivity of the earth material is poor indicating poor earth material.

Table 4.3: Summary of the geo-electric parameter

S/N	Location	Longitude	Latitude	Elevation	VES R1				
					LAYERS NO.	Thickness (m)	DEPTH(m)	Probably lithology	Resistivity (Ω m)
1	AmaegbuEbenator(opp.ALL SAINTS)	N5°48' 36''	E6°58' 12''	165ft	1	0.50	0.5	Clayey top soil	392
					2	0.70	1.2	Clay	54.50
					3	8.40	9.6	Clayey sand	666.00
					4	>8.40	>9.6	Sand	4070.00
					VES R2				
2	Amaegbuebenator(off Vc's wife's House)	N5°48' 36''	E6°58' 12''	161ft	1	2.10	2.10	Clayey top soil	136.00
					2	11.40	13.50	Clayey sand	980.00
					3	>11.40	>13.50	Dry sand/sandstone	16800.00
					VES R3				
3	Awoidemili(opp.upjesus block industry)	N5°48' 36''	E6°58' 12''	161ft	1	1.50	0-1.50	Clayey top soil	182.0
					2	3.40	1.50-4.90	Sand	2780.0
					3	7.50	4.90— 12.40	Clay	243.0
					4	>7.50	>12.40	Sand/dry sand	11300.0
					VES R4				
4	Awoidemili (Beside EzeOkoligwe road)	N5°49' 8''	E6°57' 0''	152ft	1	0.70	0.70	Clayey top soil	114.0
					2	8.00	8.70	Clay	288.0
					3	>8.00	>8.70	Sand	3290.0
5	Awoidemili (New Relevation church)	N5°49' 5''	E6°56' 4''	150ft	VES R5				
					1	1.50	1.5	Clayey top soil	138.00
					2	8.40	8.70	Clay	485.00
					3	>8.40	>8.70	Sand/dry sand	32300.00
6	Awoidemili(off Mouka Foam)	N5°49' 56''	E6°56' 4''	150ft	VES R6				
					1	0.90	0.90	Clayey top soil	96.00
					2	1.30	2.20	Clay	74.80
					3	19.40	21.60	Clayey sand	604.00
					4	>19.40	>21.60	Sand	3180.00

VES Goelectric correlation

The 6 VES were horizontally stacked together and correlated, this was done in a bid to define soil distribution along the studied section. VES was stacked side by side running from the SE – NW. Maximum penetration depth is 37m, this will allow us see the distribution of soil type of this depth. Fig 4.9 below shows the correlated geoelectric layers.

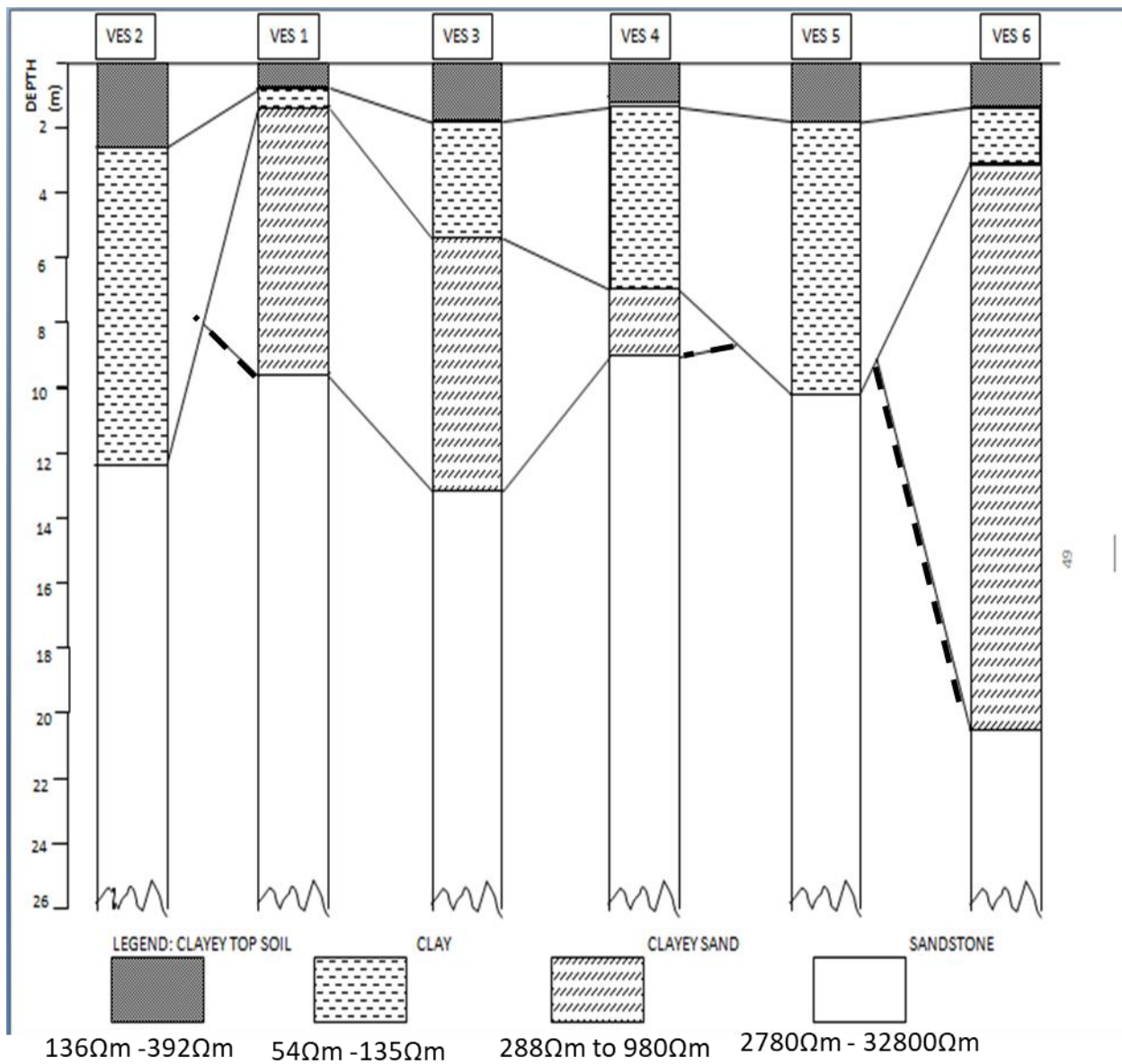


Fig 4.9: Geo-electric section of the Study Area

The VES shows that the studied road has between 3-4 layers interpreted as topsoil, clay, clayey sand and sand. The resistivity for topsoil was observed to be 136 Ω m -392 Ω m, clay resistivity varies from 54 Ω m -135 Ω m, clayey sand ranges from 288 Ω m to 980 Ω m and sand ranges from 2780 Ω m - 32800 Ω m. the geoelectric correlation showed a fair-good correlations of the soil type distribution, dotted lines were used to mark doubtful contacts. At an average of 2m below the surface a topsoil-clay was observed across the VES points, indicating that the near surface (Subgrade) is of poor quality. And it could also be observed that at an average of 12m below, the quality of the soil improved (sand with resistivity above 2700 Ω m). This is possible indication that the road is resting on a fairly- poor subgrade material.

4.1.2 Geotechnical results

Geotechnical results of three samples are presented below. The samples were collected between 1-1.5m beneath the earth surface at zone A, B and C which coincides with the failed portion of the roads. Samples were analyzed at Arab contractors, Alakwo Imo state. Result of analysis are presented below.

Sieve analysis

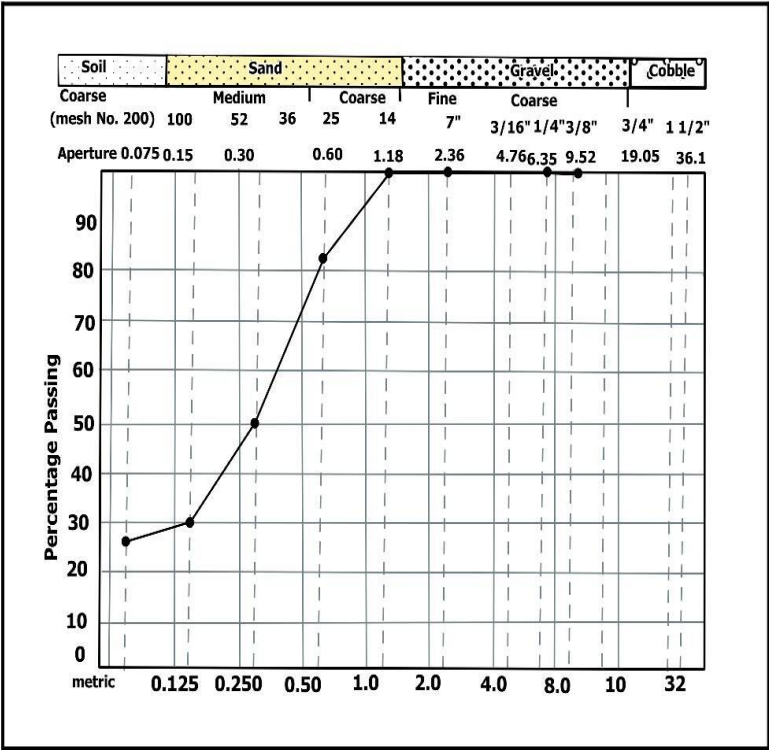


Fig 4.10: Graph of Particle Size Analysis of sample S01

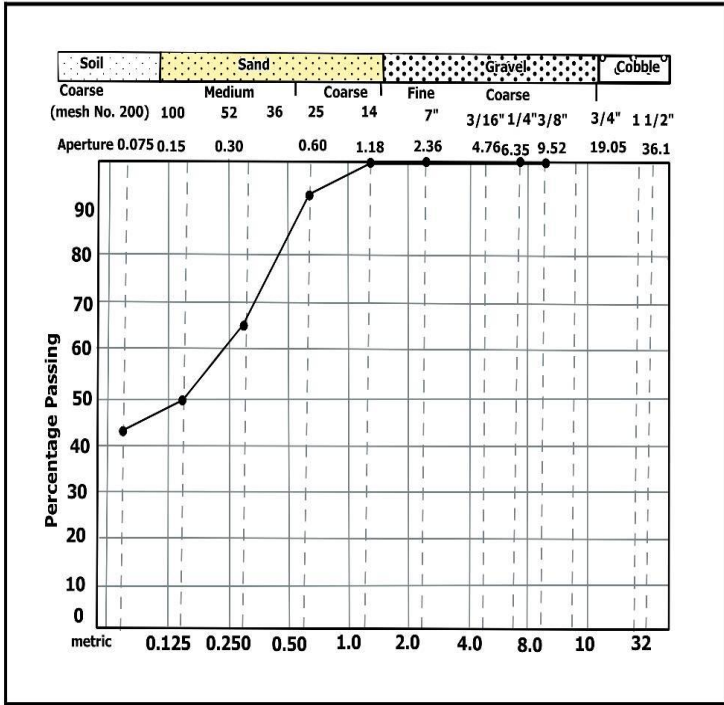


Fig 4.11: Graph of Particle Size Analysis of Sample S02

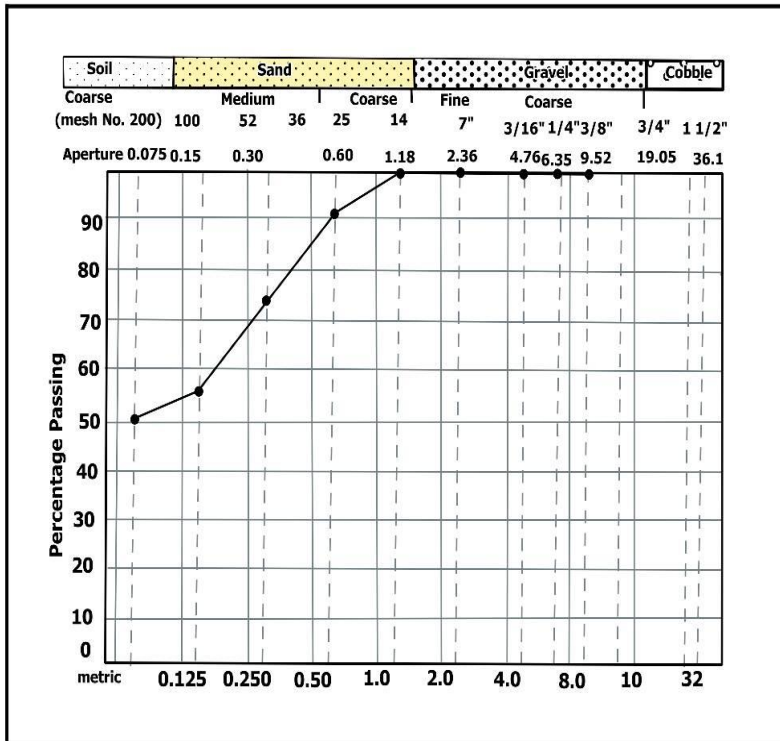


Fig 4.12: Graph of Particle Size Analysis of Sample S03

Table 4.4: AASTHO classification of soil

AASHTO Soil Classification System (from AASHTO M 145 or ASTM D3282)											
General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max
0.425 (No. 40)	30 max	50 max	51 min
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min	41 min
Plasticity Index	6 max	...	N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min ¹
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

Note (1): Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30

Sieve analysis describes the distribution of soil grain sizes and this is an important analysis used to classify soil based on their grain size. The strength of a soil depends on so many factors and grain size is one of the most important factors. Results of the three samples indicates that grain sizes varies from clay-silt-sand-gravel at different proportion. Sample S01 contains 1.93% of gravel, 47.5% of sands, 24.5% of slit and 26% of clay. Sample S01 according to AASTHO M145 classification is classified as granular material belonging to group class A-2 with 26% passing 0.75mm diameter (see Table 4.4). Sample S02 contains 0.375% of gravel, 36.18 % of sand, 22.3% of silt and 41.15% of clay.

S02 is therefore according to AASTHO M145 classified as highly compressible silty-clay material belonging to group A-7 with 41.15% passing of sieve diameter 0.775mm. Sample S03 contains 0.29% of gravel, 29.21 % of sand, 20.8% of silt and 49.7% of clay. According to

ASTHO M145 S03 is classified as highly compressible silty-clay material belonging to group A-7-2 with 49.7% passing of sieve diameter 0.775mm (see Fig 4.13).

Atterberg limit

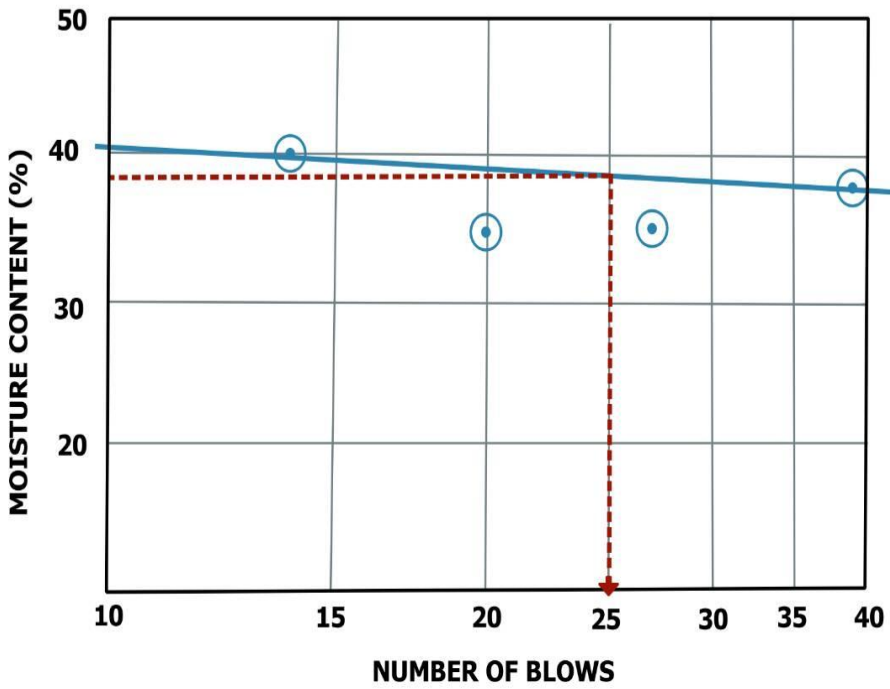


Fig 4.13: Atterberg Limit for Sample S01

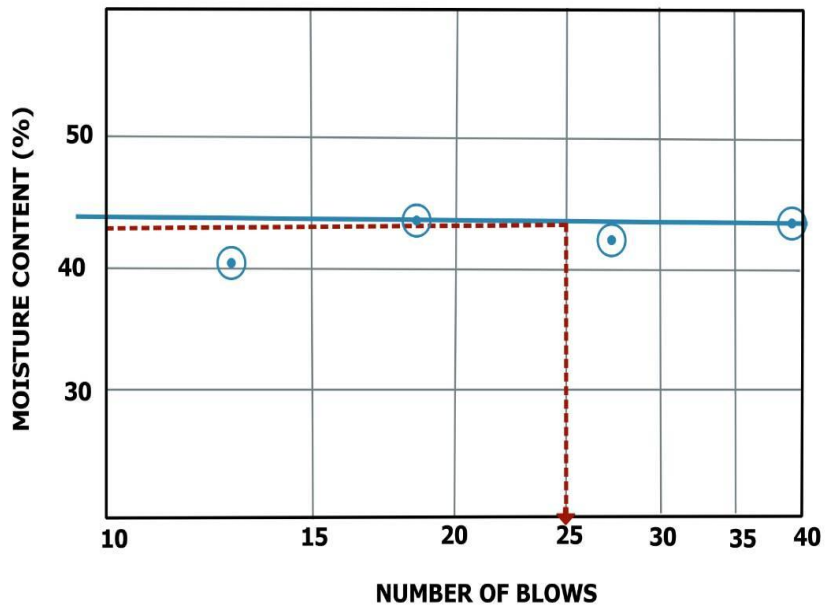


Fig 4.14: Atterberg Limit for Sample S02

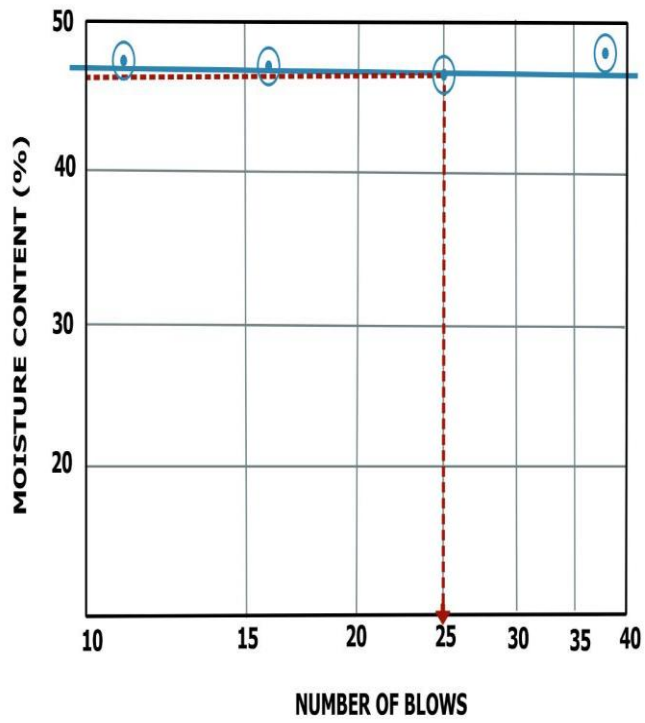


Fig 4.15: Atterberg Limit for Sample S03

Atterberg Limit includes: Plastic limit, liquid limit and shrinkage limit. This test is used to define the plasticity of the soil (the critical water content that will allow the soil to behave in a semi-solid, plastic, liquid or viscous manner). This test was conducted in accordance with AASTHO ASTM D4381 standards. Summary of the plastic limit for the investigated samples are shown in Table below.

Table 4.5: Results of Atterberg Limit

Atterberg Limit	S01	S02	S03	FMW,1997 requirement for Subgrade materials
Shrinkage Limit	13.6%	6.4%	11.1%	<8%
Liquid Limit	38.5%	42.5%	46.9%	<= 50%
Plastic Limit	21.6%	35.4%	31.1%	-----
Plastic index	16.9%	7.1%	14.9%	<= 30%

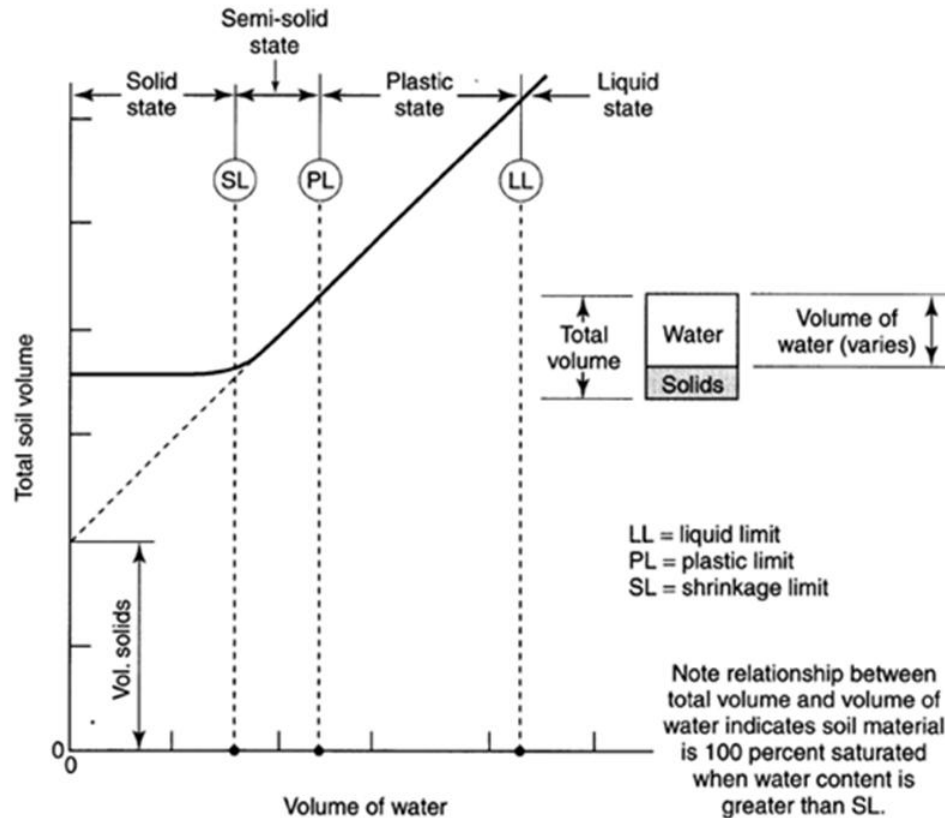


Fig 4.16: Variation of total soil volume and consistency with change in water content for a fine-grained soil (from McCarthy and David 2006).

The Atterberg Limit result revealed that the Plasticity index (a computation of corresponding Liquid Limits (LL) and Plastic Limit (PL)) are relatively low (16.9%, 7.1% and 14.9% for S01, S02 and S03 respectively). Several standards exist for defining the swell potential of plasticity of fine soils. The results of the samples were compared with some of the standards (Table 6 -8) in order to be able to properly define their plasticity. The LL for S01, S02 and S03 are 38.5%, 42.5% and 46% respectively and according to Bell, 2007 are referred to as intermediate plasticity while their swell potential are classified as medium according to Holts and Gibbs 1956. Their plastic index observed to be relatively low (16.9%, 7.1% and 14.9% for S01, S02 and S03 respectively) Murthy used plastic index in classifying plasticity and according to Murthy, it is

classified as medium plasticity (see Table 4.6). While when compared to Sower, 1979 S02 and S03 is classified to be slightly plastic but S01 is classified to be medium plastic (see Table 4.6).

Table 4,6: Classification of plasticity base on plastic index (Murthy 2009)

PI (%)	Plasticity
0	Non plastic
< 7	Low plastic
7-17	Medium plastic
>17	Highly plastic

Table 4.7: classification of soil plasticity base in plastic index (Sower, 1979)

Plasticity Index	Classification	Dry Strength	Visual-Manual Identification of Dry Sample
0 - 3	Nonplastic	Very low	Falls apart easily
3 - 15	Slightly plastic	Slight	Easily crushed with fingers
15 - 30	Medium plastic	Medium	Difficult to crush with fingers
> 30	Highly plastic	High	Impossible to crush with fingers

Table 4.8: The relationship between LL, SL, plasticity and potential of swell (Holts & Gibbs 1956)

Liquid limit	Shrinkage limit (SL)	Plasticity	Classification of potential swell	Description
<35	>15	Low plasticity	Low	Lean or <u>silty</u>
35 – 50	10 – 15	Intermediate plasticity	Medium	Intermediate
50 – 70	7 – 12	High plasticity	High	Fat
70 – 90	<11	Very high plasticity	Very high	Very fat
>90		Extra high plasticity		Extra fat

Table 4.9. :Relationship between the LL, PL and the clay mineral (after Carter and Bentley, 1991)

Clay minerals	Ca ⁺		Na ⁺	
	LL	PL	LL	PL
Montmorillonite	123 – 177	65 – 79	280 – 700	86 – 97
Illite	69 – 100	36 – 42	61 – 75	34 – 41
Kaolinite	34 – 73	26 – 36	29 – 52	26 – 28

The results when compared to Carter and Bentley, 1991 (see table 4.9) indicates suspicion of possible presence of sodium rich kaoline mineral as the plasticity of soil is influenced by the volume of its clay fraction and the type of clay minerals present (Arora, 2008). The plasticity and liquid limit decreases in the following order

Montmorillonite > Chlorite > Illite > Kaolinite

The intermediate plasticity characteristics of the soils indicate intermediate susceptibility to loss of strength upon saturation and are classified as fair subgrade material according to the FMW, 1997 specification for Roads and Bridges.

Compaction test

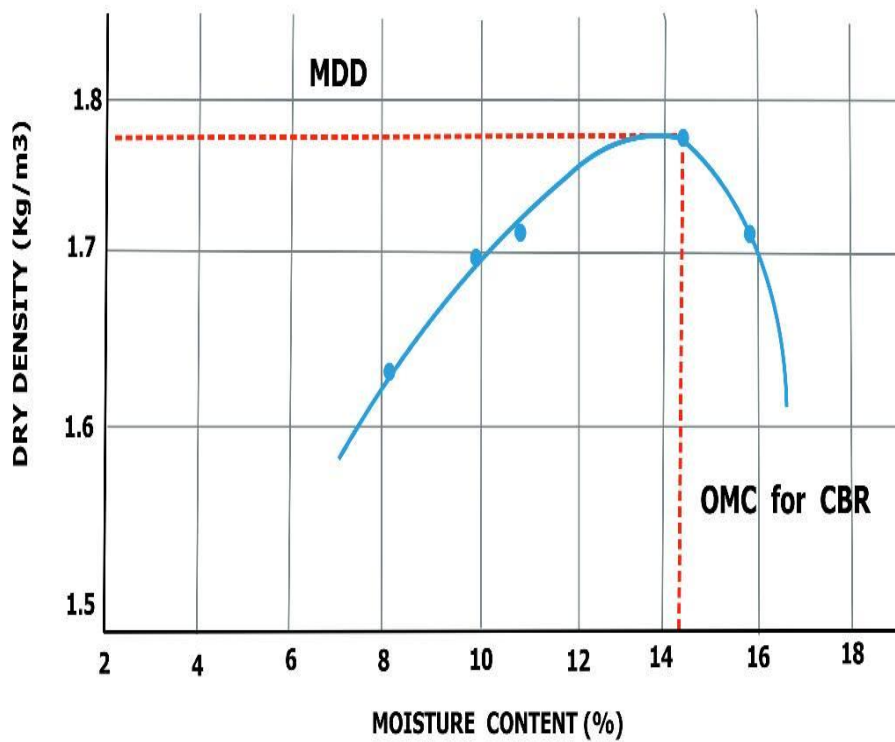


Fig 4.17: Graph of Compaction Test for Sample S01

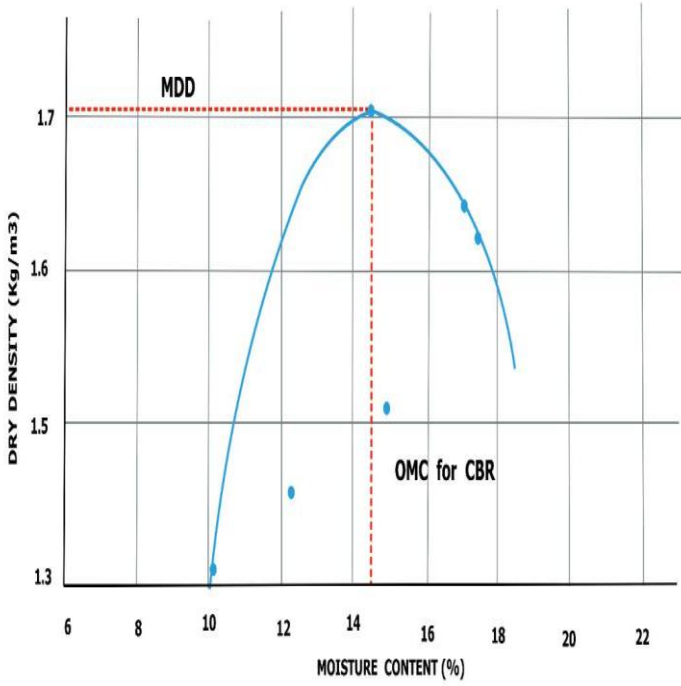


Fig 4.18: Graph of Compaction Test for Sample S02

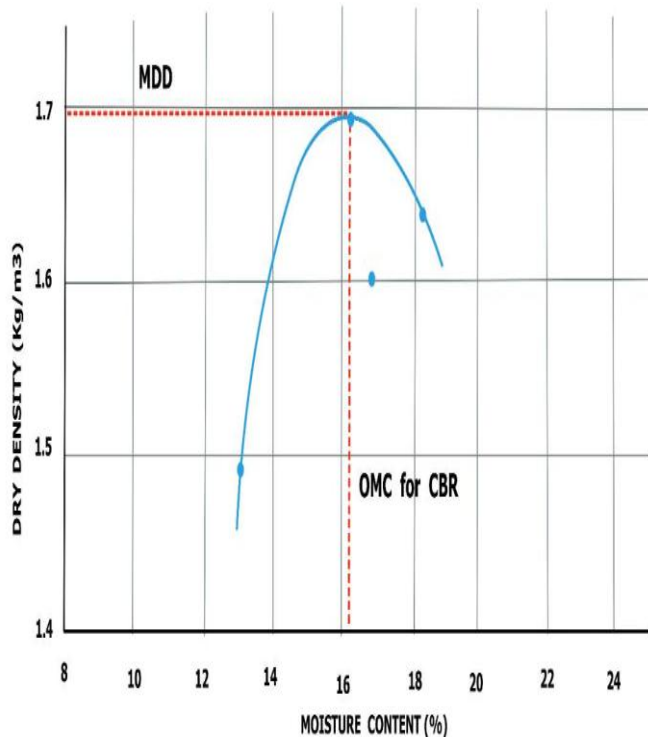


Fig 4.19: Graph of Compaction Test for Sample S03

The test was carried out to establish the relationship between moisture content and the dry density and ultimately produce a cost effective way of carrying out soil stabilization. The test was conducted according to the ASTM D698 standard. The expected value for dry density varies with the type of soil being compacted and also with its water content. For any compactive effort, the dry density of a soil will vary with its water content. A soil compacted dry will reach a certain dry density. If compacted again with the same compactive effort, but this time with water in the soil, the dry density will be higher, since the water lubricates the grains and allows them to slide into a denser structure. Air is forced out of the soil, leaving more space for the soil solids, as well as the added water. With even higher water content, a still greater dry density may be reached since more air is expelled. However, when most of the air in the mixture has been removed, adding more water to the mixture before compaction results in a lower dry density, as the extra water merely takes the place of some of the soil solids. The density and water content at this point is known as maximum dry density (MDD) and optimum water content (OMC).

Sample S01 gave an MDD and OMC of 1.776g/cm^3 and 14.4% for S01, 1.713g/cm^3 and 14.6% for S02 and 1.692g/cm^3 and 16.4% for S03. It was noticed that the MDD decreases and OMC increases along NW profile of the road which conforms with the result of the sieve analysis that also show clayness to reduce along the same profile (this is because clay has high tendency of retaining water and hence would have low dry density). This is an indication that the soil is generally loose and is poor as a subgrade material, with the looseness decreasing NW along the road. So there is need to stabilize or further compact the soil if it must be used as a subgrade material. Since the strength of a soil increases it is when compacted (Bell, 2007).

California bearing ratio

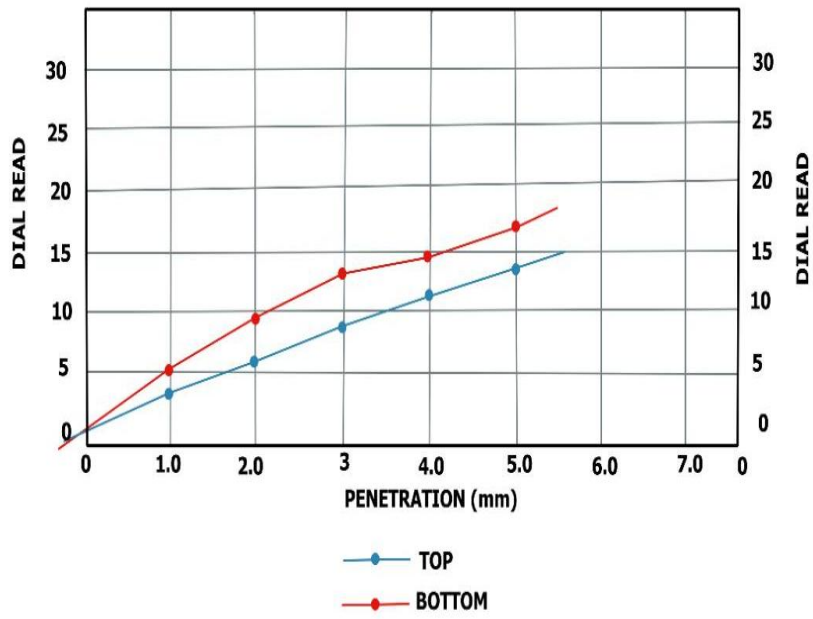


Fig 4.20: Graph of California Bearing Ratio for Sample S01

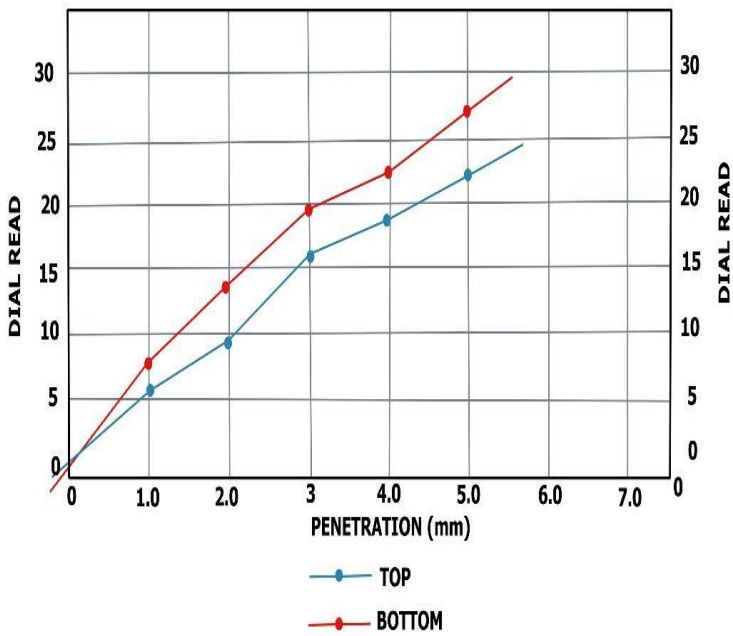


Fig 4.21: Graph of California Bearing Ratio for Sample S02

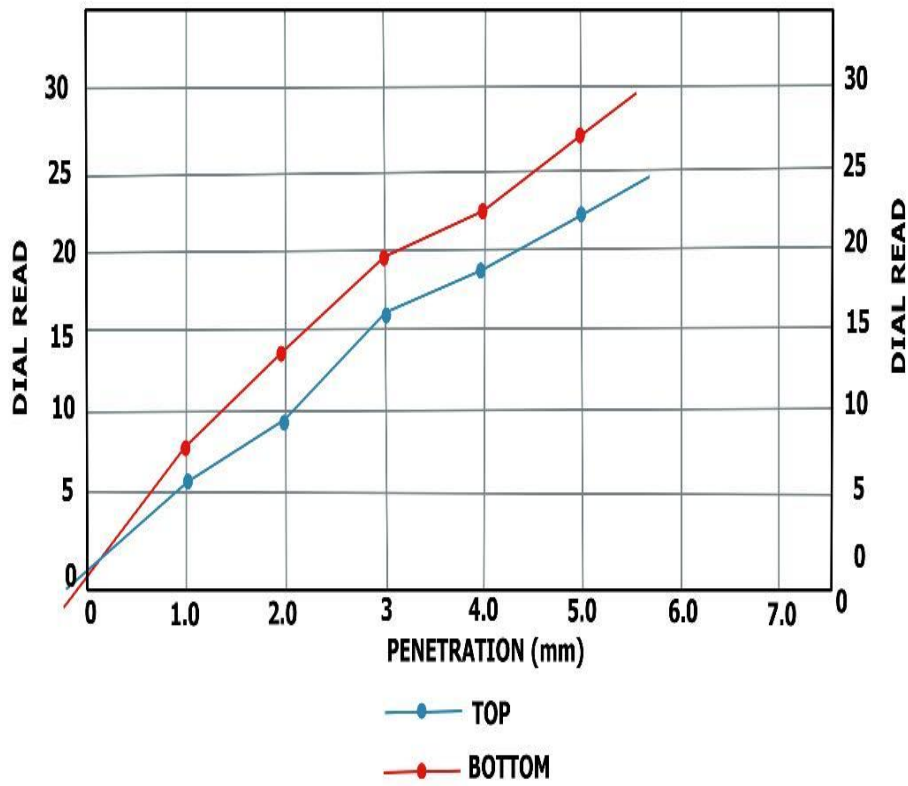


Fig 4.22: Graph of California Bearing Ratio for Sample S03

The CBR test is a simple strength test that compares the bearing capacity of a material with that of a well-graded crushed stone (thus, a high-quality crushed stone material should have a CBR of 100%). It is primarily intended for, but not limited to, evaluating the strength of cohesive materials having maximum particle sizes less than 0.75 inches. CBR use for the determination of soil strength and for designing pavement thickness. The soaked CBR was conducted according to ASTM D689.

The California Bearing Ratio (soaked) for sample S01, S02 and S03 is 2.1%, 3.4% and 16.2% respectively (see Fig). Sample S01 and S02 showed a CBR value is below the FMW, 1997 standard for roads and bridges while sample S03 showed a promising CBR value when compared to FMW, 1997 standard. It can be noted that the soaked CBR increases NW along the Orlsu-Ihiala road. The low CBR of sample S01 (even though the soil is classified as granular material), is an evident that the soil is highly porous and with moisture influx it will be highly damaging to the sub-grades of pavements constructed on them.

Table 4.10: Summary of geotechnical results

S/N	Parameter tested	Test result			Specification
		S01	S02	S03	required limit
					Subgrade material
1	Sieve analysis (% passing 200)	26.0%	42.2%	49.7%	< or = 35%
2	Atterberg Limit				
	Liquid limit	38.5 %	42.5 %	46.9%	< or = 50%
	Plastic limit	21.6%	35.4%	31.1%	-----
	Plasticity index	16.9%	7.1%	14.9%	< or = 30%
3	Compaction Test				
	Optimum water Content (OMC)	14.4%	14.6%	16.4%	-----
	Maximum Dry Density (MDD)	1.776g/cm ²	1.713g/cm ²	1.692g/cm ²	-----
4	California Bearing Ratio (CBR)	2.1%	3.4%	16.2%	>5%
5	Linear shrinkage	13.6%	6.4%	11.1%	>8%
6	Specific gravity	2.407 g/cm ³	2.586 g/cm ³	2.413g/cm ³	-----
7	Natural Moisture	18.2%	24.2%	25.4%	-----

4.2 Discussion

4.2.1 Geoelectrical

The resistivity of earth material has a relationship with the strength of the material. Earth materials having low resistivity generally indicate poor strength. It could be observed at the near surface (0-4m) the resistivity at the Northwestern part of the road is relatively low compared to that at the high resistivity values observed at Southeastern end and this correspond to the failed

and stable portion of the road. The area highlighted with red (A, B and C) represent the low resistivity (less than $130\Omega\text{m}$) materials and are found on the SE are interpreted to be clay deposit and it can be observed that this clay deposit diminishes towards the Northwestern part of the road as a result of possible transition from the clay rich Ameke Formation to the Sand rich recent Meander belt. It should be important to note that this recent Meander belt is a product of the weathered and eroded Benin Formation coming from the upland to the downland. Within the failed section of the road it is worthy to note that the most failed portion of the road is found at Awo-Idemili and this represent the shallowest elevation (151m). Water through run-off is most likely to end in this section and this could mean intense volume change in clay, with a resultant effect of rapid swelling and shrinkage when water enter or leaves the clay (water variation during in the clay during wet and dry season).

The department of Geotechnics Federal Ministry of Works Nigeria (FMW, 1997) has always stressed the nature of the subgrade materials to be the root causes of major premature road failure in Nigeria and this could be the case for this Orsul-Ihiala road. It is worthy to note that the zone represented by D and E represent the fairly stable and stable part of the road with resistivity values of $430\Omega\text{m}$ and $900\Omega\text{m}$ respectively (interpreted as clayey sand).

Several researchers (Oseji., 2005;and Onu., 1995) in the bid to carry out hydrogeological investigation within the study area all pointed out that the near surface materials is of low resistivity (generally within $48\Omega\text{m}$ - $200\Omega\text{m}$ at less 8m below the surface) with varying thicknesses. This research is therefore in line with previous studies as regards to the geomaterial of the subsurface. Though previous studies failed to investigate the strength of this poor near surface materials, but they pointed out that the material is most likely to be incompetent (Apakama, 2010 and Mgbeojedo, et al., 2018). Further deeper into the subsurface we notice a

gradual increase of resistivity values, indicating fair geomaterial strength. Hence the near surface material (subgrade) is most likely to be the problem. Six (6) VES carried out at the zone highlighted in red (zone of incompetent geomaterial) was used to revalidate the classification of the geomaterials as incompetent and to further see deeper into the subsurface.

The geo-electric section was deduced from the VES curve (see Fig13-18) and the geo-electric section as shown in the Fig 19 delineates four sub surface layers which include clayey top soil, clay, clayey sand and sandstone. The soils having resistivity values ranging from 54.50Ωm to 32300Ωm, corresponds to clay and sand respectively. First Layer was identified as lateritic clayey top soil with resistivity of 96Ωm - 392Ωm and thickness of 0.5m – 2.1m. The second layer was identified as clay/clayey sand with resistivity of 54Ωm - 980 Ωm and thickness of 7.5m– 19.4m while third/fourth layer is made of sand with resistivity of 3290Ωm - 32300Ωm

It was observed that at the 1.5m (subgrade soil) the lithology along the profile is clayey sand and clay which also correlated with the geotechnical result of the analyzed sample.

Along the (NW) profile the thickness of the clayey sand content increases while the clay decreases until it tins out at VES R5. The clay lithology of the soil especially at the subgrade is the reason for the consistent failure of the road pavement. This is because clay is very unstable as it can easily swell or shrink with water variation.

4.2.2 Geotechnical

Sieve analysis indicated 26% passing of sieve size 0.75mm for sample S01 which when compared with the AASTHOM145, BS and FMW, 1997 revealed fair-good subgrade materials for road and bridges. The implication of the result is that only 26% of the total grain size of S01 is classified as clay. Analysis of sample S02 and S03 showed that S02 and S03 can be classified

as silty-clay, composed majorly of compressible clay material with above 40% passing of sieve size 0.75mm. It could be observed that fineness (at 1.5m depth) generally increases SE along the Orsu-Ihiala road and this may be attributed to the local geology as it is observed to be the transition of the clay rich Ameke Formation to the Recent Mender belt (weathered and eroded Benin sand).

Analysis of the Atterberg Limit Liquid Limits (LL) and Plastic Limit (PL)) are relatively low (16.9%, 7.1% and 14.9% for S01, S02 and S03 respectively). The LL for S01, S02 and S03 are 38.5%, 42.5% and 46% respectively and according to Bell, 2007 are referred to as intermediate plasticity while their swell potential are classified as medium according to Holts and Gibbs 1956 (see Table5). The results when compared to Carter and Bentley, 1991 (see table 6) indicates possible calcium rich Kaoiline mineral. The plasticity of soil is influenced by the volume of its clay fraction and the type of clay minerals present (Arora, 2008). The plasticity and liquid limit decreases in the following order:

Montmorillonite > Chlorite > Illite > Kaolinite

The intermediate plasticity characteristics of the soils indicate intermediate susceptibility to loss of strength upon saturation and are classified as fair subgrade material according to the FMW, 1997 specification for Roads and Bridges.

The maximum dry density (MDD) and optimum water content (OMC) are 1.776g/cm³ and 14.4% for S01, 1.713g/cm³ and 14.6% for S02 and 1.692g/cm³ and 14.4% for S03. It was noticed that the MDD decreases and OMC increases along NW profile of the road which conforms with the result of the sieve analysis that also show clayness to increase along the same profile (this is because clay has high tendency of retaining water and hence would have low dry density). This

is an indication that the soil is generally loose and is poor as a subgrade material, with the looseness decreasing NW along the road. So there is need to stabilizer or further compact the soil if it must be used as a subgrade material. Since the strength of a soil increases it is when compacted (Bell, 2007). The California Bearing Ratio (soaked) for sample S01, S02 is within the FMW, 1997 specification for roads and bridges while and S03 is FMW, 1997 specification of Roads and bridges. It can be noted that the soaked CBR increases NW along the Orlsru-Ihiala road. The low CBR of sample S01 (even though the soil is classified as granular material), is an evident that the soil is highly porous and with moisture influx it will be highly damaging to the sub-grades of pavements constructed on them. The result of the linear shrinkage can be classified as medium/very high potential of swell according to Holtz and Gibbs (1956) see Table 10 shows that sample S01 and S03 has medium potential of swell while sample S02 has very high potential of swell and this might be the reason for the highest failure of the road observed at Amaebo LGA. From the specific gravity some deduction can be made. Soils with measurable organic content or with porous particles, such as diatomaceous earth, may have value below 2.0. On the other hand, soils which contain heavy substances such as iron may have values above 3.0. The specific gravity of studied samples is 2.401gm/cm^3 , 2.586gm/cm^3 and 2.413gm/cm^3 for S01, S02 and S03 respectively. Which indicates moderate organic content and moderate porosity (especially sample S01).

The moisture content of the samples is 18.2%, 24.2%, 25.4% for S01, S02 and S03 respectively. This result conforms to the sieve analysis that classified S01 as granular material and also agrees that the clayness increases NW of the studied road. When compared with the OMC this results showed a linear relationship. As expected Natural moisture content increases with increasing optimum water content. And this gives a stronger confident level for the interpreted results since the NMC and OMC are in conformance.

CHAPTER FIVE

CONCLUSION AND RECOMENDATOIN

5.1 Conclusion

The investigated area lies within the Anambra and Niger Delta basin consisting of the Eocene Ameke Formation, Oligocene Ogwashi-Asaba Formation and Recent Mender belt (weathered and eroded Benin sands). Electrical resistivity and geotechnical methods were used to investigate the causes of the consistent failure of the Orsul-Ihiala road. 2-D Wenner resistivity was first carried out to produce a resistivity 2-D section of surface along the road that covers the failed, fairly stable and stable portion of the road. 1-D (VES) was then carried out at the weak zone (these zones coincide with the failed portion of the road). Interpretation of electrical resistivity results show that the road was founded on a weak subgrade material (clay) with resistivity values of $10\Omega\text{m}$ - $135\Omega\text{m}$. the thickness and depth of this layers varies across the road with the worst portion of the road coinciding with the least resistivity value (below 2m). Investigating deeper into the subsurface, 3-4 geoelectric layers was interpreted (topsoil with resistivity of $96\Omega\text{m}$ - $136\Omega\text{m}$, clay with resistivity of $54\Omega\text{m}$ - $135\Omega\text{m}$, clayey sand with resistivity of $288\Omega\text{m}$ to $980\Omega\text{m}$ and sand with resistivity of $2780\Omega\text{m}$ - $32800\Omega\text{m}$. This implies deeper soil especially the sand (found at an average depth of 12m deep) is of good quality, since the strength of a soil is a function of its grain size, mineral and composition. The compaction of this sand is also believed to be good since dry sand within this area gave a resistivity of above $16000\Omega\text{m}$.

The geotechnical results showed that the soil is classified as granular (S01) and silty clay (S02 and 03) materials with % passing of sieve size 0.75mm of 26%, 42.2% and 49.7% for S01, S02 and S03 respectively. The samples classified as group A-2 subgrade, group A-7 subgrade and

group A-7-5 Subgrade as per AASTHO classification. The result of the Atterberg Limit showed that the plastic index (PI) is relatively low (16.9%, 7.1% and 14.9% for S01, S02 and S03 respectively). The LL for S01, S02 and S03 are 38.5%, 42.5% and 46% respectively and are referred to as intermediate plasticity. Implying the samples (S01 and S03) are of medium plasticity and intermediate swell potential while sample S02 is of Intermediate plasticity and has high potential of swell, according to Bell, 2001; Holtz and Gibson, 1956 and Murthy 2009 correlations. With LL and PL range of 34-73 and 26 – 36 the samples falls under calcium rich kaolinite as per Carter and Bentley, 1991 correlation standard. Results from compaction shows that the MDD and OMC are 776g/cm^3 and 14.4% for S01, 1.713g/cm^3 and 14.6% for S02 and 1.692g/cm^3 and 14.4% for S03. Low MDD with high OMC is an indication that the soil is generally loose.

The soaked California Bearing Ratio (CBR) for sample S01, S02 and S03 are 21.1%, 3.4% and 16.2% respectively. S01 and S02 fall within the FMW, 1997 classification of subgrade materials for roads and bridges while sample S03 falls outside this standard.

The specific gravity of studied samples is 2.401gm/cm^3 , 2.586gm/cm^3 and 2.413gm/cm^3 for S01, S02 and S03 respectively. And it indicates moderate organic content and moderate porosity (especially sample S01). The moisture content of the samples is 18.2%, 24.2%, 25.4% for S01, S02 and S03 respectively which also relates linearly to the observed OMC. Samples showed increasing moisture content and optimum moisture content, traversing NW- SE. The material is generally rated as a fair to poor subgrade material (as per general specification Roads and Bridges, Revised 1997, FMW, Nigeria).

Electrical resistivity and Geotechnical method has proven itself to be a great tool in carrying out subsurface structural studies and characterization due to its non-invasive, non-destructive and affordable nature. The results acquired from the electrical resistivity survey (2-D and VES) in the study area show that the study area is dominated by clayey topsoil and clay packages within the subgrade with resistivity range of $54\Omega\text{m}$ – $136\Omega\text{m}$ and this resistivity increases along the NW-SE trend of the road and this could be as a possible transition from the clay rich Ameke Formation to the Clayey sand of the recent Mender belt (prototype of weathered and eroded Benin sands). The quality of soil is a function of the grain size, minerals and composition. Generally sands and sandstones are good materials for laying engineering structures due to their mineralogy and composition. Geoelectric investigation suggests that the road was founded on a clay subgrade, the clayey content diminishes toward the Northwestern part of the road (heading to Ihiala state from Orsu). Geotechnical analysis revealed that clay material is of the poor- fair quality as per ASSTHO and FMW, 1997 for subgrade materials. This is because clayey subgrade soils have propensity of engrossing water which makes it swell and under enforced wheel load stress will subsequently result to road failure, as witnessed at the study area. Poor drainage system was also observed to be a contributing factor responsible for the possible failure of road.

5.2 Recommendations

Having carefully carried out this research, the following recommendation can be made:

1. The soils should be improved upon before they can be used as sub-grade material in road construction, improved by stabilization using lime, ordinary Portland cement, fly ash, wood, rice husk ash, sugar cane e.t.c.

2. Alternatively Cut and replace method can be adopted: this is the excavation of the poor soil and refilling it with a good quality soils which are suitable subgrade materials imported, provided they are within economic haulage distance from the construction site.
3. The poor drainage pattern of the road pavement also lead to its failure due to ponding and this can be overcome by creation of well-structured drainage systems.

5.4 Contribution to knowledge

This research work has made the following contributions to knowledge:

1. It has provided information on the cause(s) of the failed segments of the investigated road.
2. The information documented in this study will aid in the rehabilitation or reconstruction of the road especially the compaction test results (for soil stabilization).
3. This studies shows that Geophysical investigation can always complement Geotechnical studies in highway site investigation and construction

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APPENDIX

