**Journal of Geotechnical and Transportation Engineering**
Volume 7 | Issue 1
**Integrated Studies of Highway Deterioration along Ijebu Ode - Ore Road, Southwestern Nigeria**
Ifarajimi et al.
Received: June 2021
Accepted: January 2022
Corresponding Author:
Williams T. Ifarajimi;
williamstomjim@gmail.com
www.jgtte.com

# Integrated Studies of Highway Deterioration along Ijebu Ode - Ore Road, Southwestern Nigeria

Williams T. Ifarajimi
Department of Geosciences, University of Lagos, Akoka, Nigeria
williamstomjim@gmail.com

Michael I. Oladapo
Department of Applied Geophysics, Federal University of Technology, Akure, Nigeria
oladapom@yahoo.co.uk

Sunday Bayode
Department of Applied Geophysics, Federal University of Technology, Akure, Ondo State, Nigeria
sundaybayode@yahoo.com

*Abstract*

Electrical resistivity tomography (ERT) has been applied with geotechnical techniques such as Cone Penetrometer Test (CPT) and laboratory tests for subsoil characterization to investigate the causes of the deteriorated highway pavement. Eleven Electrical Resistivity (ERT) Profiling lines were established using Wenner array configuration in combination with ten cone penetrating data and eleven soil samples. The inverted ERT data consist of two to three geoelectric layers and were interpreted as topsoil (clay/sandy clay/clayey sand/sand), sand/saturated sand and dry sand/highly resistive sand/weathered rock with overlying resistivity values ranges between 23 – 550 Ωm, 100 – 1000 Ωm and 500– 2800 Ωm respectively. The cone penetrometer test (CPT) value ranges from 30 to 82 kg/cm$^2$. In addition, the laboratory analyses conducted on the bulk soil samples taken at 0 - 1 m depth includes; the optimum moisture content (OMC), maximum dry density (MDD), and California Bearing Ratio (CBR) ranges from 11.3 to 12.2%, 1720 kg/m$^3$ to 1960 kg/m$^3$ and 8 to 13% respectively, while for the Liquid Limit and Plasticity Index tests of the soil samples gives 28 to 52% and 9 to 17% respectively indicating that the subsoil material within study area are of poor to good geotechnical properties. The results of the integrated approach, including both geophysical and geotechnical methods have helped to identify the cause of the highway deterioration in some part of the study area which is attributed to the poor subgrade material along the region. Thus, the need for soil improvement can be implemented to enhance the stability of the subgrade materials in the poor region for subsequent road construction and design.

*Keywords:* Electrical Resistivity Tomography, Cone Penetrometer Test, subsurface characterization, road construction

## 1. Introduction

Highways provide inter-regional and international communication links with important centers in neighboring provinces for businesses and services. Good transportation medium serves as a lifeline of a nation's economy [1]. Due to the importance of highway transportation, detailed attention should be put in place for its development during the preconstruction stages, construction activities, and post construction/maintenance of the road. Pavement deterioration is the process by which distresses develop in pavement as a result of the joint effects of road usage, design, traffic loading/congestions and environmental conditions that were evident of an adverse condition in pavement. Thus, it affects the serviceability, smooth ride appearance and structural condition of the pavement [2]. Field observations and laboratory experiments conducted by [3, 4] discovered that highway failure are not solely caused by road usage or design/construction problems. Hence, it could also arise from inadequate knowledge of the characteristics and behavior of the residual soils on which the roads are built and non-recognition of the influence of geology and geomorphology during the design and construction phases [5]. Factors responsible for the deterioration of the highway pavement could arise from the near-surface geological sequences, arising from structures such as fractures, fault, sink holes, cavities, paleo-channels and shear zones. However, highway deteriorations are naturally led by the resultant destructive signs such as peeling, rutting, cracking, potholes and deformation [6 – 8]. In accordance with [9 - 11] highway deterioration ranges from geological, geomorphologic effects, road usage, non-adherence to standard practices during construction and lack of maintenance culture. Environmental observation and geotechnical investigation was carried out by [12] to study sources of irregular occurrence of deformation attributes on highway pavement in Ogbomoso town. It was observed that the pavement failure is due to shallow groundwater levels that exist within the study area. Ademilua [13] concluded that, poor soil properties such as low maximum dry density (MDD), poor bearing capacity, high compressibility, and high liquid limit, plasticity index, high optimum moisture content (OMC) and low California bearing ratio (CBR) are usually factors responsible for road failures. The selected failed segments have led to losses of life, properties, and delay in travel time by the road users. Despite several rehabilitation processes engaged by the Federal Ministry of Road Management Agency (FERMA) these segments have remained danger zones. Therefore, this study attempts to investigate the subsurface condition responsible for the accelerated deterioration of the highway pavement using integrated approach of geophysics and geotechnics.

## 2. Geology of the study area

The study area is located in SW-SE direction from Ijebu-Ode Bridge to Ajebamidele bridge between kilometer 218+080 and 166+530 Benin Bound (BB) (Fig. 1). The highway spans about 52 kilometers and between latitude $6.808431^0$N and $6.737820^0$N and longitude $3.893223^0$E and $4.343336^0$E. The lithological cross section revealed that the study area cut across different lithological bounds of the Dahomey Basin with a Tertiary cretaceous sequence (Fig. 2). The oldest formation in the basin is Abeokuta Formation reported to have a thickness range of 250-300m [14]. The Basin consists of arkosic sandstones and grits, tending to carbonaceous towards the base. Abeokuta Formation is in turn overlain by the Ewekoro/Akinbo/Oshosun Formations. The Ewekoro Formation consists of a sequence of sandstone, shale, limestone and clay varying between 100-300 m thick. This is further overlain by the Ilaro Formation, which has been reported to be deposited in transitional environment [14]. It consists of poorly sorted sandstones alternating with shale and clay. The sequence is capped by the Coastal Plain Sands which consists of a sequence of predominantly continental sands and some lenses of shale and clay [15]. Tertiary sediments thin out to the east and are partially cut off from the sediments of Niger Delta Basin against the Okitipupa Basement Ridge [16].

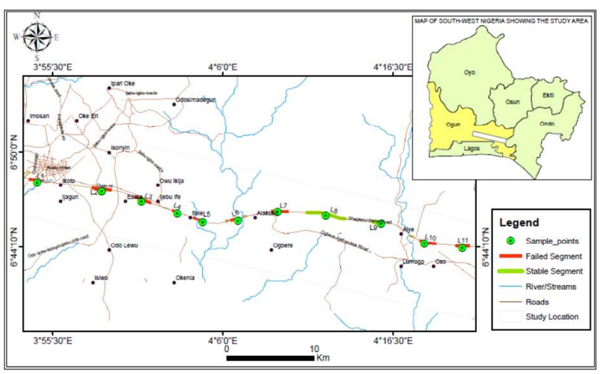


Fig. 1. Base Map of the study area

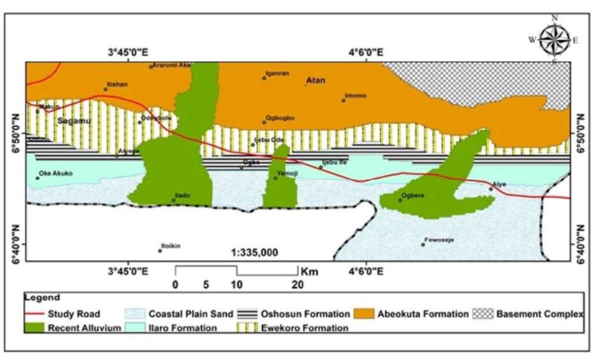


Fig. 2. Geological map of the Study Area showing the study highway [17]

## 3. Materials and Methods

The study involves the acquisition of geophysical and geotechnical information of the investigated area. 2D ERT investigations were carried out along eleven main survey traverses (L1 to L11 in Fig. 1). The ERT lines were acquired using a Wenner array configuration. The electrode spacing varied from 1.0 m to 6.0 m progressively. The observed acquired apparent resistivity data were processed and inverted with AGI EarthImager 2D inversion modeling software, using a least squares inversion modeling approach with regularization technique [18]. The geotechnical properties of the subsoil were carried out using the British standard code of practice for site investigation [19]. Ten (10) Dutch cone penetrometer test (CPT) signified as CPT1 – CPT10 were acquired using a 2.5 tonnes capacity penetrometer machine. The CPT tests terminate at depths ranges between 0.6 m and 5.0 m. The geotechnical data were acquired by taking bulk of disturbed soil sample from pits at the selected locations. A total of eleven (11) samples were collected for the geotechnical laboratory analysis, to determine the behavior of the soil and the engineering properties of the subgrade materials. The moisture content of soil samples from each location were obtained by finding the ratio of the weight of water in the soil sample to the dry weight of the soil sample. Sieve analysis was conducted on the soil samples to determine the particle size distribution of the soil. The consistency test limits were conducted to determine the plasticity index and the liquid limit of the soil materials, compaction and California bearing Ratio (CBR) of the soil material were all determined from the bulk soil samples that was taken from the selected segments.

## 4. Results and Discussion

The results of the electrical resistivity data acquired are presented as 2-D resistivity tomography; the inverted results are discussed with respect to their geological settings. The cone penetrometer test (CPT) results summaries are presented in Table 1.0, soil strength was evaluated and interpreted based on the horizons using guide to estimating soil types by [20]. The graphical representation of the depth of investigation were plotted against the cone resistance reading and presented in Fig 12 and 13. The summaries of the laboratory analyses made from the collected bulk soil samples were shown in Table 2. The particle size distribution of the collected soil samples were determined, Atterberg limits tests was carried out to determine the behavior of the soil, its strength and stability. The compaction characteristics of the subgrade were evaluated from the Optimum Moisture Content (OMC) and the maximum dry density (MDD) derived from the samples. The California Bearing Ratios (CBR) of the samples was determined after soaking the materials into fluid for four days (96 hours). The summary of the obtained results were presented in Table 2.

### *4. 1. Discussion*

#### *4. 1. 1. The 2-D Resistivity Structure along the Sedimentary Terrain*

The highway segments that fall within the sedimentary section of the study includes; location 1, 2, 3, 4, 5, 10 and 11. The variation in resistivity distribution within these sections of the study highway is presented in Fig. 3. The 2-D inversion structures depict two/three geologic units. These includes topsoil (intercalation of clayey sand and sand), sand/saturated sand and highly resistive sand/dry sand. The topsoil is predominantly sandy, with resistivity distribution range from 60 - 400 Ωm and varying layer thickness between 0.5 and 2.5 m. However, location 2 (Fig. 3) exhibit a non-uniform distribution of resistivity and a lateral discontinuity typical of a geological contact at near surface. The western section of this location between 0 and 40 m is predominantly clayey sand and sand, the layer thickness could not be determined. Location 5 (Fig. 3) resistivity structure revealed lateral discontinuity of structure which could be due to faulting within the near surface between 40 and 90 m along the traverse. The second geoelectric layer has resistivity distribution values that range from 250 - 1000 Ωm. The thickness of this stratum varies from 1 – 2 m. The layer is mainly composed of saturated sand/sand. Location 1 (Fig. 3) revealed lateral discontinuity and displacement of sediments at the western flank of the road segment between 0 and 28 m. The segment consists of clay/sandy with resistivity range of 24 – 45 Ωm and layer thickness that range between 1 and 2.5 m. The eastern segment of location 16 between 85 and 90 m also exhibit low resistivity range from 11 - 24 Ωm typical of clay materials. The third identified layer is mainly composed of high resistive sand/dry sand with resistivity values that range from 400 -1800 Ωm.

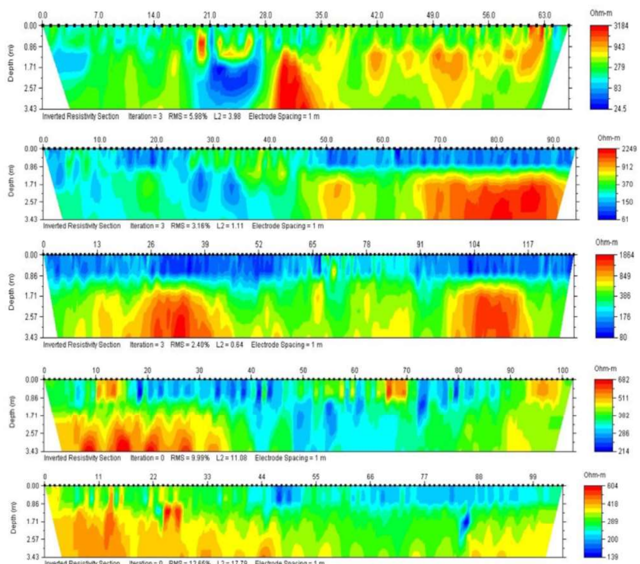

Fig. 3. 2-D Resistivity Tomography Profile (Wenner Array) of the sedimentary terrain at Locations (1, 2, 4, 5 and 10) respectively

### *4. 1. 2. The 2-D Resistivity Structure along the Basement Terrain*

The highway segments that falls within areas with outcrop exposures within the basin (Inselberg) were refers to as basement terrain. The segments include Location 6, 7, 8 and 9. The resistivity distribution tomography within these sections is presented in Fig. 4.

Locations 7 and 8 were the controlled stable segments. The result of the 2-D resistivity inversion revealed three geoelectric layers which consist of the topsoil (Clayey sand/sand), saturated sand and weathered basement rock. The topsoil has resistivity range from 50– 200 Ωm with layer thickness that varies from 1 - 1.7 m. The layer is composed of clayey sand/sand. Location 8 (Fig. 4) exhibit high resistivity values towards the eastern flank with resistivity range of 800 – 1000 Ωm. The resistivity values of the second layer range from 170 – 350 Ωm, this has thin layer thickness that varies from 1 – 2 m. the layer thin out towards the western flank of location 8 (Fig 4) between 0 and 54 m. The third identified geoelectric layer has resistivity values that range from 700 – 2204 resistivity values that range from 170 – 350 Ωm. the layer revealed lateral nonhomogeneous and discontinuity at shallow depths, the layer is suspected to comprise of weathered rock/rock. However, there is evidence of outcrops (inselberg) within the study vicinity. The result of the 2-D resistivity inversion at locations 6 and 9 revealed two geoelectric layers which consist of the topsoil (Sandy clay/Clayey sand/sand), and clay/sand/weathered basement rock. The topsoil has resistivity distribution range from 23 - 180 Ωm, the layer thickness varies between 1 – 2.2 m. this layer has intercalation of sediments, location 6 (Fig. 4) exhibit high resistivity values that range 1000 - 5000 Ωm within depth of 1 – 2 m below the first layer.

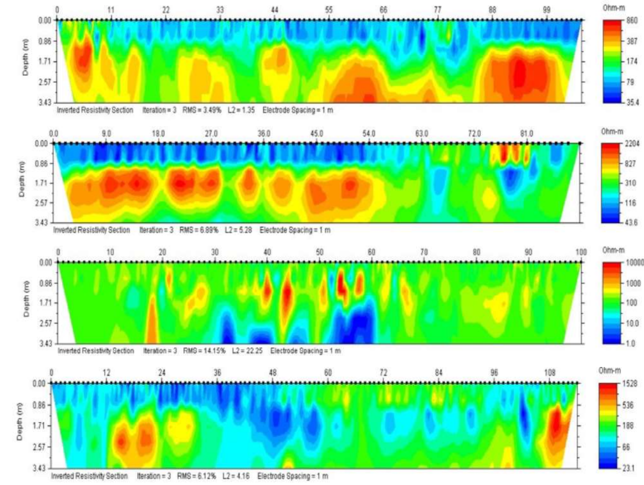

Fig. 4. 2-D Resistivity Tomography Profile (Wenner Array) of the Basement terrain at Locations (7, 8, 6, and 9) respectively

**Geotechnical Investigation**

The soil strength evaluation and delineation of soils horizon, conducted at CPT points 01, 02, 03, 04, 05, 09, and 10 respectively, delineate four geological layers comprising of soft clay, stiff clay/very loose sand, very stiff clay/ loose sand and medium dense sand.

CPT points 01, 02, 05, 09, and 10 have similar horizons. The first horizon has average cone resistance ($q_c$) value that range from 29 - 40 kg/cm$^2$ with layer thickness range from 0.2 – 0.9 m. The second layer consists of medium dense sand, with average cone resistance ($q_c$) value that range from 50 - 100 kg/cm$^2$ and layer thickness value that varies from 0.3 – 1.66 m. CPT 09 (4.48), third horizon has average cone resistance ($q_c$) value of 30 kg/cm$^2$with layer thickness of 0.4 m is significant of loose sand/very stiff clay. The bedrock has average cone resistance ($q_c$) value of 80 kg/cm$^2$ and layer thickness of 1.1 m is significant of medium dense sand/hard clay.

CPT point 03 exhibits three distinct geologic horizons. Layer 1, 3 and 5 have cone resistance ($q_c$) value range from 22 – 40 kg/cm$^2$ with layer thickness of 2.67 m is identified as loose sand. Layer 2 and 4 comprised of very loose sand with average cone resistance ($q_c$) value of 18 kg/cm$^2$, the layer has thickness of 0.87 m. The bedrock consists of medium dense sand with average cone resistance ($q_c$) value of 66 kg/cm$^2$ and layer thickness of2.5 m.

CPT point 04 (Fig. 5) has cone resistance ($q_c$) value that range from 10 - 20 kg/cm$^2$ at layer 1 and 3 with layer thickness of 0.8 m. The layers are significant of stiff clay materials. The second horizon has cone resistance ($q_c$) value of 2 kg/cm$^2$ and thickness of 0.2 which is significant of soft clay. The bedrock horizons is typical of very stiff clay, the average cone resistance ($q_c$) value is 68 kg/cm$^2$ with layer thickness of 0.75 m.

The soil strength evaluation and delineation of soils horizon, carried out at CPT 07 and 08 (stable segment) identified two horizons consisting of loose sand and medium dense sand (Fig. 6).

The first horizon has cone resistance ($q_c$) value ranging from 31- 40 kg/cm$^2$ and thickness value ranges between 0.25 – 0.6 m which signifies loose sand. The second identified stratum is typical of medium dense sand with cone resistance ($q_c$) value ranges from 77 - 82 kg/cm$^2$. The thicknesses of the layer vary from 1.0 - 2.25 m. CPT Point 06 delineate three geological layers comprising of stiff clay, very stiff clay and hard clay.

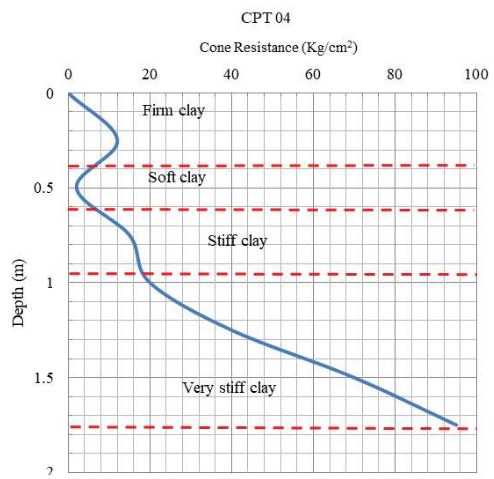


Fig. 5. Graph of Depth against cone resistance for CPT.

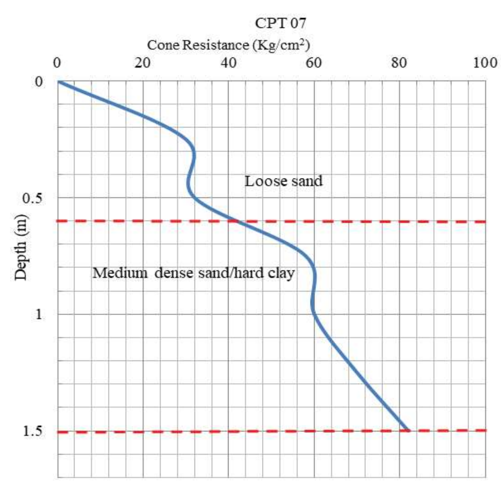


Fig. 6. Graph of Depth against cone resistance for CPT 07

The results of the liquid limit (LL), plastic limit (PL) and Plasticity Index (PI) for the collected soil samples as presented in table 2 shows that the liquid limit varies from 19 to 59%, while the plastic limit and plasticity index varies from 0 to 39% and non-plastic (NP) to 20% respectively. However, from the guideline of [21, 22], the liquid limit should not exceed 35% for its suitability as subgrade materials. The values of the liquid and plastic limits are generally high and signify the clayey nature of the soil samples as displayed in the seven soil samples of locations (4, 6-11) which are symptomatic of poor subgrade materials not suitable for road construction. The remaining four samples from locations (1 -3 and 5) exhibit LL and PL value that do not exceed the specifications limit. Hence, they could be categorized as good sub-grade materials suitable for road construction. The particle size distribution analysis was constrained by specification of [22] requirement for subgrade soil materials to have less than 35% amount of fines. The comparison between [22] and the obtained results indicates that six out of the soil samples did not agree with the required specification of less than 35% amount of fines. Therefore such soil is liable to frequent swelling and shrinkage resulting from the seasonal change effect within the investigated area. The abundance of the fine grains/clay material is not suitable for subgrade materials. The compaction analysis of the subgrade materials along the investigated highway has maximum dry density (MDD) and optimum moisture content (OMC) value range from 1510 to 1990 kg/m$^3$, and 6.3 to 19.4% respectively. The MDD value range of 1940 - 1990 kg/m$^3$ categorized the good region with corresponding OMC values of 6.3 to 12.2%. The poor zones were considered by low value of MDD that ranges from 1510 to 1780 kg/m$^3$ with a high OMC percentage of 13.1 to 19.4%. Hence the higher the value of MDD and relatively lower OMC, the more suitable is the subgrade material to support imposed load. The compaction results from the good region conform to the acceptable standard. The results obtained from the CBR analyses ranges from 6 to 20% and were categorized as poor to excellent. The [21] specification on highway design, accepts (soaked) CBR value for subgrade soil within the range of 10 to 25% for flexible pavement design. Therefore, any CBR $\leq$10% should be categorized as poor, also CBR value between (11-15%) be classified as good, while the CBR value between 15 – 20% is classified as excellent. The CBR values with $\leq$10% denote a poor subgrade materials. The CBR within the good region falls within the acceptable limit for good subgrade materials.

Integrating both geoelectrical and geotechnical results for subsoil characterization revealed both the presence of competent and incompetent layer inform of sand and sandy clay/ clayey sand unit within the upper segment of the classified good region and poor zone respectively. The results of geophysical investigation and the geotechnical analysis conducted on the soil samples in accordance to the acceptable standard of practice and procedure were used in classifying the highway into regions (good and poor). The good regions referred to as the subsurface materials that falls within the limit of the acceptable standards of practice, while the poor zones were the subgrade materials within the locations that are susceptible to anomalous dilatation, swelling and shrinking. Thus, compromise the integrity of road construction in some part of the investigated area. It is however suggested that areas with poor subgrade material should be enhanced using soil improvement or other cost effective means that will boost the stability of the road construction. Therefore the above findings have further demonstrated that conducting geophysical technique prior too boring and other geotechnical methods can give an insight about the subsoil characterization for road construction design. Integrated geophysical methods such as ERT with geotechnical approach will help and improve the confidence levels of the engineers in making the right decisions regarding the suitable approach to reconstruct the road.

## 5. Conclusion

The combinations of electrical resistivity tomography and geotechnical investigation were carried out along Ijebu Ode - Ore highway to understand the deterioration of the highway. The results show that the study highway was classified into two to three lithological units. The topsoil mainly comprised of sandy clay/clayey sand and sand.

Table 1: Summary of the cone penetrometer test (CPT) at the study area

| LOCATIONS | Depth (m) | Thickness (m) | $q_c$ (kg/cm$^2$) | $q_a$ (kPa) | Soil Type |
|---|---|---|---|---|---|
| **CPT 01** | 0.9 | 0.9 | 29 | 78 | Loose Sand |
| | 1.5 | 0.6 | 100 | 270 | Medium Dense Sand |
| **CPT 02** | 0.35 | 0.35 | 30 | 81 | Loose Sand |
| | 2 | 1.66 | 72 | 195 | Medium Dense Sand |
| **CPT 03** | 0.62 | 0.62 | 28 | 76 | Loose Sand |
| | 1.1 | 0.52 | 15 | 41 | Very loose sand |
| | 1.55 | 0.45 | 23 | 62 | Loose Sand |
| | 1.9 | 0.35 | 18 | 49 | Very loose sand |
| | 3.5 | 1.6 | 34 | 92 | Loose Sand |
| | 6 | 2.5 | 66 | 179 | Medium Dense Sand |
| **CPT 04** | 0.4 | 0.4 | 12 | 32 | Firm Clay |
| | 0.6 | 0.2 | 2 | 5.4 | Soft Clay |
| | 1 | 0.4 | 18 | 48 | Stiff Clay |
| | 1.75 | 0.75 | 68 | 184 | Very Stiff Clay |
| **CPT 05** | 0.25 | 0.25 | 40 | 108 | Loose Sand |
| | 1 | 0.75 | 82 | 221 | Medium Dense Sand |
| **CPT 06** | 0.25 | 0.25 | 20 | 54 | Stiff Clay |
| | 1.05 | 0.8 | 31 | 84 | Very Stiff Clay |
| | 2 | 0.95 | 66 | 177 | Hard Clay |
| **CPT 07** | 0.5 | 0.5 | 31 | 84 | Loose Sand |
| | 1.5 | 1 | 82 | 221 | Medium Dense Sand |
| **CPT 08** | 0.25 | 0.25 | 40 | 108 | Loose sand |
| | 2.5 | 2.25 | 77 | 208 | Medium dense sand |
| **CPT 09** | 0.2 | 0.2 | 40 | 108 | Loose Sand |
| | 0.5 | 0.3 | 50 | 135 | Medium Dense Sand/Hard Clay |
| | 0.9 | 0.4 | 30 | 81 | Loose Sand/Very Stiff Clay |
| | 2 | 1.1 | 80 | 216 | Medium Dense Sand/Hard Clay |
| **CPT 10** | 0.28 | 0.28 | 40 | 108 | Loose Sand/Very Stiff Clay |
| | 1 | 0.72 | 81 | 218 | Medium Dense Sand/Hard Clay |

Table 2: Summary of the Geotechnical Results.

| | | **Particle Size** | | | **Consistency Limit** | | | **Compaction** | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **L (km)** | **G %** | **S %** | **C %** | **Silt %** | **LL %** | **PI %** | **GI** | **OMC %** | **MDD kg/m$^3$** | **CBR %** | **Remark** |
| S1 | 0.0 | 89.0 | 3.6 | 7.4 | 19 | NP | 0 | 6.3 | 1940 | 20 | Good |
| S2 | 8.0 | 64.0 | 22.3 | 5.7 | 21 | 7 | 0 | 11.0 | 1960 | 15 | Good |
| S3 | 0.0 | 77.0 | 14.5 | 8.5 | 22 | 7 | 0 | 10.9 | 1990 | 16 | Good |
| S4 | 2.0 | 49.0 | 36.8 | 12.2 | 44 | 15 | 4 | 15.7 | 1780 | 6 | Poor |
| S5 | 7.0 | 74.0 | 11.0 | 8.0 | 23 | 8 | 0 | 10.5 | 1970 | 14 | Good |
| S6 | 2.0 | 55.0 | 28.8 | 14.2 | 59 | 20 | 5 | 19.4 | 1510 | 2 | Poor |
| S7 | 9.0 | 47.0 | 28.8 | 15.2 | 49 | 15 | 4 | 15.3 | 1760 | 8 | Poor |
| S8 | 20.0 | 53.0 | 12.5 | 14.5 | 43 | 13 | 0 | 13.1 | 1780 | 11 | Poor |
| S9 | 15.5 | 43.2 | 33.1 | 8.2 | 45 | 15 | 1 | 14.8 | 1840 | 8 | Poor |
| S10 | 19.0 | 52.0 | 14.9 | 14.1 | 40 | 13 | 0 | 12.2 | 1960 | 12 | Good |
| S11 | 13.0 | 35.0 | 39.4 | 12.6 | 52 | 17 | 7 | 16.2 | 1780 | 9 | Poor |

*L denotes Location, G indicates Gravel, S: Sand, and C: Clay.

The second horizon denotes loose to medium sand while the last stratum represent medium dense sand/weathered rock. The results of the geotechnical studies shown that the Cone Penetrometers Test (CPT) value ranges from 30 to 82 kg/cm$^2$. Additionally, the laboratory analyses conducted on the bulk soil samples comprises; the Optimum Moisture Content (OMC), Maximum Dry Density (MDD), and California Bearing Ratio (CBR) ranges from 11.3 to 12.2%, 1720 kg/m$^3$ to 1960 kg/m$^3$ and 8 to 13% respectively, while the Liquid Limit and Plasticity Index tests of the soil samples gives 28 to 52% and 9 to 17% respectively indicating that the subsoil material within study area are of poor to good geotechnical properties. The plasticity characteristics of the investigated highway indicate that the subgrade materials within the good region exhibit clay fraction of low plasticity to non-plastic, while the subgrade materials within the poor zones displayed an intermediate to high plasticity. From the integrated results of the geophysical and geotechnical studies, the poor zones of the studied highway are due to the lithology (poor subgrade/sub-base) failure associated with high constituents of clay materials that manifests as low resistivity readings, high moisture contents, high plasticity and low bearing capacity of the road formation among other physical properties constituted the failure. To solve this problem, the need for soil improvement can be adopted to enhance the stability of the subgrade materials for the road construction design.

**References**

[1] O. O. Olubomehin, "Road transportation as lifeline of the economy in Western Nigeria 1920 to 1952", *African Journal of History and Culture*, vol. 4, no. 3, pp. 37-45, 2012.

[2] M. M. E. Zumrawi, "Investigating Causes of Pavement Deterioration in Khartoum State", *International Journal of Civil Engineering and Technology,* vol.7, pp. 203–214, 2016.

[3] A. O. Adeleye, "Geotechnical Investigation of Subgrade Soil along Sections of Ibadan-Ile Highway", Unpublished M.Sc. Project, Obafemi Awolowo University, Ile-Ife, 2005.

[4] L. A. Ajayi, "Thought on Road Failures in Nigeria", *The Nigerian Engineer*, vol. 22, no. 1, pp.10 – 17, 1987.

[5] M. I. Oladapo, M. O. Olorunfemi, J. S. Ojo, "Geophysical investigation of road failure in the basement complex area of South-western Nigeria", *Research journal of Applied Sciences* vol. 3, no. 2, pp. 103-112, 2008.

[6] O. O. Olofinyo, O. Olabode, I.O. Fatoyinbo, "Engineering Properties of Residual Soil in Part of Southwestern Nigeria: Implication for Road Foundation", *SN Applied Sciences*,no. 1, pp. 507,2019

[7] I. I. Sabinus, K. I. Kalu, I. S. Nwankwo, "Geoelectrical and geotechnical evaluation of foundation beds of Naze, Owerri Southeastern Nigeria", *Journal of Earth Sciences and Geotechnical Engineering*, vol. 1, no. 3, pp. 77-94, 2014.

[8] I. A. Adeyemo, A. A. Akinlalu, K. A. Mogaji, O.O. Odumosu, "Integrated Analysis of Geophysical Data for Road Networks Sub Base Lithology Integrity Assessment Case Study in Crystalline Basement Complex, Southwestern Nigeria", *Journal of Geography, Environment and Earth Science International*. vol. 24, no. 4, pp. 15-28, 2020.

[9] I. A. Adeyemo, G. O. Omosuyi, "Geophysical investigation of road pavement instability along Akure-Owo Express Way, Southwestern Nigeria", *American Journal of Scientific and Industrial Research*, vol. 3, no. (4, pp. 191-197, 2012.

[10] K. A. N. Adiat, A. A. Akinlalu, A. A. Adegoroye, "Evaluation of road failure vulnerability section through integrated geophysical and geotechnical studies", *NRIAG Journal of Astronomy and Geophysics*. vol. 6, pp. 244- 255, 2017.

[11] G. M. Olayanju, K. A. Mogaji, H. S. Lim, T. S. Ojo, "Foundation integrity assessment using integrated geophysical and geotechnical techniques: Case Study in Crystalline Basement Complex, Southwestern Nigeria", *Journal of Geophysics and Engineering*. vol. 14, pp. 675-690, 2017.

[12] O. S. Oladeji, and T. B. Adedeji, "Causes of nonuniform deformation features on highway pavement', *Journal of Sci. Eng. And Tech*. vol. 8, no. 4. 2001.

[13] O. Ademilua, "Geotechnical characterization of subgrade soils in Southwestern Part of Nigeria", In: Proceedings of first and second international conferences of the Nigerian Association of Engineering Geology and the Environment, Lagos, Nigeria, 2018, vol 1, pp. 42–48.

[14] R. A. Reyment, "Aspects of the Geology of Nigeria", Ibadan University Press, Ibadan. 1965, pp. 145.

[15] A. B. Durotoye, "Quartenary sediments in Nigeria. In: Kogbe C. A. (ed) *Geology of Nigeria*", *Elizabeth Press, Lagos*. 1975, pp 431 – 451.

[16] M. E. Omatsola, and O. S. T. Adegoke, "Ecotonic Evolution and Cretaceous stratigraphy of Dahomey Basin", *Journal of Mining Geology*, vol. 18, no.1, pp. 130 -137, 1981.

[17] K. F. Oyedele, S. Oladele, O. Adedoyin, "Application of Geophysical and Geotechnical Method to Site Characterization for Construction Purposes at Ikoyi, Lagos, Nigeria", *Journal of Earth sciences and Geotechnical Engineering,* Vol.1, no. 1, pp. 87-100, 2011.

[18] M. H. Loke and R. D. Barker, "Rapid least-squares inversion of apparent resistivity pseudosections by a quasi-Newton method", *Geophysical Prospecting*, vol. 44, pp. 131–152, 1996.

[19] British Standards Institution (BSI). British Standard Methods of Practice for Site Investigation. B.S 5930, 1999.

[20] J. H. Schmertmann, "The Measurement of In-Situ Shear Strength" Proc. ASCE Specialty Conference In-Situ Measurement of Soil Properties, Vol. 2, pp. 57- 138, 1975.

[21] British Standard BS 1377, Methods of test for soil for civil engineering purposes, 1975.

[22] Federal Ministry of Works and Housing (FMW&H), General Specification for Roads and Bridges, 1997.