AN INVESTIGATION INTO THE CAUSE OF ROAD FAILURE ALONG SAGAMU-PAPALANTO HIGHWAY SOUTHWESTERN NIGERIA

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ABSTRACT

Road Failure in Nigeria has been a monumental disaster that has resulted to loss of lives and damage of vehicles which has also in turn increased the travel time of motorist plying on these roads. In order to unravel the cause of this disaster geotechnical, geochemical, mineralogical and geophysical tests were conducted to evaluate the cause of failure along Sagamu-Papalanto Highway southwestern Nigeria. The laboratory tests result conducted shows that the percentage amount of fines ranges from 12-61.3%, natural moisture content ranges from 6.8 to 19.7%, the liquid limit in the range of 25.1-52.2%, linear shrinkage between 3.96 to 12.71%, plastic limit ranges from 18.2-35%, the plasticity index ranges from 5.2 to 43.9%, maximum dry density from 1.51 to 1.74 g/cm³, the specific gravity ranges from 2.52 to 2.64 and the CBR between 3-12%. The Cone Penetrometer Test (CPT) has a resistance value in the range of 20–138 kgf/cm². There is the predominance of kaolinite as the major clay minerals and the main oxides in the study area shows SiO₂, Al₂O₃, Fe₂O₃, K₂O, Na₂O, MgO and CaO. 2D Electrical Resistivity Wenner Array Method was employed in the investigation. Four profiles covering a distance of 250 meters for profiles 1, 2, and 3 and 500 meters for profile 4 each were established parallel and perpendicular to the road pavement along the sections of the road. Data were gathered along the four profiles using ABEM Terra meter SAS 1000. The recorded field data were filtered and inverted using 2-D modelling inversion algorithm with the aid RES2DINV Software. The results revealed a low resistivity values for profile 2 and 3 ranging from 100 Ωm – 300 Ωm, between distances of 20 m – 240 m along the profile to a depth of 7.60 m and a low resistivity value ranging from 50 Ωm – 111Ωm, between distances of 80 m – 120 m along the profile to a depth of 15m from the surface of the profile the topsoil along the profile. The low resistivity values and low shear strength obtained from the profiles shows the study areas comprises of incompetent materials which has the propensity to retain water and eventually results to swelling and collapse under imposed load with subsequent failure as the resultant effect. Sections of the road
with sandy and clayey materials should be scooped out from the subsurface to a depth of 3 m – 5 m from the top soil of the road and put back with competent fill materials.

**Keywords:** Disaster, Road, Failure, Soil, Liquid limit, Plastic limit, Geophysical Investigation

UOE carried out the field and Laboratory work drafted the manuscript

IO and UBU Conceived the study participated in its design, coordination and gave academic guidance.

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**Introduction**

Road transportation is of paramount importance to every society and country, as its dictate the pace of socio economic development of such society as it known that no economy can thrive where there is no good road. Moreover, roads plays an important part in attaining national development and contribute to the general achievement; performance and social functioning of the society. In view of the fact that is a robust positive interrelatedness between a country’s economic growth and the standard of its road network, a country’s road network should be designed and constructed in a systematic and well organized way in order to optimize and maximize the social and the economic satisfactory benefits (Ighodalo, 2009). It is of importance
to highlight that that Nigeria roads shows a distinctiveness and characterized with long cracks, potholes and lots others pavement defects known to road construction. These have possess a serious challenge and disaster that in Nigeria one can hardly travel a mile without coming across long cracks and potholes which have resulted to a spike in accidents occurrence and a plunge on the nation’s economic development. Each and every single road built have a specified stipulated design life but roads often times fail long before the planned expected date, some fail straight away after construction, more after flooding, some stay up to half of its life cycle while others last to its entire full life expectancy with appropriate maintenance.

Adegoke-Anthony and Agada (1980) reported that excessiveroad usage, poormaintenance and design in construction practices have been responsible and known cause of road failures. Laboratory experiments and field monitoring and observation have revealed that road failures are not predominantly, unsatisfactory supervision, road usage, non-compliance to design specification, substandard construction materials problems solely but can evenly emanate from insufficient knowledge of the behavior and the characteristics of the residual soils on which the road are built (Ajayi, 1987). Different explanation have been suggested for the incessant and ceaseless breakdown and failure of roads in Nigeria which includes the appearance and the existence of expansive clays such as chlorite, halloysite, montmorillonite etc. (Mesida, 1986; 1987), heterogeneity; dissimilarity of the subgrade materials (Mesida, 1987; Adeleye, 2005), existence and appearance of unexposed linear features, which includes fractures, rock boundaries and joints (Momoh et al., 2008). Geological elements and components such as the near surface geologic sequence, the nature of the subgrade material (top soil), occurrence of geological structures which includes faults, fractures, shear zones, occurrence of ancient stream channels composed of geohazards that can damage the stability and solidity of any road structure. Various researchers in an attempt to unravel the cause of continual road failure in Nigeria have identified the underlying geologic conditions primarily amidst others factors responsible for this catastrophe (Oladapo et al., 2008;Momoh et al., 2008.,Adiat et al., 2009).

Various authors and researchers have carried out researches on the investigation of the road failures in Nigeria using geophysical methods. Momoh et al. (2008) investigated the cause of highway failurealong the basement complex of southwestern Nigeria using geophysical methods by employingvery low frequency electromagnetic (VLF-EM), electrical resistivity such as Schlumberger Vertical Electrical Sounding (VES), Dipole –dipole electrical methods were applied across Ilesha- Owena highway with the aim of outlining and identifying the subsurface structural characteristics inside the subgrade soil and marking out the bedrock relief with the aim of confirming the probable reasons of pavement failures (Momoh et al., 2008). Throughout the length of the failed sections, water permeating stratum, linear attribute presumed to be fracture zones, faults, buried stream channel and joints were marked out (Momoh et al., 2008). Oladapo et al. (2008) researched into the cause of road failures in the basement complex area of southern Nigeria using Electrical Resistivity Method encompassing Wenner Vertical Electrical Sounding (VES) techniques and Dipole–dipole it was revealed that the failed section of the road are delineated by fairly low resistivity value of less than 200 Ωm with stable section having a resistivity value that is greater than 400 Ωm (Oladapo et al., 2008). Adiat et al. (2009) investigated the cause of road and pavement failure using Electrical Resistivity and Electromagnetic methods, Very Low Frequency Electromagnetic profiling and lateral resistivity profiling same at 10m intervals and 21 Schlumberger Vertical Electrical Sounding (VES) were carried out to appraise the subsurface solidity of a 3km road at Igbara –Oke to Ibuji in south western Nigeria. It was revealed that the failed section of the road was a
result low resistivity of the near surface materials and the superficiality of the aquiferious region that on which the road pavement was laid (Adiat et al. 2009). Akintorinwa et al. (2010) conducted a research on the investigation of road failure using geophysical methods along Ilesa-Akure highway with the objective of unravelling the cause of pavement failure; the research studied two stable sections and four failed sections it was revealed that the subsoil on which the road pavement was laid which serves as the control have high resistivity value greater than 200 Ωm whereas the failed section have a low resistivity value of 200 Ωm (Akintorinwa et al; 2010). Osinowo et al. (2011) researched on the causes of road failure along the basement complex terrain in southwestern Nigeria using integrated geotechnical and geophysical methods, the researchers employed Electrical Resistivity method and Very Low frequency Electromagnetic in order to delignate road sections with inconsistent electrical feedback and explained the findings in connection to the lithology, water saturation and structures (Osinowo et al.; 2011). It was revealed from the study that geotechnical and geophysical investigation provide a very practical way for delineating near surface earth which can be productive in site preparation preceding to completion(Osinowo et al.;2011). Adeyemo and Omosuyi (2012) investigated by using geo-electric sounding , Very low Frequency Electromagnetic (VLF-EM), and ground magnet magnetic profiling top unravel the cause of instability of road pavement along Akure-Owo expressway; it was concluded from the study that low resistivity values acquired underneath the failed section which denotes clay and the vulnerability of the road pavement is likely precipitated by the appearance of near surface bedrock depressions filled by low resistivity eroded weathered materials, representative of problematic expansive clay and sandy clay that is classified unfit as construction materials (Adeyemo and Omosuyi ,2012). Jatto et al. (2014) investigated the cause of road failure along Sarkin Pawa- Mangoro Highway Niger north central Nigeria using Magnetic method and Schlumberger Vertical Electrical Sounding (VES); the research employed 28 VES stations at a partition of 50mwhich was set up side by side to the road pavement on individual side of the road; it was concluded from the study that the geo-electric section along the stable section showed mainly resistive subsurface whereas the magnetic profile revealed a uniformed subsurface void of near surface water table, geological substratum and a substantial low magnetic linear attributes that is considered to be fault within the basement or an ancient stream channel which has been engulfed with sand.

Sagamu-Papalanto highway is one of the busiest highway in southwestern roads due to the presence of industries in the area. The road under investigation links to the Lafarge cement company at Ewekoro, Dangote Cement Company at Ibese, NNPC gas plant station at Oloruungosho and the deplorable state of this road has result to series of accident and an avenue for crime due to the high rate of kidnapping and armed robbery attack as a result of slow movement of vehicles plying the road and in later time as resulted to slow rate of socio economic development due to people abandoning the road for other alternative route. The road under investigation is a flexible pavement which is made up of the subbase, subgrade, base and the wearing course. The subgrade is the natural soil that act as foundation for the road and is directly overlap by the subbase which consists of soils imported from another area. The base of the road consist of aggregates obtained from basement rocks from the site whereas the wearing course made up of bitumen. As at the moment of this study, a significant port of the road orientation and alignment has failed. The distress was obvious in different forms as rutting along the road, raveling, cracks and pothole (Fig 1).In some segment of the road the corrugation, potholes and cracks are occasionally up to 1.5-2meters large 1m deep(Fig 1). The road is totally without drainage provision. The various types of failure at different locations are described in Table 1. It was also noted that the road is completely devoid of drainage facilities. This objectives of this
study is to utilize integrated geophysical and geotechnical methods in investigating the causes of road failure and evaluating the geo-electrical properties of soil in the study area and their implication to road and pavement failure.

Figure 1: Showing road failure in the study location

Table 1: showing the Locations of distress.

| S/n | Coordinates | Failure type | Location | Remark                                         |
|-----|-------------|--------------|----------|------------------------------------------------|
| 1   | 3°23´29.627´´E 6°53´1.217´´N 3°33´19.68´´E 6°53´12.48´´N 3°24´27.36´´E 6°53´46.48´´N | Cracks | 1, 2, 11, 20 | Seriously cracked with the bituminous layer completely altered. |
| 3°24′17.12″E 6°53′17.38″N | | | |
|--------------------------|-----------------|---------------------|
| 2 | 3°17′38.27″E 6°52′46.56″N | Potholes | 4, 5, 7, 9, 19, |
| 3°19′22.08″E 6°52′46.56″N | | | Crater shaped defect with fairly extensive with in some places cutting as deep as 0.5 m in some of the locations. It retains considerable quantity of rain water and provide an easy passage for the entry of rainwater |
| 3°19′22.08″E 6°52′46.56″N | | | |
| 3°17′12.48″E 6°52′64.56″N | | | |
| 3°16′46.56″E 6°52′20.64″N | | | |

| 3 | 3°15′28.47″E 6°51′54.72″N | Corrugation and Raveling | 3, 8, 6, 10, 12, 14, 16, 17, 18, |
| 3°13′19.04″E 6°52′20.64″N | | | The portion are fully broken down with the structural components of the road entirely pulled out and removed |
| 3°11′35.52″E 6°53′35.52″N | | | |
| 3°12′10.41″E 6°52′47.50″N | | | |
| 3°22′26.88″E 6°52′34.57″N | | | |
| 3°15′28.18″E 6°52′46.32″N | | | |
| 3°16′20.54″E 6°52′20.64″N | | | |
| 3°53′12.22″E 6°52′53.22″N | | | |
| 3°34′42.24″E 6°53′12.48″N | | | |
2.0 Location, Geomorphology, Climate and Geology of the study area.

The study area lies within Longitude 3°11’35.627”E to 3°34’42.25”E and Latitude 6°51’1.217”N to 6°54’16.457”N on the Sagamu –Papalanto Expressway in Ogun State Southwestern Nigeria (Fig. 2). The road extends up to 50km long which links to other connecting cities like Ewekoro, Ibese, Ifo and Lagos -Ibadan Express way which also links to other part of the country. Sagamu-Papalanto study area is enclosed the north by Oyo and Osun States in the south by Lagos State in the east by Ondo State and in the west by Benin Republic. It transverses between Papalanto Junction through several villages and settlement to Sagamu Interchange. The geology of the area comprises of sedimentary units which consists of argillaceous sediment which is soft and friable and some siliceous and ferruginous materials (Fig 3). The Geology and stratigraphy of the Dahomey basin has been grouped into six lithostratigraphic Formations namely, from oldest to youngest namely: Abeokuta, Ewekoro, Akinbo, Oshosun, Ilaro, and Benin Formations (Elueze&Nton 2004;Nton 2001;Okosun 1990;Omatsola& Adegoke 1981;Ako, et al.,1980) The Abeokuta Formation represents an irregularity accompanied and lay directly above the basement complex, Ewekoro, Oshosun and Ilaro Formations overlie these in turn and all are overlayed by the coastal plain sands of the Benin Formation. The studied area is located on sedimentary Formation of the south western Nigeria and underlain by the basement complex Adegoke et al. (1976). It belongs to the Ewekoro Formation which is the Tertiary formed during the Paleocene and Eocene period which also forms a greater depression of artesian basin for groundwater Formation mostly made up of shale/clay (Ubido etal, 2017, 2018). The area has a wide spread humid tropical climate pronounced by the presence of wet and dry seasons. The wet seasons starts in April and terminates in October whereas the dry seasons runs from October to March. The mean annual rainfall is approximately 1375mm and the style and pattern of rainfall is dual mode in nature, in addition to an annual peak in the month of June and a secondary maximum in the month of September (Balogun, 2003).The mean yearly temperature also differ from 22°C wet season mean to 30 °C dry season mean whereas the humidity range from 40% December mean to 80% July mean. The whole of the study has area has moist air that comes from the Atlantic Ocean throughout the year. The relief is of moderately uneven and undulating; the area is predominated by dendritic drainage pattern it implies quite similar resistance of the underlying rocks to weathering.
Figure 2: Location and accessibility map of the study area.
3.0 Materials and Methods

Visual inspection and reconnaissance survey was undertaken in the course of the fieldwork to evaluate and assess the physical conditions of the highway pavements. The underlying rocks were identified and their structural trends were noted. Twenty disturbed soil sample was collected using an auger from the sub grade, sub base and base course. The materials were taken across the road sides closed by the failed road and distressed sections in a manner that the soil samples reflect the different topographic state of the road arrangement. The samples were kept safe and packed in an air-tight sack bag to retain its natural moisture. Marks were placed on the collected samples to indicate soil descriptions, sampling depths and dates of sampling. The samples collected was sent to the Lagos state material testing Laboratory. The samples were spread on different matting to ease air drying and all the clods and lumps in the samples were broken down and reduced to fine particles before been subjected to geotechnical test and carried according to code specifications (ASTM D6913) for Sieve analysis, (ASTM D4318) Atterberg’s Consistency Limit tests, (ASTM D854) Specific Gravity, (ASTM D698) Compaction, and (BS1377) California Bearing Ratio (CBR) tests. The detailed methods of these geotechnical analysis are highlighted in (Shirsavkar, 2010; Pumnia et.al.2005; Arora 2009; PhaniKumar 2004; Mir and Sirdharan 2013; Al-Rawas 2011).

Geochemical and mineralogical analysis of eight selected soil samples, were carried out through the use of X-ray diffraction and Florescence methods using the techniques of (Carrol D 1971) and the clays minerals were recognized and the percentage abundance was calculated using the
area method (International Joint Committee Properties on Mineral Powder Diffraction Standard 1980).

The Dutch Cone Penetrometer Test (CPT) The Dutch cone penetrometer test was used by driving hardened steel cone uninterruptedly into the ground and by taken its measurement of its resistance to penetration. The CPT machine has a peak angle of 60º and a hinge area of 10cm², it comprises of a steel frame conveying a driving head which accommodate a hydraulic pressure capsule and the driving head is lifted or let down by a manually controlled winch. The cone body is forced into the ground by using the steel rods that is fastened to the driving head. These rods are shielded from friction with the soil by an empty outer rods. The cone driving rods and outer rods were forced at the same time into the ground for an interval of 200mm. The operated pressure is then exerted to the inner rods wholly and the cone is moved forward separately of the outer rods for an interval of about 40mm at the standard of nearly 100mm/sec. The force required to progress the cone is transferred across the capsule in the driving head to a gauge and the penetration resistance indicated on the gauge is recorded. The outer tube is then advanced and the whole assembly is driven a further 250mm where the operation is repeated (Ubido et al.;2017). Four cone penetration test (CPT)) were carried out along the profiles of the geophysical survey sampling in-situ testing and describing soils were in accordance with the British Standard Code of Practice for Site Investigations, B.S 5930:1999, and with Method of Testing Soils for Civil Engineering Purposes, B.S 1377:1990(Ubido et al ;2017,2018).

Electrical Resistivity Imaging (ERI) is a geo-electrical techniques used to acquire high level resolution 2D image of the earth surface. The ERI was implemented during the investigation, using Wenner array electrical configuration. The data gathered from the field was ascribed into the software RES2DINV to create a 2D resistivity image of the earth subsurface under study. The result acquired from the study were plotted in form of a pseudo-section which gives an estimated near picture of the subsurface attribute. These results acquired are used to derive the qualitative exposition of the profiles. Near subsurface resistivity contrast at shallow depth of electrode spacing of 10m is directed at probing the lateral depth variation in the electrical properties of the profiles subsurface. Stretching the Wenner array spacing to 25m, 35m, 45 m, 55m, 65 m, 75 m, 85m, 95m, 105 m etc. which is distinguished by its responsiveness for its vertical variation in the subsurface resistivity beneath the midpoint of the array. The materials used include ABEM Terra meter SAS 1000, measuring tape, masking tape, hammers electric, cable steel electrode, RES2DINV software, Global Positioning Satellite (GPS). Traverse I, II III and IV were set -up along the road profile laterally that showed serious cracks and babbling of the pavement. The electrode spread (Wenner array configuration) for data collection along the four profiles surveyed, used contrasting electrode spacing of 10m, 20m, 30m and 40m accordingly. A total length of 250m, 250m, 250m and 600m was surveyed for profile I, II, III and IV exclusively. Four horizontal profiles were set up which had a length of 250m each and the last 600m for convenience of space due to vehicular movement was appraised to delineate the geo-electric properties of the section of the road in the study area.

3.1 Results and Discussion
The results of the geotechnical test like sieve analysis, Atterberg’s Consistency Limit tests, Specific Gravity, Compaction, and California Bearing Ratio (CBR) tests as shown in Table 2a and b.

Table 2a: Results of the geotechnical test.

| S/no | UE/1 | UE/2 | UE/3 | UE/4 | UE/5 | UE/6 | UE/7 | UE/8 | UE/9 | UE/10 |
|------|------|------|------|------|------|------|------|------|------|-------|
| Soil layer | SBC | SBC | SBC | SBC | SBC | SBC | BC | BC | BC | BC |
| % clays | 26.1 | 18.5 | 39 | 31.3 | 36.7 | 27 | 26.5 | 26.8 | 26.2 | 48.9 |
| % silt | 37.2 | 12.78 | 9.6 | 7.4 | 11.9 | 11.9 | 16.4 | 15 | 11.7 | 12.1 |
| % fines | 62.8 | 30.2 | 47.2 | 39.5 | 45.4 | 37.4 | 41.9 | 39.8 | 36.1 | 58.0 |
| %sand | 36.0 | 63.4 | 49.4 | 48.3 | 56.8 | 57.3 | 49.3 | 61.3 | 37 | 31.7 |
| %grav el | 3.2 | 7.7 | 4.3 | 13.2 | 3.8 | 9.3 | 10.8 | 2.7 | 25.9 | 9.3 |
| mc % | 6.8 | 12.6 | 20.6 | 14 | 16.4 | 19.7 | 14.9 | 7.5 | 15 | 15.4 |
| FS (%) | 33 | 22 | 21.2 | 21.4 | 32 | 26.9 | 43.9 | 2.0 | 18.6 | 22 |
| LS (%) | 6 | 8.16 | 11 | 12.4 | 11 | 12.4 | 10.2 | 6 | 6 | 8.29 |
| Spec gravity | 2.64 | 2.52 | 2.53 | 2.53 | 2.52 | 2.53 | 2.54 | 2.56 | 2.57 | 2.62 |
| liquid limit | 56.1 | 30.4 | 49.7 | 46.4 | 46.9 | 47.9 | 52.2 | 25.1 | 41.7 | 42.4 |
| Plastic limit | 35.6 | 22.4 | 26.2 | 32.4 | 29.9 | 27.5 | 29.2 | 19.9 | 29.7 | 30.5 |
| PI | 20.5 | 8.0 | 23.5 | 14.0 | 17.0 | 20.4 | 23.0 | 5.20 | 12 | 11.9 |
| Activity | 0.78 | 0.43 | 0.60 | 0.44 | 0.46 | 0.75 | 0.86 | 0.19 | 0.45 | 0.24 |
| MDD (g/cm³) | 1.52 | 1.65 | 1.67 | 1.51 | 1.61 | 1.53 | 1.65 | 1.78 | 1.67 | 1.65 |
| OMC | 24 | 15 | 21 | 18 | 21 | 17 | 12 | 11 | 16 | 18 |
Table 2b: Results of the geotechnical test.

| S/no | UE/11 | UE/12 | UE/13 | UE/14 | UE/15 | UE/16 | UE/17 | UE/18 | UE/19 | UE/20 |
|------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
|      | SGS   | SGS   | SGS   | SGS   | SGS   | SGS   | SGS   | SGS   | SGS   | SGS   |
| % clays | 24.2  | 21.1  | 26.9  | 27    | 13.4  | 12.1  | 22.6  | 8.5   | 21.8  | 27.5  |
| % silt  | 11.2  | 12.7  | 6.1   | 14.9  | 15.8  | 14.9  | 8.7   | 7.5   | 14.8  | 14    |
| % fines | 34.4  | 31.8  | 32    | 42.9  | 27.2  | 28    | 28.3  | 12    | 35.6  | 38.5  |
| %sand  | 56.2  | 60.5  | 59.4  | 51.4  | 64    | 70.1  | 50.5  | 80.1  | 55.7  | 49.4  |
| %grav el | 8.4   | 6.7   | 10.6  | 4.7   | 9.8   | 4.9   | 20.2  | 6.9   | 11.7  | 10.1  |
| mc %   | 15.8  | 12.6  | 12.2  | 18.2  | 11.5  | 13.2  | 22    | 12.5  | 11.6  | 9.4   |
| FS (%) | 31    | 8.14  | 26    | 26.7  | 11    | 5.35  | 5.17  | 6.17  | 31    | 26    |
| LS (%) | 9.57  | 10.29 | 11.29 | 11.6  | 8.86  | 3.86  | 3.96  | 7.43  | 11.71 | 12.71 |
| Specfic gravity | 2.56 | 2.55 | 2.58 | 2.56 | 2.57 | 2.55 | 2.54 | 2.57 | 2.50 | 2.58 |
|----------------|------|------|------|------|------|------|------|------|------|------|
| liquid limit   | 34.3 | 36.3 | 33.6 | 40.5 | 26.8 | 36.6 | 31.3 | 40.81 | 45.4 | 46.2 |
| Plastic limit  | 21.7 | 25.2 | 22.6 | 22.1 | 18.2 | 18.7 | 20.5 | 22.3 | 23.8 | 21.6 |
| PI             | 7.1  | 11.1 | 11   | 18.4 | 8.6  | 17.9 | 10.8 | 18.51 | 21.6 | 24.6 |
| Activity       | 0.29 | 0.52 | 0.40 | 0.68 | 0.64 | 1.47 | 0.47 | 2.17 | 0.99 | 0.89 |
| MDD (g/cm³)    | 1.70 | 1.72 | 1.71 | 1.69 | 1.74 | 1.70 | 1.72 | 1.72 | 1.64 | 1.64 |
| OMC            | 16   | 15   | 13   | 13.5 | 14   | 12   | 15   | 16   | 17   | 16   |
| CBR %          | 3    | 4    | 12   | 5    | 4    | 4    | 5    | 9    | 3    | 4    |
| USCS           | CL   | CI   | CL   | CI   | CL   | CI   | CL   | CI   | CI   |     |
| AASHT O        | A-26 | A-26 | A-26 | A-7  | A-24 | A-2  | A-24 | A-2  | A-2  | A-7  |
| Soil group     | CS   | CS   | CS   | CS   | CS   | SiC  | CS   | CS   | CS   | CS   |

Note: MC Moisture content, SG specific gravity, MDD maximum dry density, PI plasticity index, LS linear shrinkage, SS sandy silt, CS clayey sand, GS gravelly sand, SC sandy clay, SiC silty clay, NP non plastic ; SBC sub-base soil; Base course (BC) materials; sub-grade soil (SGS).

Table 2a and b shows the result of the natural moisture content of the investigated soil; the natural moisture content plays an important role in increasing or reducing density indices of soils which will differ based on the depth of the soil, previous rainfall data and the existing drainage conditions (Ramamurthy and Sitharam, 2005, Bowles, 1984); from the result of the study the moisture content is in the range of 7.5% and 15.4% for base course between 6.8% and 19.7% for sub-base course and between 9.4 and 18.20% for subgrade. These values are fairly high considering the time of the test, indicating the soil potential for water retention, property of fine grain soils. The moisture content is one of the component that affect the dry density of soils, (Garber and Hoel, 1999).The periodic rainfall fluctuations in the moisture contents of soil will result to a large volume changes in the clayey soils. Underwood (1967) stated that soils with natural moisture content of 5-15% are appropriate engineering materials where soils with natural moisture content values varying from 20-30% are unsuitable engineering materials. The results obtained for the moisture content of the soil samples is in the range of 6.8-19.7 which in comparison with the (underwood, 1967) position shows that only 9 (nine) samples have adverse to slightly unfavourable values of moisture.
The grain size distribution is shown in Tables 2a and b and Figs. 4a and b give the summary for the soil grain size curve for soil classification. The grain size is important to the strength of the soil. A well graded soil means there are particles of various sizes present in the soil. The prevalent grain sizes ranges from sandy silt, silty clayey to clayey sand. The clay fragment feasibly exerts a dominant control on the mass behavior even though when available in any soil for use in engineering. The tiny smaller particles filled up with voids produced by more massively huge particles resulting to stability and smaller pores for water to fill. From the studied locations the soils are classified as A-2-6, A-6, and A-7 according to AASHTO classification (1978) the soils in these group are considered as clayey soils which are classified as fair to poor materials for road use and soils classified as A-2-7 soil on AASHTO soil classification is regarded as poorly graded, poor graded(GP) on USCS soil classification with group index of 0 which is of silty, clayey gravel and sand material (Gopal and Rao 2011). Also, most of the soil samples had a very high percentage finer than 0.0075 fraction that is >35%, which varied between, 31.7% and 61.3% for base course and 30.2% and 62.8% for sub base and 12% and 42.9% for sub grade respectively. The result of the study shows that that 60% did not meet the requirement for subgrade soil while 40% meet the condition when compared with (underwood, 1967) which states that subgrade soils have less than 35% fines.
Figure 4: Stacked grain size distribution for soil samples.

**Figure 4a:** Stacked grain size distribution for the soil samples UE1-10

**Figure 4b:** Stacked grain size distribution for the soil samples UE11-20
Tables 2a and b and Fig. 5 shows the consistency limits of the studied soil. From the result of the study the liquid limit is in the range of 25.1% and 52.2%, Plastic limit in the range of 19.9%-30.5% and plasticity index in the range of 5.20-23.5 for sub-base course whereas the liquid limit is in the range of 30.4% and 56.1%, Plastic limit in the range of 22.4%-35.6% and plasticity index in the range of 8.0-23.5 for sub-base course and liquid limit is in the range of 26.8% and 46.2%, Plastic limit in the range of 18.2%-25.2% and plasticity index in the range of 7.1-24.6% for subgrade. The liquid limit values range from 20.1 to 55.1%. The requirements for roads suggest that for a material to be acceptable as subgrade materials it should have a liquid limit less than 40%. 40 percent of the studied samples did not meet this requirement whereas 60% of the studied samples met the specification. (Ola 1983) stated that soils with plasticity index that is lower than 25% shows low to medium swelling potential. The results of the study were plotted on the (Cassagrande, 1947) plasticity chart (Fig. 5) Which shows that 64% of the soils were plotted within the field of inorganic silts while 36% were plotted in the field of inorganic clays. Only samples 2, 11, 13, 15, 16, and 14 possess low plasticity while the rest falls within the field of medium plasticity while the rest falls within the field of medium to high plasticity Table 3 and this satisfies the condition that the studied area have medium to highly plastic soil. Additionally when the plasticity index lies between 20 and 35% then it satisfies the condition for high swelling potential and between 25 and 41% state for a high degree of expansion (Gopal and Rao, 2011). It is the predominant factor in the selection of materials for subgrade and subbase. It gives much detailed facts on the properties and behavior of clays in contrast to grain size data (Lambe, 1951). Excessive plasticity frequently results to fluctuations which arises from plastic flow upon the application of axle load (Adeyemi 1995, 2002), soils with extremely high liquid limit are often prone to have low bearing capacity. (Cassagrande, 1947) stated that soil samples with low, medium and high plasticity will have low, medium and high compressibility together. It was observed that the subgrade, sub base and base course in most of the locations had liquid limit (LL) higher than the recommended value (<35%) Table 4 and the Plasticity index greater than the specified value (12%) Table 3 while the subgrade soils attained prescribed value of (</ 80) and PI of (< 55%).
Figure 5: Plasticity chart of the studied soil samples (Casagrande, 1974).

| Table 3: Expansive soil classification based on plasticity index |
|---------------------------------------------------------------|
| Swell potential       | Plasticity index (%) | Holtz and Gibbs (1956) | Chen (1988) | IS: 1498 (1970) |
| Low                   | <15                  | 0–15                    | <12         |
| Medium                | 15–28                | 10–35                   | 12–23       |
| High                  | 25–41                | 20–55                   | 23–32       |

Table 4: Expansive soil classification based on liquid limit

| Swell potential       | Liquid limit (%)   | Chen (1965)           |
|-----------------------|--------------------|-----------------------|
| Low                   | <30                | Snethan et al. (1977) |
| Medium/marginal       | 30–40              | IS: 1498 (1970)       |
| High                  | 40–60              | 50–60                 |
| Very high             | >60                | 70–90                 |
Tables 2a and b shows the linear shrinkage values of the soil from 3.96 to 12.71 (Brink et al, 1982; Ola, 1983) reported that soils with linear shrinkage values exceeding 8% will be active and have a serious swelling potential and are not fit for foundation materials Table 5 and Table 6. (Gidigasu ,1973) reported that soils having linear shrinkage value greater than 10% will constitute a field compaction problem, from the result of the study 50% of the soil samples will constitute a field compaction problems.

Table 5: Expansive soil classification based on shrinkage limit (Holtz and Gibbs 1956)

| Swell potential | Shrinkage limit (%) |
|-----------------|---------------------|
| Low             | >15                 |
| Medium          | 10–16               |
| High            | 7–12                |
| Very high       | <11                 |

Table 6: Expansive soil classification based on shrinkage limit (Altmeyer 1956)

| Volume change   | Shrinkage limit (%) |
|-----------------|---------------------|
| Non-critical    | >12                 |
| Marginal        | 10–12               |
| Critical        | <10                 |

The specific gravity results are summarized in (Tables 2a and b), the values of the specific gravity of the studied soil range from 2.52-2.64. The specific gravity is known to link the strength of soil and are in turn used as a criteria for selecting an appropriate materials for highway pavement construction materials especially when used along with other pavement materials and low specific gravity are connected with clay mineralogy and weathering of feldspar that gave rise to the clay (Okogbue, 1988; Owoyemi and Adeyemi 2012). The studied soils is said to be free of organic matter as soils with organic matter have specific gravity values less than 2.0.

Activity of the soil results are summarized in (Tables 2a and b and Figure 6). The result from the study was gotten from the approach of (Skempton, 1953) clay content and Atterberg limit into an exclusive parameter. (Skempton, 1953) proposed three classes of clay based on their activity specifically; the normal clays with activity values ranging between 0.75 and 1.25, 1.25 and the active clays with activity values greater than 1.25 and the inactive clays with activity values less than 0.75 Table 7. The activity values from the studied soil ranges from 0.19- 2.17 which indicate active to inactive clays; consequently the studied soils have low – medium and negligible high expansion ability as gotten in the activity chart (Fig 6).

Table 7: Expansive soil classification based on the activity

| Activity (A c) | Nature of the soil | Probable degree of swell |
|----------------|--------------------|--------------------------|
|                |                    |                          |
The free swells results are summarized in (Tables 2a and b), the values of the free swell index of the studied soil range from 5.17 – 43.9; the soils fall in the range of kaolinite and Illite. Consequently, they have low to modest swelling potential Table 8(Onana, 2017; Aghamelu and Okogbue 2011).

Table 8: Expansive soil classification based on shrinkage index (IS 1498)

| Degree of expansivity/swell potential | Shrinkage index (%) |
|-------------------------------------|---------------------|
| Low                                 | <15                 |
| Medium                              | 15–30               |
| High                                | 30–60               |
| Very high                           | >60                 |

Results of the compaction test (Tables 2a and b, and Figs 7a and b) showed that Maximum dry densities (MDD) and Optimum Moisture content; The OMC varied between 12% and 18%, 15% and 24%, 12% and 17% for base course, sub base and sub grade respectively in all the locations similarity, the MDD is quite very low for base course, sub base and sub grade soil which varied between 1.65 and 1.78g/cm³, 1.51 and 1.67g/cm³, 1.64 and 1.74g/cm³ respectively. The MDD of base course at every location is less than the specified value. (i.e.>/ 2.0g/cm³) and MDD of subbase is as well less than the specified value of >/2.0g/m³ in most location and the MDD of sub grade is less than the specified value of >/ 1760g/m³. According to Nigeria general
specification (1994) recommends that soil should be in the ranges of 1.50 to 1.78 g/m$^3$ for the MDD and optimum moisture content (OMC) should range from of 8.56-12.02%. Based on these results Underwood (1967) states that the soil samples have fair to poor foundation attribute. This comparative low maximum dry density value is potentially a responsible factor to the persistence incidence of road failure.
Figure 7: Stacked compaction curves for samples UE1-20

Fig 7a: Stacked compaction curve for samples UE1-10.

Fig 7b: Stacked compaction curve for samples UE11-20.
Figure 8: Stacked CBR curves for samples UE 1-20.
(Tables 2a and b, and Figs 8a and b) shows the result of CBR (%). The CBR values for location varied between 3% and 5% for base course material, 4% and 8% for sub base course and 3%-12% for base course. These results fell below the maximum of 80% recommended by the (FMWH, 1997). The CBR value of sub grade materials are meet in some of the location with the stated specification value of >/5%. Moreover the implication of the CBR and compaction analysis is that failure may not be certainly due to the frail sub base and sub grade other than where compaction is in not sufficient. The results shows that the materials used for real construction of the exiting road (base course) indicate that it was of inferior quality. (FMWH, 1997) proposed that a California Bearing Ratio (CBR) greater than 10% for subgrade materials. The results that the CBR values of some soils are lower than the recommended value. The corresponding minimum CBR ratio values for the soils in that were studied is partly accountable for the failure of high way pavement that was studied.

Geochemical and mineralogical properties

Table 9; shows the mineralogies of the selected eight samples. The prominent clay mineral types Obtained are quartz, Illite, Kaolinite and Dickite. There was no presence of montmorillonite and smectite which is accordance with was reported by (Okogbue, 1988) that montmorillonite is not present in the south western Nigerian soils studied. From the Table 3and 4 shows the area is of low to medium plasticity which confirms with the presence of Kaolinite as the major mineral due to its minimal affinity for water which makes the study area to be well drained and the occurrence of Muscovite may result to field compaction issue (Gidigasu, 1976; Ogunsanwo, 1988; Onana e tal, 2007; Paige-Green 2003) Which can be seen from the low values of the maximum dry density gotten from the studied soils.

The result of geochemical analysis is presented on Table 10. The main total chemical elements, in decreasing order, were SiO₂, Al₂O₃, and Fe₂O₃, with averages of 44.77%, 21.9%, and 21.60%, respectively. The studied soil samples are characterized by high amounts of silica from 40.54% to 53.98%; appreciable amount of sesquioxides (Al₂O₃ and Fe₂O₃) which varied from 17.10% to 29.5% for Al₂O₃, and 2.87% to 29.41% for Fe₂O₃, reasonable amount of bases (K₂O and CaO) the other chemical element concentrations were all lower than 5% (Table 10).The silica(S)/sesquioxides ratios(R) (Table 10) range from 0.81 to 1.67.(BS 1377 1990 and Onana e tal, 2007) classified soils based on the silica/sesquioxides ratio as laterite if the is less than 1.33; lateritic if ratio ranges from 1.33 to 2 and non-lateritic if it is greater than 2. According to this classification, 5%of the studied soils classify as non-lateritic soils while the rest 95%are lateritic.

Table 9: The mineralogies of the selected eight soil samples

|        | UE/2 | UE/7  | UE/9  | UE/10 | UE/11 | UE/13 | UE/17 | UE/20 |
|--------|------|-------|-------|-------|-------|-------|-------|-------|
| Quartz (%) | 78   | 85.6  | 47    | 58    | 70    | 45    | 65    | 35    |
| Kaolinite | –    | 12.14 | 13    | 42    | –     | –     | –     | –     |
Table 10: Result of geochemical analysis of selected eight soil samples

| % Oxide | UE/2  | UE/7  | UE/9  | UE/10 | UE/11 | UE/13 | UE/17 | UE/20 |
|---------|-------|-------|-------|-------|-------|-------|-------|-------|
| SiO₂    | 41.30 | 41.00 | 53.98 | 43.50 | 47.47 | 44.25 | 46.14 | 40.54 |
| Al₂O₃   | 21.19 | 22.99 | 29.50 | 22.20 | 21.40 | 17.10 | 23.60 | 17.00 |
| Fe₂O₃   | 29.41 | 25.61 | 2.87  | 24.30 | 23.43 | 16.44 | 29.41 |       |
| CaO     | 0.44  | 0.62  | 0.14  | 0.19  | 1.04  | 0.84  | 0.80  | 0.68  |
| Na₂O    | 0.16  | 0.11  | 2.17  | 0.51  | 0.41  | 0.26  | 0.18  | 0.41  |
| K₂O     | 3.25  | 3.40  | 9.86  | 4.25  | 4.67  | 5.62  | 4.86  | 5.60  |
| MgO     | 0.32  | 1.59  | 0.17  | 0.73  | 0.72  | 0.77  | 0.50  | 0.72  |
| TiO₂    | 2.64  | 1.75  | 0.18  | 2.93  | 1.85  | 2.16  | 1.26  | 2.50  |
| MnO     | 0.10  | 0.05  | 0.06  | 0.04  | 0.14  | 0.10  | 0.06  | 0.04  |
| ZnO     | 0.03  | 0.04  | 0.01  | 0.01  | 0.02  | 0.01  | 0.02  | 0.01  |
| CuO     | 0.02  | 0.01  | 0.01  | 0.03  | 0.01  | 0.02  | 0.04  | 0.01  |
| Total   | 98.86 | 97.17 | 98.95 | 98.69 | 98.08 | 94.56 | 93.9  | 96.92 |
| S/R     | 0.81  | 0.84  | 1.66  | 0.93  | 1.13  | 1.09  | 1.15  | 0.87  |

4.1. Cone Penetrometer Test

The Cone Penetration Test (CPT) data collected are presented in graphs (Figs.9, 10, 11 and 12) and a summary of locations and depths is available in Table 11. It was carried out to determine the strength of near surface materials and also access the in situ strength of the subsoil tested. The graphs of the cone penetrometer reading are presented as penetration rate against depth in (Fig.12, 13, 14, and 15). The results revealed low cone resistance value which indicates clayey sand material for sample point 1, 2, 3 and 4. The linear natures of the graph of sample point 3 and 4 shows the subsurface materials offer no resistance to the driven cone unlike in the case sample point 1. The results on the CPT test for sample points indicates that the depth range of 2-16m penetrated which is unfit for erecting the foundation and subgrade material which therefore make soil to swell and eventually collapse under imposed wheel load stress which leads to failure. The low value in shear strength also make the materials undesirable as subgrade materials.
materials for road pavement which will continually expand and contract under different weather condition.

Table 11. Results of CPT for the studied locations.

| Location | CPT point 1 | CPT point 2 | CPT point 3 | CPT point 4 |
|----------|-------------|-------------|-------------|-------------|
| Test Hole | CPT 1       | CPT 2       | CPT 3       | CPT 4       |
| Depth of Penetration (m) | 2 | 6 | 16 | 12.5 |
| Cone Resistance (kgf/cm²) | 20 | 45 | 138 | 72 |
| Undrained shear strength (Cu)KN/m² | 10 | 7.50 | 8.63 | 5.76 |
| Allowable bearing capacityKN/m² | 17.13 | 12.85 | 14.77 | 9.86 |
| Ultimate bearing capacity | 50 | 37.5 | 43.125 | 28.5 |
| Remarks | Dark grey, stiff medium grained clayey sand | Dark grey, medium grained clayey sand | Dark grey, stiff, fine-medium grained Clayey sand | Dark grey, stiff, fine-medium grained Clayey sand |

Using the established standard used by Lagos State Material Laboratory test for CPTs in foundation engineering, “Simplified Description of the use and Design Methods”(Ubido et al, 2017, 2018)

Undrained shear strength (Cu)

\[
Cu = \frac{qc}{Nk}
\]  
Where \(qc\) = cone end resistance value
\(Nk\) = Point of refusal or termination point

Allowable Bearing Capacity

Ultimate bearing capacity is \(5.14 \times Cu\), where \(5.14\) is constant

\[
\text{Allowable bearing capacity} = \frac{5.14 \times Cu}{3}
\]
Figure 9: showing CPT plot for point 1 along profile 1

Figure 10: showing CPT plot for point 2 along profile 2
Figure 11: showing CPT plot for point 3 along profile 3

Figure 12: showing CPT plot for point 4 along profile 4
Geophysics
Fig. 13(a) represents the Pseudo-section from the apparent resistivity measurement along profile A. It revealed a resistivity value in the range of 1110 Ωm – 1576Ωm, between two major segments at distances ranging from 10 – 90 m and 110 m – 170 m along the profile to a depth of 2.50m – 11.02m and 2.50 m – 15m respectively. The arrows show sections along the profiles with low resistivity. Fig. 13(b) shows the representation of the Pseudo-section produced from the calculated apparent resistivity values showing a resistivity values similar to the results obtained along the profile in Fig. 13(a). Fig.13(c) is the inverse model of the 2D plot which reveals the resistivity sections along the profile. Two distinctive zones were observed with low resistivity values along profile A, which occurs at the distance between 122.5 m – 130.0 m along profile A to a depth of 5m from the top soil were observed. Also at distances between 200 m – 235 m, slightly moderate resistivity values ranging from 1200 Ωm – 1600 Ωm to a depth of 18 m from the surface of the profile was revealed.
Figure 13: Resistivity model for profile A.

Figure 13a: Measured apparent resistivity of profile A.

Figure 13b: Calculated apparent resistivity of profile A.

Figure 13c: Inverse model resistivity section of profile A.
Figure 14: Resistivity model for profile B.

Fig. 14(a) represents the Pseudo-section from the apparent resistivity measurement along profile B. It shows a low resistivity value ranging from 100 $\Omega m$ – 300 $\Omega m$, between distances of 20 m – 240 m along the profile to a depth of 7.60 m from the surface of the profile. Fig. 14(b) shows the representation of the Pseudo-section produced from the calculated apparent resistivity values showing relatively low resistivity values in the same region along the profile similar to the position of the result obtained in Fig. 14(a). Fig 14(c) is the inverse model of the 2D plot which reveals the resistivity sections along the profile B. Different sections from the model along the profile shows a low resistivity values with few area along the profile having resistivity greater than 200 $\Omega m$. Two distinct sections along the profiles with very low resistivity values were
noticed at distances ranging from 10 m – 45 m, 60 m – 110 m and 180 m – 235 m to a depth of 7.60 m, 2.80 m and 5.60 m respectively.

**Figure 15**: Resistivity model for profile C.

Fig. 15(a) represents the Pseudo-section from the apparent resistivity measurement along profile C. It model revealed a low resistivity value ranging from 50 Ωm – 111Ωm, between distances of 80 m – 120 m along the profile to a depth of 15 m from the surface of the profile. Fig. 15(b) shows the representation of the Pseudo-section produced from the calculated apparent resistivity values; it shows a low resistivity values in the same section along the profile similar to the position of the result obtained in fig. 15(a). Fig. 15(c) is the inverse model of the 2D plot which reveals the resistivity sections along the profile C. The major sections along the profile shows low resistivity values. Two distinctive zone with very low resistivity values were observed at distances ranging from 40.0 m – 140 m, 190.0 m – 210.0 m to a depth of 21 m and 25 m respectively.
Fig. 16(a) represents the Pseudo-section from the apparent resistivity measurement along profile D. It shows a low resistivity value in the range from 100 Ωm – 350Ωm, at a distances of 10 m – 500 m along the profile to a depth of 13m from the surface of the profile. Fig. 16(b) shows the representation of the Pseudo-section produced from the calculated apparent resistivity values, it shows relatively low resistivity values along the profile similar to the position of the result obtained in fig. 16(a). Fig 16(c) the inverse model of the 2D plot which reveals the resistivity sections along the profile D. A number of sections along the profile show slow resistivity values to depth of 15m.
Conclusion

It can be concluded from the geotechnical, mineralogical, geochemical and geophysical investigations carried out

1. Based on AASHTO classification of soils 20% of the soils classify as A-2-4 whereas others outstanding 80% are classify as A-2-6, A-5 and A-7 which are soils with fair to poor subgrade attribute.
2. Lack of drainage in the study area lead to a decrease in strength of the subgrade as a cause of water inflow and sufficient drainage system should be provided for the road to avoid the long time exclusion of water on road pavement as this incapacitate the molecular force holding the particles of the pavement jointly.
3. The low specific gravity values also contributed to the road failure as increase in specific gravity indicates minimal void ratio.
4. The high shrinkage values of are the cause of the shrinkage complication resulting in difficulties in field compaction
5. The low CBR results in the study area is a significant cause of failure in the study area
6. Low resistivity values were observed in profile 2 and 3 ranging from 100 $\Omega$m – 300 $\Omega$m, between distances of 20 m – 240 m along the profile to a depth of 7.60 m from the surface of the profile from the topsoil along the profile 2. In profile 3. It shows a low resistivity value ranging from 50 $\Omega$m – 111$\Omega$m, between distances of 80 m – 120 m along the profile to a depth of 15m from the surface of the profile the topsoil along the profile.
7. Low resistivity values below 200 $\Omega$m was obtained in some regions of profile 2 and 3 and 4, which indicates the presence of incompetent material beneath the subsurface of the road pavement. These clay materials are unfit as subgrade materials for road pavement, as it regularly expand and contract under varying weather condition. The frequent expansion and contraction of these subgrade material leads to cracks on the road pavement and eventual failure of the road. In addition moderately high resistivity values greater 200 $\Omega$m were observed along the profile 1 shows the presence existence of laterite material which is an acceptable subgrade material for road pavement.
8. Other contributing factors such as inadequate drainage system, paucity of maintenance, poor pavement coating of the road, and substandard construction materials are factors leading to a total failure of the road. Sections of the road with sandy and clayey materials should be scooped out from the subsurface to a depth of 3m – 5 m from the top soil of the road and put back with competent fill materials.
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