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RESEARCH ARTICLE

GEOENGINEERING INVESTIGATION OF AN EROSION INDUCED HIGHWAY STRUCTURAL FAILURE ALONG IFON – BENIN HIGHWAY, SOUTHWESTERN NIGERIA

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ABSTRACT

Road infrastructural is one of the most important economic indices for development of a country. Therefore in-situ cone penetration test and laboratory soil analysis were performed at two failed segments along Ifon-Benin highway, with the aim of determining cause(s) and extent of the failure. The cone penetration test was carried out to a depth of about 20 m with a lateral spacing of 20 m along the roadway. The laboratory tests conducted were grain size analysis, Atterberg limit test, compaction test, California Bearing Ratio, and undrained unconfined triaxial test. The results revealed that all the soil parameters fell short of the federal ministry of works and housing specification of Nigeria, with plasticity index (>20%), % fines (>35%), CBR values (<80%) recommended, angle of friction and cohesion are less than minimum standard of 30° and 50 Kpa respectively. The CPT revealed predominant sandy silt to clayey silt topsoil and clay substratum with compressive strength of 20 – 40 KN/m^2. The water level is higher than 3 m, consequently far below the road foundation baseline. Findings showed that the upper 6 m of the failed segments has been seriously affected by erosion and flooding. Subsequently the process resulted into excessive settlement of the silt/clayey-subgrade soil underneath the pavement structure. This makes the highway to settle largely under traffic load. In addition, incessant heavy flooding around the embankment/shoulder of the highway might have induced the failure, leading to looseness, and less-cohesion of the layers which invariably reduces subgrade support and weakens various pavement layers.

KEYWORDS

Bearing capacity, Erosion, Geologic section, Highway, Penetrative resistance.

1. INTRODUCTION

Road transportation is an important element in the physical development of any society as it controls the direction and extent of development (Daramola et al., 2018). Necessity of highway infrastructure development has been one of the most important indices in measuring growth of nation's economy. Consequently it must be given all attention it deserves (Falowo and Dahunsi, 2020). A country's economic status depends upon how well served the country is, by its roads, railways, air ports, ports, pipelines and shipping (Obaje, 2017; Ilori, 2015; Ajani, 2006; Aghamelu and Okogbue, 2011). The rate at which a country's economy grows is very closely linked to the rate at which the transport sector grows (Kadyali and Lal, 2008). Transport minimizes the time for the movement of people and goods, thus transport gives utility to economic activities and facilitates quick economic development.

Road structure consists of number of layers (subgrade, subbase, base course, wearing surface), each of which has a particular function. The wearing surface of a modern road consists bituminous bound aggregate or a concrete slab, although a bituminous surfacing may overlie a concrete base. A concrete slab distributes the load that the road has to carry, whereas in a bituminous road, the load primarily is distributed by the base beneath. The base and sub-base are below the surface course and generally made of granular material, although in heavy-duty roads, the base may be treated with cement. The subgrade refers to the soil immediately beneath the sub-base. However much the load is distributed by the layers above, the subgrade has to carry the load of the road structure plus that of the traffic. Consequently, the top of the subgrade may have to be strengthened by compaction or stabilization (Hartley, 1974; Black and Lister, 1978). The strength of the subgrade, however, does not remain the same throughout its life. Changes in its strength are brought about by changes in its moisture content, by repeated wheel loading, and in some parts of the world by frost action. Although the soil in the subgrade exists above the water table and beneath a sealed surface, this does not stop the ingress of water. As a consequence, partially saturated or saturated conditions can exist in the soil. Also, road pavements are constructed at a level where the subgrade is affected by wetting and drying, which may lead to swelling and shrinkage, respectively, if the subgrade consists of expansive clay (US Army Corps of Engineer, 2004).

The highway embankment is also important element of pavement structure must also have sufficient bearing capacity to prevent foundation failure and also be capable of preventing excess settlements due to the imposed load (FWHA, 2006; Hadjigeorgiou et al., 2006). It should be

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designed as to protect the subgrade from getting damaged due to capillary rise in groundwater, provide ready drainage of surface water, ensure the needed geometric design standards, possess stable slopes and must not undergo excessive settlement. Subgrade strength being a vital parameter in pavement design, must be accompanied with adequately designed and constructed embankment. It is interesting to know that most highways in Nigeria lack surface drainage (which deals with arrangements for quickly and effectively leading away the water that collects on the surface of pavements, shoulders, slopes of embankments and cuts and the land adjoining the highways). Degradation of many highway pavements are traceable to the surface water ingress through cracks and joints in the embankment and shoulder of the highway (Ozegin et al., 2011; Momoh et al., 2008; Daramola et al., 2018). Studies of past road failures showed some major causes of pavement failure, vary from geology and geomorphology; usage; poor design and construction problems; use of substandard material for road construction; poor maintenance culture; poor highway facilities such as drainages; poor workmanship and supervision; and non-compliance to local standard of practice.

Highway failure can be classified as either functional or structural. Functional failure is caused by traffic loading and movement over a road, and topography. On the other hand, structural failure results from elemental deterioration of the road materials in response to climatic and hydrogeochemical changes. Structural failure may also result from differential settlement, poor road design and shear failure (Oni and Olorunfemi, 2016). In Nigeria, incessant occurrence of road pavement failure/collapse has attracted the interests of many professionals involved in road design and construction. The blame-game between these professionals and government at all levels has not contributed positively to this ugly menace in the transport sector. However it's true that most design engineer and construction engineer always compromise design calculations and or models due to cost consideration, unforeseen problematic terrain/site condition. Most of the roads that were initially designed to carry heavy duty vehicles/large axle load are compromised in standard on site, hence most Nigeria roads is increasingly becoming unsafe with spectacular increase of traffic plying the roads. Even most of the road users are impatient as "they" all want to avoid potholes, rutting, cracks, pitting on most of the failed portions of the highways at the same time, leading to dangerous overtaking/maneuvers resulting in accident, loss of lives and properties.

In view of this background, the attention of the researchers was drawn to a road failure along Ifon – Benin highway, which connects two States of Ondo and Edo together. The failure occurred at two spots on the highway in the same form. Many accidents had occurred along these failed segments leading to loss of lives and properties, especially for road users who prefer driving at night. Consequently the Edo and Ondo State governments sealed those spots and prohibited vehicular movement along the failed portions, and also by providing alternative roadway. This was to prevent further accidents and encroachment of any form in the trouble spots. In fact, at first, the prohibition restrained the researchers from probing into causes of the failure, until permission (for the investigation) was sought from the government. This type of highway failure is very common on Nigeria roads most especially in the sedimentary environment where excessive flooding, weathering, and erosion constantly erode the soft foundation soil (Jegede and Olaleye, 2013; Jegede, 2000; Ngah and Nwankwoala, 2013). The states of the two failed segments have made travelling along the road very unpleasant, armed robbers and kidnappers/unscrupulous people have made the failed portions their abode, where criminal and nefarious activities are carried out daily. This has consequently denied the area of its viable economic activities, with attendant pollution and environmental degradation. Hence a necessity for this research.

The engineering behaviour of near surface sub-tropical and tropical soils and rocks is a function of the impact of their interaction with the road environment and the weathering processes (Anochie-Boateng et al., 2010; George and Uddin, 2006). Such characteristics may be attributed to their composition, the nature of their pore fluids, their mineralogy or their fabric (Paige-Green, 2003). Therefore the focus of this research is to determine the cause of the failure (subsidence) of the failed segments along the highway. The objectives of study are to identify soil nature and establish the extent in depth of failure along the road alignment; determine the extent of dissolution of the subgrade material; assess the geotechnical properties of the subsoil within the failed and stable segments based both on field and laboratory tests, and its influence on the highway collapse; and proffer reasonable engineering re-construction/rehabilitation method or approach. The study incorporates in-situ cone penetration test (CPT) and laboratory geotechnical soil analysis (Adeoti et al., 2016; Roy and Bhalla, 2017). These methods have proved their effectiveness in foundation studies and road failure investigation (Cetin and Ozan, 2009; Jung et al., 2008; Kurup and Griffin, 2006; Mayne, 2007; Moss at al., 2006; Rogers, 2006; Oyedele and Okoh, 2011; Olorode et al., 2016). CPT soundings can be very effective in site characterization (Oyedele and Olorode, 2010; Olayanju et al., 2017; Coker et al., 2013; Coker, 2015; Ayolabi et al., 2012.). In addition, this study would assists in unraveling the cause of failure and determination of engineering properties of soil to ensure proper feasible re-design and successful re-construction of the failed segments of the highway structure.

2. MATERIALS AND METHODS

2.1 Study Area

The studied highway cut across two States of Nigeria, namely Ondo State and Edo State in southwestern and south-south regions respectively (Fig. 1). Two major segments of the highway had failed sometimes in year 2016 (Figs. 2 and 3) without due rehabilitation/reconstruction. The first segment is located in Ifon (Ose Local Government) while the second segment is in Owan area of Edo State (Fig. 3). The pavement failures at both failed segments manifested as local subsidence, potholes, depressions of diverse dimensions, disintegration, blow-up, sub-grade intrusion, surface stripping, cracking and sliding. The study area is underlain by sedimentary Benin and Ewekoro Formation which has been described severally in Akujieze (2004), Akujieze and Oteze (2006). Drifts and soil-cover characterized the formation over lateritic reddish brown clayey sand capping highly porous friable white sands, pebbly sands and clay stringers with basal indurated ferruginous pebbly - coarse grained sandstone. The Benin Formation is poorly bedded and occasionally cross - bedded at greater depths (Ntom, 2001; Omatsola and Adegoke, 1981). However the local geology of the studied sites are shale/mudstone. Sedimentary rocks differ according to their sedimentary environments. The mechanical properties of the sedimentary strata depend on the sediment facies characteristics. Shale and mudstone are soft rock whose strength in terms of cohesion are usually reduced due to high (pore) water pressure, occasion by erosion/flooding.

The investigated road fall within the coastal region of Nigeria and the elevation is of low lying with elevation of not more than 100 m above the sea level. The Nigeria coastal zone is within the tropical climate areas. The rainy season is April to November and dry season is in December to march. The area has an annual rainfall varies between 1,500 and 4,000mm (Ibe, 1988). Temperature in the coastal area is modulated by the cloud cover and by the damp air. However, the mean monthly temperature vary between 24°C and 32°C. Mangrove and rain forests characterized the vegetation of the study area.

2.2 Fieldwork, Sampling and Geotechnical Investigation

Geoengineering investigation is devoted to the study and solution of the engineering and environmental problems which may arise as a result of the interaction between geology and the works and activities of man, as well to remediation of geological hazards. Rock/soil physical and mechanical properties are very important parameters for geological engineering design and construction (Meng et al., 2002). Therefore for this study, after thorough reconnaissance and desk studies, the cone penetrometer test (CPT) using Dutch cone penetrometer (Cetin and Ozan, 2009; Jung et al., 2008; Kurup and Griffin, 2006) and geotechnical laboratory analysis of soil samples underlying the highway at different depths were employed for the in-situ subsoil characterization. The cone penetration test is economical and supplies continuous records with depth. The failed segments were distinguish into segments 1 and 2, representing Ifon-Sobe and Owan-Uyere respectively.

The test was performed at ten points straddling both the failed and stable segments along the highway (Fig. 4). The tests were carried out to a depth of about 20 m with a lateral spacing of 20 m. The length of the failed segments in each location was about 70 m. Most of the test reached refusal before the anchors pulled out of the subsurface (Oladunjoye et al., 2017). The layer sequences were interpreted from the variation of the values of the cone resistance with depth (Oyeyemi et al., 2017). In order to constrain or control the interpretation of the test, a control stable segment was established at each segments outside the failed portions. From the series of recorded gauge readings, cone resistance and sleeve friction were plotted against depth. On the basis of the expected resistance contrast between the various layers, inflection points of the penetrometer curves were interpreted as the interface between the different lithologies (Roy and Bhalla, 2017). The graph generated was interpreted using Robertson's soil classification chart (1990) as shown in Fig. 5; and Das (2000) soil classification using cone resistance value shown in Table 1.

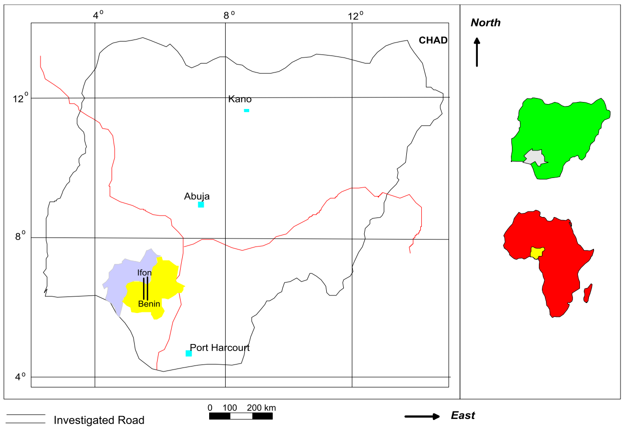


Figure 1: Map showing the Location of the Study Area on Nigeria and Africa maps

Figure 2: Failed Segment 1 at Ifon in Ondo State, showing (a) collapse pavement structure (b) eroded foundation soil and destroyed buried cables and utilities by erosion

Figure 3: Collapsed Pavement Structure along Owan – Uyere Highway, representing Failed Segment 2

2.3 Laboratory Analysis

Eight disturbed soil samples were collected into polythene bags from foundation soil materials from borrow pits/trenches along the roadsides adjacent to both the failed and stable segments through boring/trial pit using hand auger. The disturbed soil samples were collected at regular intervals of about 0.5 m to maximum depths of 3 m and 1 m at locations 1 and 2 respectively. All samples were examined, identified, classified roughly (BS 1377, 1990) in the field and all data and information carefully recorded before the conduct of laboratory tests. However the natural moisture content of each of the samples was determined within twenty four hours upon collection using ASTM D 2216-92 specification.

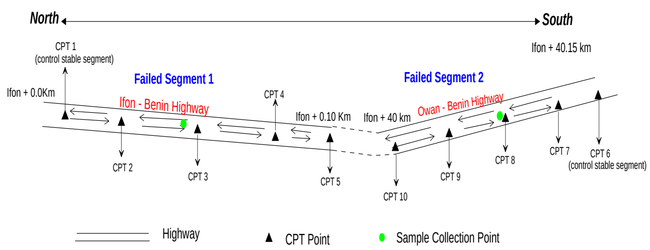


Figure 4: Sketch of the field layout of the CPT Points and trial pit sampling points along investigated failed segments of the Highway

The other tests carried out include grain size analysis (ASTM D 422-90), specific gravity (ASTM D 854-92), consistency (Atterberg) limits (ASTM D 4318-95), compaction test (BS 1377, 1990), undrained unconfined triaxial test (ASTM D 2166-91) and California bearing ratio (ASTM D 1883-94) in accordance to procedures specified by the American Standard for Material Testing for civil engineering purposes.

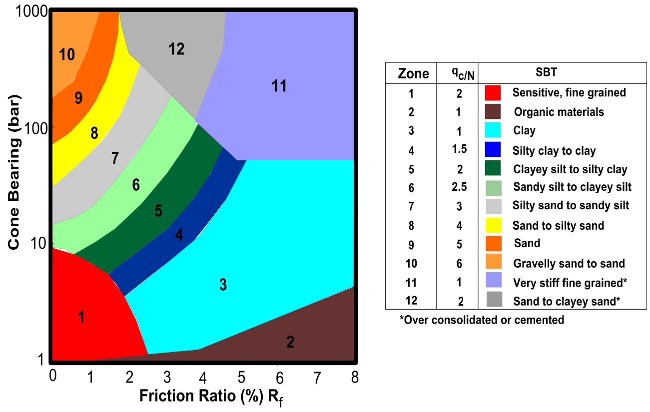


Figure 5: Robertson Chart for Soil Classification using cone resistance value and friction ratio (after Robertson, 1990)

Table 1: Cone resistance value for a corresponding lithology of cohesive and granular soils (Das, 2000)

Cone resistance value (kg/cm²)	Soil Type	Inferred CU values (KN/m²)	Cone resistance value (kg/cm²)	Relative Density
0-4	Very soft clay	20	20-40	Very loose to loose
0-6	Soft clay	20-40	40-120	Medium Dense
6-10	Firm clay	40-75	120-200	Dense
10-20	Stiff clay	75-100	Above 200	Very Dense
Above 20	Very stiff clay to Hard clay	100-150 And 150		

3. Results

The results of the geotechnical tests are presented in Tables 2 and 3 for locations 1 and 2 respectively. At location 1, the natural moisture content (NMC) at the depth of 1 - 3 m ranged from 16.5 to 21.6 % with an average of 19.3 %, and at location 2, it ranged from 22 to 23.2 % with an average (avg.) of 22.9 %. The particle size analysis of the soil involves determining the percentage by mass of particles within the samped soil. Therefore subsoil at depths 1 to 3 m have % sand ranging from 14.2 – 53.0 (avg. 31.4), % fines is in between 47.0 – 85.8 (avg. 68.6 %) at location 1. At location 2, the % sand varies from 11.6 – 48.7 (avg. 30.15 %), % fines ranges from 51.3 – 88.4 (avg. 69.85 %). The specific gravity values in the study area ranged from 2.53 to 2.69 in location 1 and 2.66 to 2.67 in location 2. The Liquid Limit of the subsoil at the depth of 1 to 3 m at location 1 ranges from 54.1 to 65.8% and an average of 57.0 %. At location 2, it varies from 55.2 % to 56.3 % (avg. of 55.8 %). The plastic limit (PL) of the subsoil at location 1 and 2 ranges from 17.5 to 32.0 % (avg. of 26.7 %) and 25.1 to 28.3 % (avg. of 26.7 %) respectively. The range of plasticity index obtained from locations 1 and 2 ranged from 21.2 to 48.33 % (avg. of 33.8 %) and 28.1 to 30.1 % (avg. of 29.1 %) respectively. The maximum dry density for the soil samples varied between 1540 Kg/m^3 and 1670 Kg/m^3 (avg. 1610 Kg/m^3) at location 1, and 1650 - 1720 Kg/m^3 (avg. 1685 Kg/m^3) at location 2, while their optimum moisture content ranged between 21.2 % - 29.6 % (avg. 25.3 %) and 21.9 – 23.1 % (avg. 22.5 %) respectively. The California bearing ratio (CBR) of the subsoil at locations 1 and 2 ranged from 24 to 60 % (avg. of 42.1 %) and 24.6 to 29.3 % (avg. of 26.9 %) respectively. The shear strength parameters of the soil samples in terms of angle of friction and cohesion for locations 1 and 2 vary between 17 – 26° (avg. 22°) and 22 – 28 ° (avg. 25°); and 19 – 32 Kpa (avg. 24.7 Kpa) and 26 – 34 Kpa (avg. 30 Kpa) respectively.

The plots of cone resistance, sleeve resistance, and friction ratio against depth for stable segment and failed segments at location 1 are shown in

Figures 9 and 10. The control stable segment along this highway at location 1 (CPT 1) comprised five major geologic units found at different depths: silty sand to sandy silt (0 – 2 m), clay/clay silt to silty clay (2 – 6.5 m), sand to silty sand (6.5 – 9.0 m), clay (9 – 12.0 m), silty sand to sandy silt (12 – 16.5 m) and sand (16.5 – 18.75 m). The cone resistance obtained from upper 3 m, 3 – 10 m and 10 – 18.25 m varied from 10 – 48 kg/cm² (firm to stiff clay), 12 - 99 kg/cm², and 45 – 186 kg/cm², with corresponding sleeve resistance of 15 - 62 kg/cm², 62 – 114 kg/cm², and 114 - 248 kg/cm², while the friction ratio ranged from 0.62 – 5.33, 0.86 – 5.23, and 1.24 – 4.25 respectively.

CPT 2 is characterized by sandy silt to clayey silt/silty sand to sandy silt (0 – 2 m), clay (2.25 – 11.5), clayey silt to silty clay (11.5 – 13.25 m), clay (13.25 – 14.75 m) and sandy silt to clayey silt (14.75 – 18.25 m). The cone resistance, sleeve resistance and friction ratio values obtained from upper 3 m, and 10 – 18.25 m varied from 3 – 33 kg/cm² and 12 - 101 kg/cm²; 5 – 36 kg/cm² and 52 - 190 kg/cm²; and 0.51 – 5.12 and 1.65 – 5.87 respectively. In addition, the upper 6 m has been seriously affected by the erosion resulting into low resistance observed on the graph (Figure 9a). The geologic units under CPT 3 at location 1 consist of silty sand to sandy silt (0 – 2 m), clay (2 – 10.5 m), and silty sand to sandy silt/clay (10.5 – 18.25 m). The cone resistance obtained from upper 3 m, 3 – 10 m and 10 – 18.25 m vary from 6 – 33 kg/cm² (firm clay/very loose sand), 4 - 13 kg/cm² (soft/firm clay), and 11 – 129 kg/cm² (stiff clay/medium-dense sand silt), with corresponding sleeve resistance of 7 - 36 kg/cm², 18 – 59 kg/cm², and 52 - 195 kg/cm², while the friction ratio ranged from 0.40 – 4.52, 4.25 – 5.98, and 1.11 – 5.21 respectively.

Table 2: Summary of the Results of Subsoil Geotechnical tests for Location 1: Ifon – Sobe Failed Segment

Depth (m)	NMC (%)	Grain size Analysis % Gravel	% Sand	% Fines	Specific Gravity	Atterberg Limits Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)	Compaction MDD (Kg/cm²)	OMC (%)	Un-soaked CBR (%)	UU Triaxial Test Angle friction (°)	Cohesion (KPa)	USCS Group	AASHTO Group
0.5	16.5	-	22.0	78.0	2.67	65.80	17.47	48.33	1540	27.10	39.23	26	32	CH	A-7-6
1.0	18.3	-	53.0	47.0	2.69	52.14	22.69	29.45	1590	29.55	24.15	24	25	CI	A-7
1.5	20.3	-	14.2	85.8	2.60	52.77	22.58	30.19	1640	22.81	44.36	22	19	CI	A-7-6
2.0	20.4	-	23.0	77.0	2.53	53.16	32.00	21.16	1670	22.45	49.21	17	28	ML	A-7
2.5	19.5	-	43.7	56.3	2.57	54.71	18.62	36.09	1650	21.20	59.68	22	19	CI	A-7-6
3.0	21.6	-	27.9	72.1	2.55	59.68	24.29	35.39	1580	28.30	36.72	25	23	CI	A-7-6

Table 3: Summary of the Results of Subsoil Geotechnical tests for Location 2: Owan – Uyere Failed Segment

Depth (m)	NMC (%)	Grain size Analysis % Gravel	% Sand	% Fines	Specific Gravity	Atterberg Limits Liquid limit (%)	Plastic limit (%)	Plasticity Index (%)	Compaction MDD (Kg/cm²)	OMC (%)	Un-soaked CBR (%)	UU Triaxial Test Angle friction (°)	Cohesion (KPa)	USCS Group	AASHTO Group
0.5	23.2	-	11.6	88.4	2.66	55.23	25.11	30.12	1650	23.12	24.59	22	26	CI	A-7
1.0	22.5	-	48.7	51.3	2.67	56.30	28.25	28.05	1720	21.85	29.31	28	34	CI	A-7

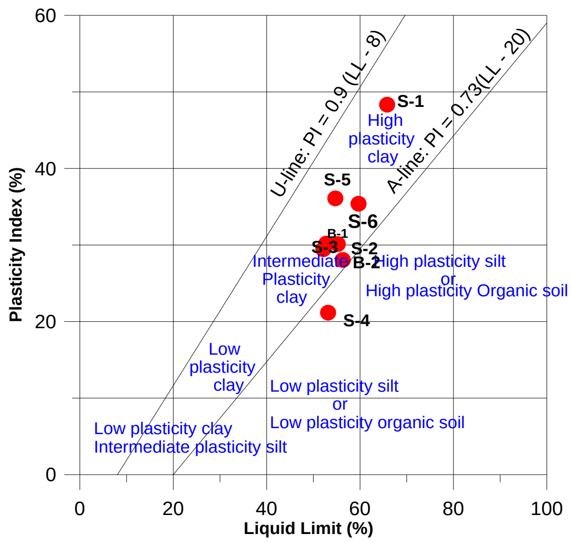


Figure 6: Casangrande Plasticity Chart for the Studied Soil Samples showing predominant intermediate plasticity clay

	Granulary Materials (35 % or less passing No. 200)							Silt-Clay Materials (More 35 % or less passing No. 200)			
General Classification	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve analysis, percentage passing:											
No. 10 (2.00 mm)	50max	....	....	....	....	....	....	....	....	....	....
No. 40 (0.425 mm)	30max	50max	51max	....	....	....	....	....	....	....	....
No. 200 (0.075 mm)	15max	25max	10max	35max	35max	35max	35max	35max	36min	36min	36min
Characteristics of fraction passing No. 40 (0.425 mm)											
Liquid Limit	....	....	....	40max	40min	40max	41min	40max	41min	40max	41min
Plasticity Index	6max		N.P	10max	10max	11min	11min	10max	10max	11min	11min
Usual types of significant Materials	Stone fragments Gravel and Sand		Fine Sand	Silty - Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as subgrade	Excellent to Good							Fair to Poor			

Figure 7: AASHTO Soil Classification System for Civil Engineering Highway Construction

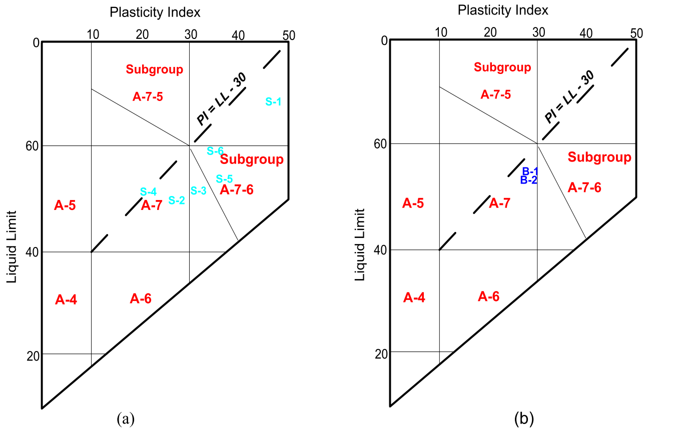


Figure 8: AASHTO soil classification Chart for A-7 Subgroup for study sites

CPT 4 is underlain by four geologic sequence, comprising clayey silt to silty sand (0 – 1.75 m), clay (1.75 – 11.0 m), sandy silt to clayey silt (11.0 – 15.75 m) and silty sand to sandy silt (15.75 – 18.25 m). The cone resistance values at upper 3 m, 3 – 10 m, and 10 – 18.25 m varied from 4 – 15 kg/cm², 3 – 15 kg/cm² (very soft – stiff clay with inferred unconfined compressive strength varying from 20 – 80 KN/m²), and 12 - 102 kg/cm² (stiff clay/loose – medium dense sand silt), with sleeve friction values ranging from 15 - 32 kg/cm², 14 - 62 kg/cm², and 68 - 188 kg/cm² respectively. The friction ratio obtained was in between 2.14 – 5.11 (upper 3 m), 4.02 – 5.69 (3 -10 m depth), and 1.23 – 5.12 (10 – 18.25 m depth).

At CPT 5, the geologic units comprised four geologic units, which are sandy silt to clayey silt (0 – 2 m), clay (2.0 – 7.25 m), clayey silt to silty clay (7.25 – 12 m) and sand silt to clayey silt/silty sand to sandy silt (12 – 18 m). The cone resistance obtained from upper 3 m, 3 – 10 m and 10 – 18.25 m varied from 3 – 40 kg/cm², 3 - 33 kg/cm² (very soft – very stiff clay, with inferred CU of 15 to >20 KN/m²), and 9 – 151 kg/cm², with corresponding sleeve resistance of 14 - 25 kg/cm², 17 – 56 kg/cm², and 48 - 201 kg/cm², while the friction ratio ranged from 0.45 – 5.13, 1.54 – 5.36, and 1.02 – 5.69 respectively.

At failed segment 2, the plots of cone resistance, sleeve resistance, and friction ratio against depth for stable segment (CPT 6) and failed segments (CPT 7 – 10) are shown in Figures 11 and 12. The control stable segment along this highway comprised five major geologic units found at different depths with clay and silt material alternation; clayey silt to silty clay (0 – 4.25 m), clay (4.25 – 8.0 m), clayey silt to silty clay/sand silt to clayey silt (8 – 14.75 m), clay (14.75 – 17.5 m), silty sand to sandy silt (17.5 – 18.75 m) and clay (18.75 – 20 m). The cone resistance values obtained from upper 3 m, 3 – 10 m and 10 – 18.25 m ranged from 3 – 25 kg/cm² (very soft clay/loose silt sand), 10 - 49 kg/cm², and 36 – 180 kg/cm², with corresponding sleeve resistance of 5 - 52 kg/cm², 52 – 136 kg/cm², and 136 - 240 kg/cm², while the friction ratio ranged from 1.51 – 2.31, 2.15 – 5.48, and 1.25 – 5.36 respectively.

The CPT 7 is characterized by sensitive soil/silty clay to clay (0 – 2.75 m), clay/sandy silt to clayey silt (2.75 – 10.5), clayey silt to silty clay (10.5 – 12.0 m), silty sand to sandy silt/sandy silt to clayey silt/clayey silt to silty clay/ silty clay to clay (12 – 16 m) and clay (18 – 20 m). The cone resistance, sleeve resistance and friction ratio ranged from 3 – 21 kg/cm², 4 - 15 kg/cm², and 0.67 – 2.93 for depth 0 – 3 m; 2 - 29 kg/cm², 9 - 98 kg/cm², and 2.36 – 5.36 for depth between 3 – 10 m; 14 - 129 kg/cm², 86 - 198 kg/cm², and 1.25 – 5.69 for depth between 10 – 18.25 m. At CPT 8, the geologic units consist of sensitive fine grained soil (0 – 2.5 m), clay (2.5 – 11 m), silty clay to clay/clayey silt to silty clay (11 – 16 m), clay (16 – 18.75 m), and sandy silt to clayey silt (18.75 – 20 m). The cone resistance obtained from upper 3 m, 3 – 10 m and 10 – 18.25 m varied from 1 – 22 kg/cm² (soft - firm clay with inferred CU values of 20 – 150 KN/m²), 2 - 17 kg/cm² (very soft/firm clay), and 14 – 84 kg/cm², with corresponding sleeve resistance of 3 - 15 kg/cm², 9 – 88 kg/cm², and 74 - 205 kg/cm², while the friction ratio ranged from 0.55 – 5.21, 2.33 – 5.55, and 1.99 – 5.36 respectively.

CPT 9 is underlain by three major geologic sequence, comprising sandy silt to clayey silt (0 – 2.75 m), clay (2.75 – 11.25 m), clayey silt to silty clay (11.75 – 15 m) and clay (15 – 20 m). The cone resistance values at upper 3 m, 3 – 10 m, and 10 – 18.25 m ranged from 5 – 19 kg/cm², 2 – 17 kg/cm² (very soft – stiff clay with inferred CU varying from 20 – 80 KN/m²), and 14 - 58 kg/cm² (stiff clay - medium dense sandy silt), with sleeve friction values ranging from 5 - 19 kg/cm², 12 - 89 kg/cm², and 76 - 201 kg/cm² respectively. The friction ratio obtained varied from 0.62 – 3.98 (upper 3 m), 1.1 – 5.69 (3 -10 m depth), and 1.88 – 5.68 (10 – 20 m depth). At CPT 10, the soil profile depicted four geologic sequence consisting of sandy silt to clayey silt/silty sand to sandy silt (0 – 4.25 m), clay/silty clay to clay (4.25 – 17.5 m), sandy silt to clayey silt (17.5 – 18.5 m) and clay (18 – 20 m). The cone resistance, sleeve resistance and friction ratio ranged from 3 – 29 kg/cm², 13 - 51 kg/cm², and 0.51 – 1.25 at depth 0 – 3 m; 2 - 25 kg/cm², 19 - 82 kg/cm², and 1.1 – 5.69 for depth between 3 – 10 m; 21 - 91 kg/cm², 89 - 223 kg/cm², and 1.88 – 5.68.

4. Discussion

At both localities, the moisture content values are generally high, and may be due to the fact that, the samples were taken during raining season. The highest moisture content (with average >20%) was obtained at location 2.

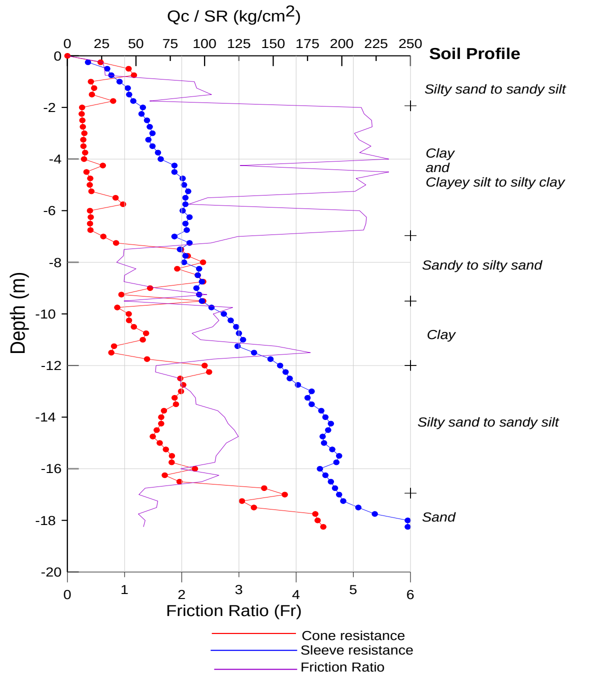
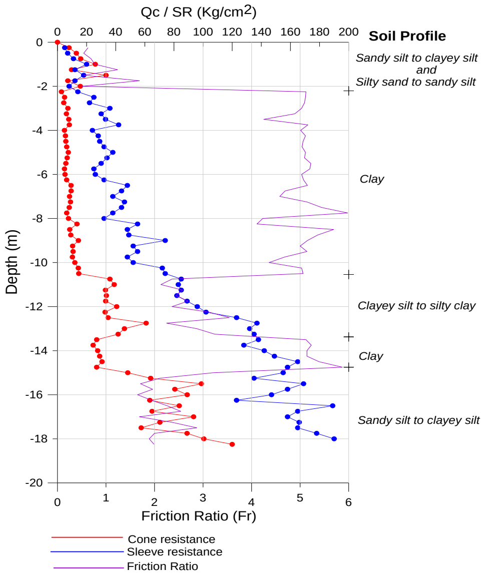


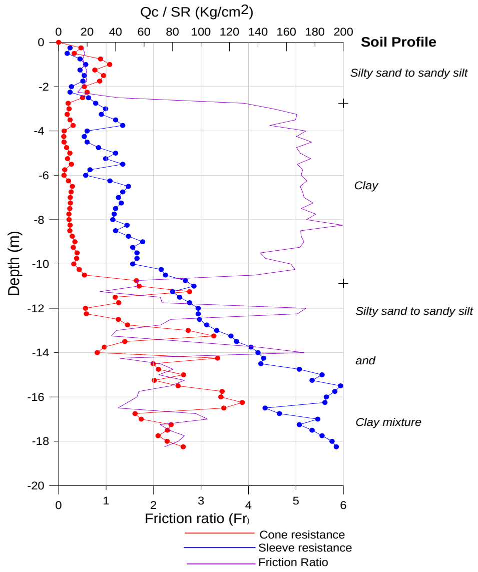
Figure 9: Plots of Cone resistance, Sleeve resistance, and friction ratio against depth at the stable segment of location 1

The subsoil within the upper 3 m is predominantly composed of sandy clay and sandy silt. All the soil samples generally contains % fines higher than maximum of 35 recommended by Federal Ministry of Works and Housing of Nigeria (FMWH, 1997) for a good subsoil material for civil engineering construction purpose. This correlates with the geology of the study area which is shale. The specific gravity of soil increases with increasing compaction and decreasing porosity and compressibility. The specific gravity of the soil depends on the amount of sand and also depends on their mineral constituents and mode of formation of the soil (Akintorinwa and Oluwole, 2018). Therefore soils with high specific gravity have high load bearing pressure and strength. The obtained specific values fall short of minimum of 2.70 recommended for subgrade material by Federal Ministry of Works and Housing of Nigeria (FMWH, 1997).

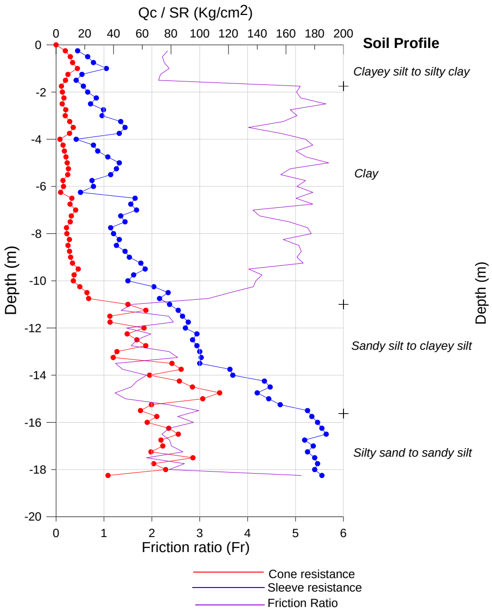
High liquid limit values (above 50 %) are indicative of poor engineering and geological properties of subgrade soils (Jegede, 1995; Ngah and Nwankwoala). Liquid Limit of 50% maximum (FMWH, 1997) is recommended for sub-grade material for engineering constructions, and most of the soil samples fall within this value. Therefore the soil can be said to be incompetent to support the pavement structure under different vehicular loadings. Generally, soils having high values of plastic limits are considered as poor foundation materials (Akintorinwa and Adeusi, 2009). FMWH (1997) recommended plasticity index of 20% maximum for subgrade material for engineering constructions. Plasticity is an important characteristics in the case of fine soils, it describes the ability of a soil to undergo unrecoverable deformation without cracking or crumbling. Plasticity is due to presence of a significant content of clay minerals (or organic material) in the soil. The void spaces between such particles are generally very small in size with the result that water is held at negative pressure by capillary tension. This produces a degree of cohesion between the particles, allowing the soil to be deformed or remolded. Any decrease in water content results in a decrease in cation layer thickness and an increase in the net attractive forces between particles.



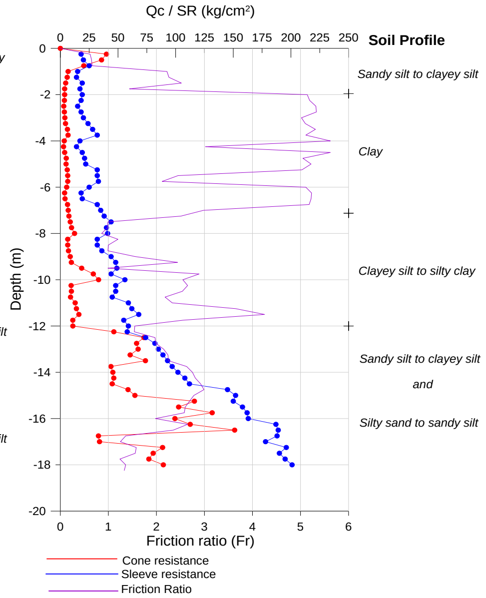
(a)



(b)



(c)



(d)

Figure 10: The Plots of CPT Values against Depth and their corresponding interpreted geologic sections along failed segment 1 (a) CPT 2 (b) CPT 3 (c) CPT 4, and (d) CPT 5

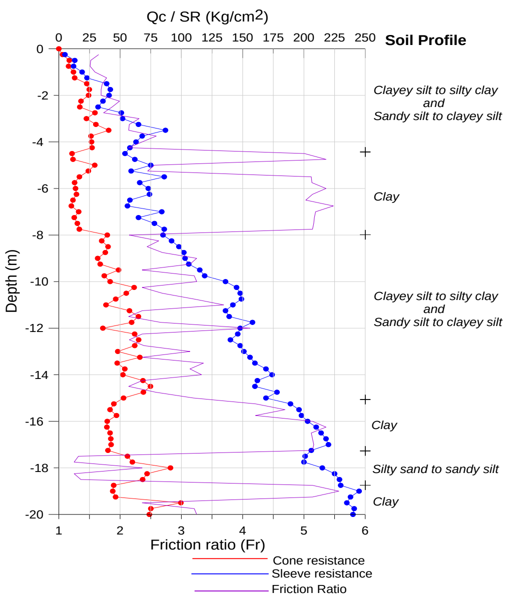


Figure 11: Plots of Cone resistance, Sleeve resistance, and friction ratio against depth at the stable segment of location 2 (CPT 6)

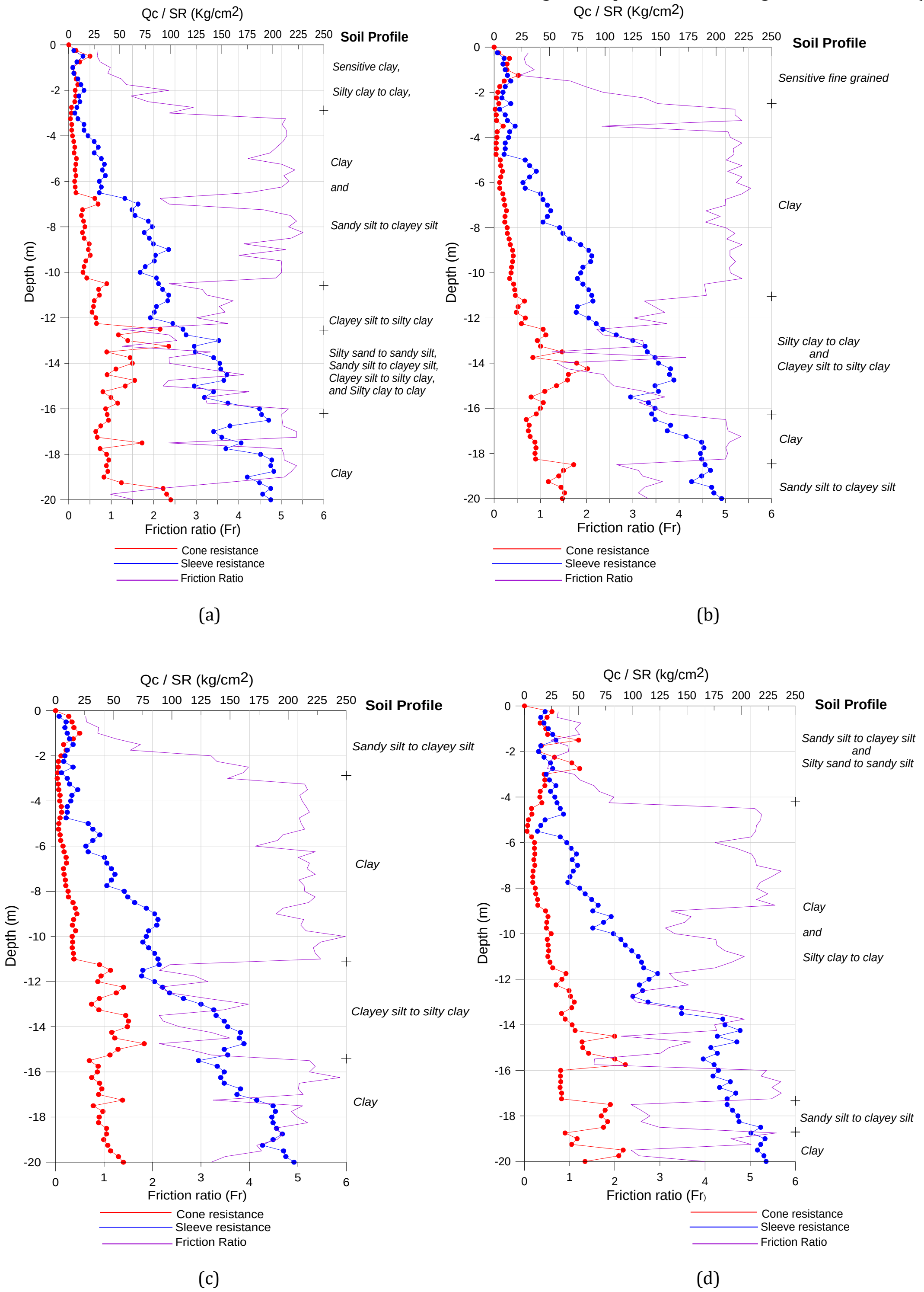


Figure 12: The Plots of CPT Values against Depth and their corresponding interpreted geologic sections along failed segment 2 (a) CPT 7 (b) CPT 8 (c) CPT 9 (d) CPT 10

The plasticity index of the soil samples fall out of the recommended range. According to Casangrande (1972) such soil can be said to be of high compressibility and plasticity and are therefore unsuitable for civil engineering construction.

However the Casangrande plasticity chart (Casangrande, 1947) in Figure 6 shows Intermediate plasticity clay (CI) and high plasticity clay (CH) for most of the sampled soils, except S-4 taken from location 1 that shows intermediate plasticity silt (MI). This also corroborates the American Association of State Highway and Transportation Officials (1982) in Figure 7 which rates the samples as fair to poor i.e. A-7/A-7-6 in location 1 and A-7 in location 2 (Figure 8). The MDD is the density of the soil at which any further increase in the moisture content leads to a reduction in the unit weight of the soil. The importance of compaction test is to improve the desirable load bearing capacity properties of a soil (Akintorinwa and Adeusi, 2009). The degree of compaction is sensitive to moisture content.

The best subsoil for foundation of engineering structures is that with high Maximum Dry Density (MDD) at low OMC (Jegede, 1999). Such soil is characterized by high shear strength and low compressibility. All the studied samples have MDD at high OMC. However samples exhibit higher MDD (greater than 1800 Kg/m^3) and lower OMC (less than 20%) are good as foundation soil for road and building structures. These values show that the soils would respond slowly to compaction. At low values of water content must soils tend to be stiff and are difficult to compact. As the water content is increased, the soil becomes more workable facilitating compaction and resulting in higher dry densities. At high water contents, however, the dry density decreases with increasing water content, an increasing properties of the soil volume being occupied by water. Thus, a higher compactive effort results in a higher value of maximum dry density and a lower value of optimum water content.

California Bearing Ratio (CBR) is a test designed to assess the strength of soil especially for highway construction. The standard specification of un-soaked CBR recommended according to FMWH (1997) is 80 % minimum for subgrade materials for road construction in Nigeria. The CBR test results has shown that none of the soil samples tested has CBR value up to 80% minimum recommended for road construction. Therefore soil showing higher shear strength parameters would be less susceptible to rupture or failure along any plane inside it. But all the soil samples have values less than minimum friction angle of 30° and minimum cohesion of 50 Kpa specification. It is observed from the CPT plots, that there is generally increase of cone resistance and sleeve resistance with depth except where there is an intercalation of clay. Consequently the erosion that leads to the failure of the road pavement and its shoulder midly permeate into the stable segment due to its degree of cohesion/compactness, which could be reason for its relatively high cone resistance and sleeve friction values obtained with the depths of 3 – 10 m, which seems to be the problematic layer seriously affected by the erosion. It is observed that the clay substratum under CPT 2 at depth 3 – 10 m, is characterized by low cone resistance values ranging from 3 – 10 kg/cm2 (soft clay, with inferred CU values ranging from 20 – 40 KN/m^2) in comparison to the stable segment (12 – 99 kg/cm^2 – stiff to very stiff clay, with an inferred compressive strength (CU) values varying from 100 to >150 KN/m^2), with corresponding sleeve resistance and friction ratio values of 22 – 74 kg/cm^2 and 4.23 – 5.20 respectively.

Consequently the clayey soil under this point is very soft/firm due to erosion, leading to high porosity and permeability resulting in high compressibility (reduction of soil strength and bearing capacity) and settlement under vehicular load (Archana et al., 2016). Therefore loose sand is cohesionless, peat and clay are more susceptible to subsidence and are also affected by environmental conditions such as temperature and moisture, which vary irregularly throughout the year (Rahim et al., 2016). The graph also depicts low resistance values to about 11.0 depth, after which it begins to increase. Consequently the clayey silt to silty sand and clay soil show less competence to carry imposed vehicular load. Even though the upper 10 m of CPT 5 shows relatively increase values of cone resistance and sleeve friction than what is obtainable under CPT 2 – 4, it can be noted that the clayey substratum (to a depth of 9.25 m) is still a challenge. Therefore the failure along this segment could have been precipitated by incompetent soil material with considerable thickness of 8 m composing of clayey/silty material. The low penetrative resistance values (generally less than 10 kg/cm2 and corresponding to soft clay with unconfined compressive strength of less than 75 KN/m^2) and their looseness could be as a result of incessant erosion along this segment which have penetrated deeply to about 15 m. Consequently considering weak geotechnical characteristics of the soil mass above 10 m, there's need for it to be excavated during future rehabilitation/re-construction of the highway and fill with competent soil material like laterite or graded with sand/gravel. Although this may not guarantee the stability of this segment after rehabilitation/construction since an underground drainage system is required, and at the same time along the shoulders of the road.

At failed segment 2, it is observed that there is generally increase of cone resistance and sleeve resistance with depth. Also clayey soil material intercalations generally have cone resistance values greater than 25 kg/cm^2 corresponding to very stiff/hard clay with inferred compressive strength of 100 – 150 KN/m^2. In addition the topsoil composed of heterogeneous clayey silt to silty clay and sandy silt to clayey silt with relatively high cone and sleeve resistance in the range of 5 – 30 kg/cm^2 and 10 – 50 kg/cm^2 respectively. Consequently the silt – clay mixture could be responsible for relative stability of this segment. At CPT 7, the upper 6.5 m is characterized by low cone resistance values corresponding to inferred compressive strength of 20 – 40 KN/m^2 signifying very soft to soft clayey/silty material. This could responsible for the failure at this point, since at this depth the stable segment is characterized by high cone resistance values ranging from 5 - 38 kg/cm^2, symptomatic of soft – hard clayey/silty soil material. Therefore the upper 18.75 m is clayey with very low cone resistance values, suggestive of soft-firm clay with unconfined compressive strength less than 75 KN/m^2. CPT 9 graph also depicts low resistance values from 4 to 10 m depth, after which it increases.

CPT 10 is predominantly made of fines (clay and silt), although at depth of 0 – 4.25, it contains small amount of sand. This could the major reason why less settlement/subsidence occurred at this location. Subsequently, the failure at this segment could have been induced by incessant heavy flooding around the embankment/shoulder of the highway. This flood water has infiltrated the subgrade material leading to looseness, and less-cohesiveness of the layer. Erosion control is an integral part of highway drainage. Excessive erosion may plug culverts, leading to saturation of the pavement structure and failure of drainage systems, and finally resulting to pavement failure. Environmental considerations that dictate erosion must be controlled. Surface drainage must be provided to drain precipitation away from the pavement structure (Gidigazu, 1983). In a simple example, cross slope directs water to the shoulder where it flows into a ditch, then down the ditch to a culvert and finally into an existing natural drainage. Therefore high degree of weathering and erosion resulting in increase in porosity and compressibility of the subsoil (Graham and Shields, 1984). At both failed segments, the upper 6 m is seriously affected by this erosion. It's expected that the soil should treated with best stabilization method during rehabilitation/reconstruction of the failed portion of the highway.

5. Conclusions

The paper investigated the geotechnical competence of the subsoil in relation to highway structural collapse along Ifon – Benin highway. Findings revealed that the failure at both segments of the studied highway was due to geotechnical weak subgrade/subsoil with AASHTO Classification of A-7/A-7-6. This weak soil layer is about 6 m thick beneath the highway structure, with cone resistance values generally less than 20 kg/cm^2 and inferred unconfined compressive strength less than 75 KN/m^2. Coupled with poor geotechnical characteristics of the subsoil, incessant heavy flooding around the embankment/shoulder of the highway also contributed to the total collapse of the highway by rapidly removing/eroding the weak subsoil (silt/clayey subsoil) characterized by low cohesion, high permeability and porosity; and high/intermediate plasticity. Nonetheless, the flood water might have infiltrated the pavement structural material leading to looseness, and less-cohesion of the layers every wet seasons. This phenomenon led to the failure of the highway under heavy axle load, since the supporting weak competence subsoil had been eroded. Although bridge was supposed to have been constructed across the two failed segments of the highway due to associated low topography peculiar to those spots. In addition, poor road design or probably (outright) refusal by contractors or engineers to keep to the agreed industrial standard or specification might have contributed to the observed failures on the road, which is becoming the norm in the civil engineering construction sector.

In view of the above, the need for effective design of roads and maintenance policy is therefore advocated. It is therefore suggested that drainages / gutters should be provided along the shoulders of the highway to prevent erosion along shoulders and side slopes. Also bridge or culvert should be constructed across both failed segments to ease flow of water along the natural course of stream/water flow. Embankments should be constructed instead of using the natural in-situ soil material. Typical requirement for elevated embankment includes the use of good drainable material spread in thin lifts and compacted to the required density. Where good material sources are easily available, embankment materials falling between the classification of A-1-a and A-4 per AASHTO M145 are preferred. Materials meeting A-1-a classification would preferred around and behind structures, while A-4 material may be acceptable within the roadway prism but away from structures. Broken up pieces of concrete may be used within the embankment fill provided the largest dimension is no greater than 1/3 of the fill height. Care should be taken to ensure that these large size materials do not nest to create large voids that could become detrimental to the performance of the embankment due to excessive settlement.

6. Declaration of Interest

The authors declare that no financial and personal relationships with other people or organizations that could inappropriately influence (bias) this study/work.

6.1 Submission Declaration and Verification

The authors declare that this research work has not been published previously and that it is not under consideration for publication elsewhere. If accepted, it will not be published elsewhere in the same form, in English or in any other language, including electronically without the written consent of the copyright holder.

6.2 Role of the Funding Source

This research did not receive any specific grant from funding agencies in the public, commercial, or not-for-profit sectors.

6.3 Competing Interest

The authors declare no competing interest exists

6.4 Authors Contribution

Author OOF designed, wrote the protocol, arranged experimental processes, and managed the literature searches. Author AA analyzed and interpreted the data. Both authors OOF and AA prepared and approved the manuscript

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