SCIENCE FORUM (JOURNAL OF PURE AND APPLIED SCIENCES) 23 (2023) 861 - 875

Science Forum (Journal of Pure and Applied Sciences)

Journal Homepage: https://atbu.edu.ng/science-forum/

Integrated Approach of Geotechnical and Geophysical Investigations into Failed Segments of the Road Along Vwei (Mararaban Jama'a) - Mangu Road. North-Central Nigeria

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ABSTRACT

The aim of this study is to examine the subsurface structural disposition and provide useful information on the soil properties with a view to proffer solution and avert future road failure occurrences. A combined scientific approach of remote sensing, geophysical and geotechnical investigations on the earth materials within and around those failed segments have become a fundamental tool to discovering the factors responsible for the insistent problems. Vertical Electrical Soundings (VES) probe of the failed portions gave three (3) to four (4) layered earth interpreted to contain clayey-sand and loose saturated sand units at the upper portion, highly saturated weathered/fractured basement and fresh basement at the lower portion. The resistivity values for the top soil is 139Ωm - 366Ωm, weathered layer is 20Ωm-101Ωm, weathered/fractured layer is 30Ωm - 473Ωm and fresh basement is 284Ωm - ∞Ωm. The thickness of the top soil is 0.6m - 1.9m, weathered layer is 5.4m to 39.1m, weathered/fractured layer is 2.0m - 25.8m and fresh basement 11.1m - 39.4m respectively. CBR result range from 23.40 – 36.40%, indicating percentage reduction in strength with wetness of 43.7% - 50.8%. Liquid limit, linear shrinkage and plastic limit index gave values in the range of 28.0 - 47.5%, 3.6 - 10.7%, 11.4 - 18.9% respectively. This study implies that integrated structural geology, geophysical and geotechnical investigations offer very useful approach for characterizing near surface earth and thus can help in preparation before engineering structures are found on same.

ARTICLE INFO

Received 12 November 2023
Received in Revised Form 14 November 2023
Accepted 15 November 2023
Published 15 January 2024
Available online 15 January 2024

KEYWORDS

Geotechnical investigation, Geophysical investigation, Vertical Electrical Sounding, Road.

1.1 Introduction

Nigerian roads hardly last one year after construction because standards are usually compromised prior to, during and after construction. Road failure is economically retrogressive since huge resources are spent to rehabilitate or even reconstruct where most of the stretch is damaged. Rehabilitation and reconstruction usually cost more since prices of materials and services might have soared, especially in an unstable economy. Failed roads portend failed economy.

Road pavement which is described as structural materials seated upon sub-soil layers, is supposed to be a continuous stretch of asphalt laid for a smooth ride or drive, but discontinuity in the road network resulting in cracks potholes, bulges and depressions give rise to road failure (Aigbedon, 2007). Any punctuation in smooth ride is regarded as road failure. Poor drainage network, poor construction materials, bad design, usage factor are some of the factors considered as responsible for these failures.

Road pavement failure in Nigeria has become a common occurrence especially after construction. Road pavement which is

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described as a structural materials seated upon sub-soil layers, is supposed to be a continuous stretch of asphalt laid for a smooth ride or drive, but discontinuity in the road network resulting in cracks, potholes, bulges and depressions gives rise to road failure. Poor drainage network, poor construction materials, bad design, usage factor are some of the factors considered as responsible for these failures. Though road usage, construction practices and maintenance have been reported to be responsible for road failures (Adegoke, et al, 1980), field observation and laboratory experiments have shown that road failures are not primarily due to road usage, inadequate supervision, poor construction materials, non-compliance to specifications/design problems alone but can equally arise from inadequate knowledge of the characteristics and behaviour of residual soils on which the roads are built (Ajayi, 1987). Consequently, geological and geophysical experts have often emphasized lack of adequate information on the nature of subsurface conditions prior to construction as a major contributor to this phenomenon. After all, every engineering structure is seated on geological earth materials (Mesida, 1987; Ajayi, 1987; Momoh et al., 2008; Oladapo et al., 2008; Adiat et al., 2009., Adeyemo and Omosuyi, 2012). Studies have identified that the non-recognition of the underlying geology in the design of these roads as a major factor causing incessant road failure in Nigeria (Momoh et al., 2008: Oladapo et al., 2008). A lot of reasons have been suggested for the incessant failure of roads in Nigeria. These include presence of expansive clays such as montmorillonite, chlorite, halloysite, (Mesida, 1986; 1987), heterogeneity of the subgrade materials (Adeleye, 2005; Mesida, 1987), presence of undetected linear features, such as joints, fractures and rock boundaries (Momoh et al., 2008). In the past two decades, the field of geophysics has proved quite relevant in highway site investigations (Nelson and Haigh, 1990). However, in an attempt to unravel causes of persistent failure of roads across the country, various researchers have identified chiefly the underlying geologic conditions among the other factors to be responsible for this mishap (Momoh et al., 2008; Oladapo et al., 2008; Adiat et al., 2009). It therefore becomes imperative to investigate the subsurface geology upon which a road structure is to be founded rather than having recourse to a post-construction investigation and remedies. In this study, the geophysical and geotechnical techniques were integrated to examine conditions of the subsurface beneath the failed segments of the road along Vwei (Mararaba Jama'a) to Mangu road

Geological factors are rarely considered as precipitators of road failure even though the highway pavement is seated on geological earth materials (Momoh et al, 2008). This is due to non-appreciation of the fact that proper design of highway requires adequate knowledge of subsurface conditions beneath the highway route. The non-recognition of this fact has led to loss of integrity of many highway routes and other engineering structures across the country. The spate of failure of the highway has been of major concern to the governments and all stake-holder because this has led to loss of lives, properties and undermine economic and social development of the country. Other previous studies have shown that the reliability of the road pavement can be damaged by the existence of geological features and/or engineering characteristics of the underlying geologic/geo-electric sequences. It is hence vital for engineers to carry out pre-design investigations of engineering sites. The research efforts to investigate the effect of the geological factors in terms of the nature of the subsoil, the near surface structures and the bed rock structural disposition as likely causes of failures along the Vwei (Mararaban Jama'a) - Mangu highway.

1.2 Statement of the Problem

In spite of the rehabilitation efforts carried out by government on roads within the area, several segments persistently fail for reasons yet to be discovered. However, several factors could be attributed to road failures which include geological, geomorphological, geotechnical, road usage, construction practices and maintenance. Along the road Vwei (Mararaban Jama'a) – Mangu are communities such as Bisichi, Foron, Heipang, Kassa, Barkin Ladi, Dorowa, Mairana ,Mangu Halle, Sabon Gari, Mangu. Also this stretch of road serves prominent Institutions like the Plateau State Polytechnic, the Yakubu Gowon Airport, the proposed Inland Port, NYSC Orientation Camp and the Plateau State University. Likewise, many rural areas are connected to this main road and so could be termed major feeder road linking neighbouring states like Bauchi, Taraba, Benue and Nassarawa. However, this same stretch of road has these major challenges of failures (Plates 1, 2, 3 and 4).

Plate 1: Failed segments of road at Bisichi Junction on the way to Mararaban Jama'a Plateau State.

Information gathered revealed that the construction companies who handled this main road did not go as far as carrying out these proposed investigations in details in order to forestall the regular occurrences of these failures at those particular spots and the benefits of this same road to the state and neighbouring settlements cannot be overemphasized that if not investigated and rectified could eventually lead to a cut-off and consequent set back.

Plate 2: Failed segments of road at Kuru Jenta Junction Plateau State.

Plate 3: Failed segments of road just after Foron Junction enroute to the Polytechnic, Plateau State.

Plate 4: Failed segments of road at Dorowa, Plateau State.

1.3 Site description and geological Setting.

The proposed study area is located in North Central Nigeria, precisely Plateau state (Figure 1). The road stretches from Mararaban Jama'a roundabout towards Barkin Ladi to Mangu and beyond, totalling about 60 Km. The identified road problematic areas are located at Gada Biu, Kassa, Barkin Ladi and Mangu Halle. The study area is characterized by the tropical climatic condition prevailing on the Jos Plateau. Here the climate is controlled by two seasons, namely, the wet and dry seasons. The wet season prevails between the months of April and October with abundance of rainfall, while the dry season starts in the month of November and ends in the month of March. The mean annual rain fall of the study area and its environs is between 1,800 – 1900mm. The vegetation is characterised by tall grasses interspersed with trees (Iloeje 1981). The trees include, shea butter, acacias, baobab (Ajayi 2003).

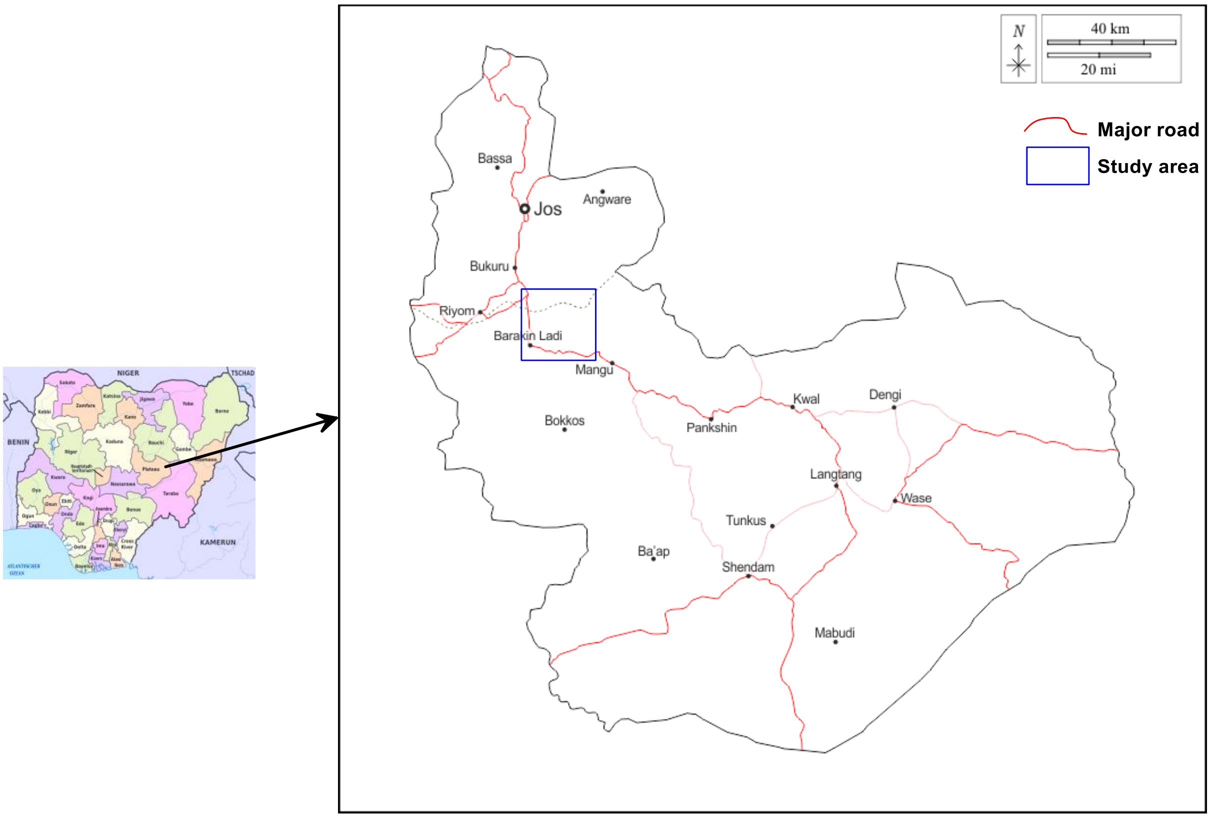


Figure 1: Location Map of Plateau State showing road networks and Study area.

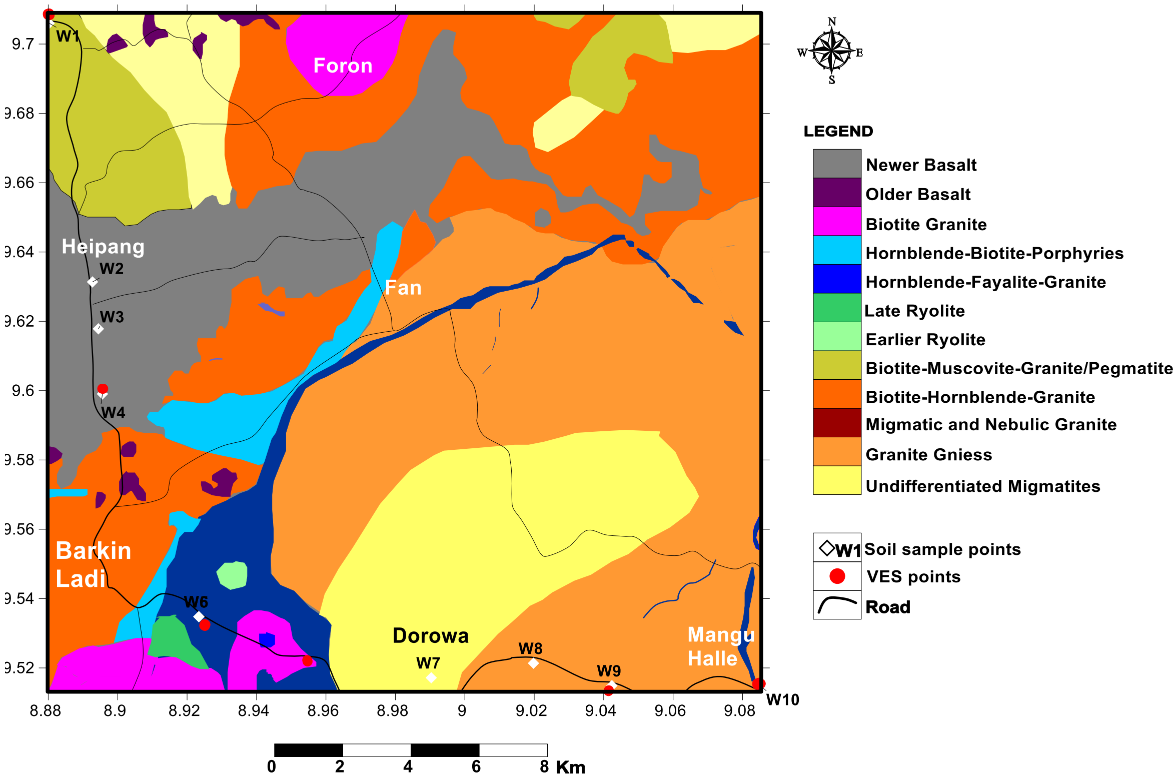


Figure: 2 Geologic Map of the Study Area (Modified after Macleod et al, 1971)

The Vwei (Mararaban Jama'a) – Mangu Road serves as a major link from the Northern Central to the Northern Eastern parts of the Country. The study area is located between

Latitudes 9° 03′ to 9° 40′ and Longitudes 8° 54′ to 9° 15′ on Naraguta S.E Sheet 168 and it covers about 60 km. The study area has an elevation that falls within the range of 3850 – 4139 m above mean sea level.

Geologically, the study area falls within the Jos-Bukuru Complex which is predominantly of biotite-granite type (Figure 2) as exhaustively studied by Falconer (1911), Falconer (1921) and MacLeod et al. (1971). In association with the biotite-granite are volcanic rocks such as basalts which are believed to have been formed during the early Cenozoic (Tertiary) “Older Basalts” and Quaternary “Newer Basalts” (Macleod et al, 1971).

1.4 Materials and methods

Integrated Structural geology, geophysical investigation involving the use of electrical resistivity and geotechnical methods were adopted for the study.

1.4.1 Structural (Lineaments) Investigation

In this study, basic maps such as lineaments and geologic maps in a digital form were produced from satellite data. The methodology of this study was based on approaches where layers of lithology and lineaments are presumed to reflect subsurface phenomenon, can generally be equated with structural elements such as folds, fractures, faults and joints (Viljoen et al., 1983); hence can serve as important factors for evaluating the reason for failed segments of the road.

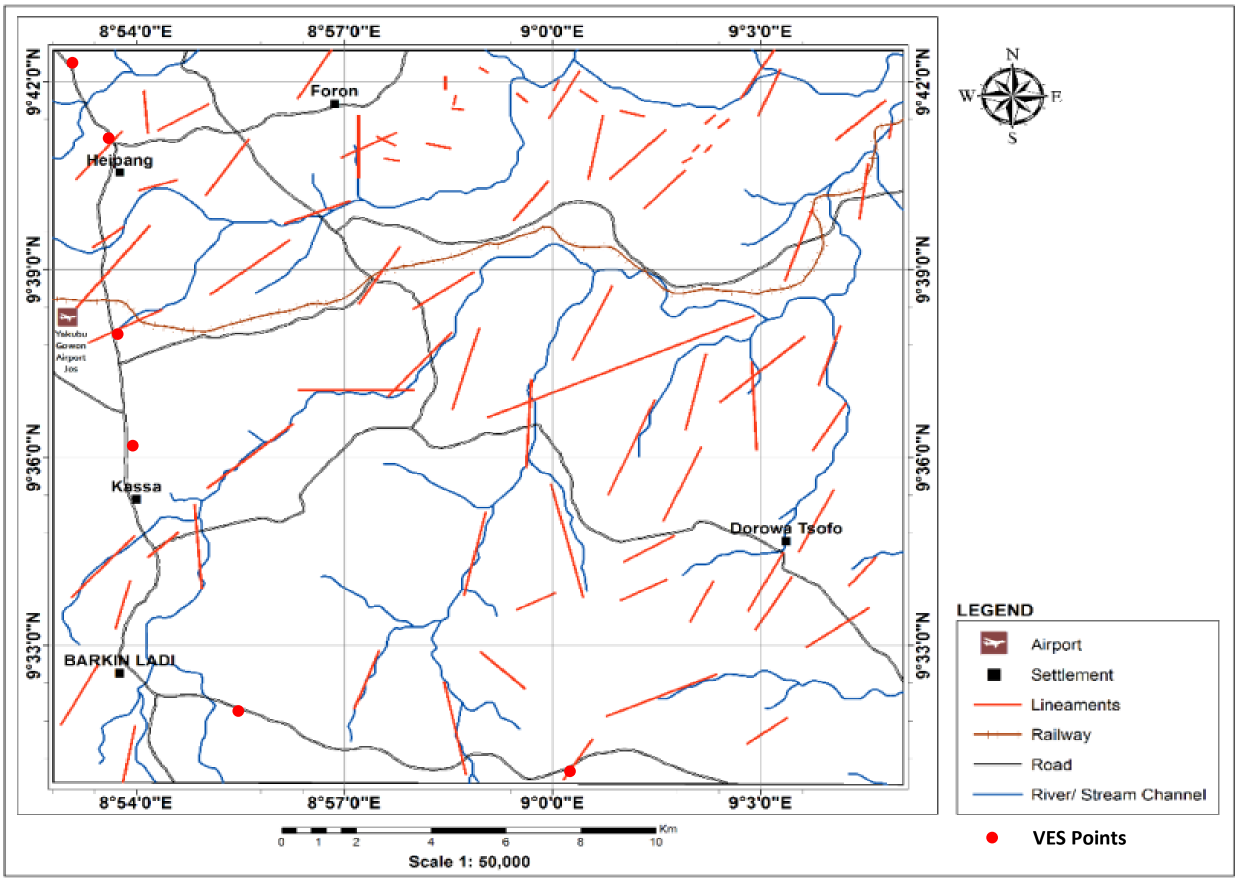


Figure 4: Lineament Map of Study area {Vwei (Mararaban Jama’a)–Mangu Road} showing VES Locations.

1.4.2 Geophysical Investigation

Twenty (20) geophysical traverses were established across both the stable and failed segments of the road pavement within the study area (Fig.2) to ascertain the subsurface layers interms of vertical and lateral extent. The VES were conducted at station spacing selected at random along the road, employing the electrical resistivity method which utilized Schlumberger vertical electrical sounding (VES) profiling in order to detect both the vertical and lateral variations in resistivity.

The resistivity measurements were made with the aid of an ABEM SAS 1000 Terrameter and its accessories. The VES data were interpreted quantitatively by partial curve matching and computer iteration technique using Win RESIST Version 1.0 and the results presented as VES curves and geo-electric sections.

1.4.3 Geotechnical Investigation

The geotechnical investigation involved collection of disturbed and undisturbed soil samples with the aid of soil auger at 1.0 m depths from ten (10) test pits established randomly apart on each of the traverse (Fig. 2). Consistency Limit Test generally known as the Atterberg limit gave the plasticity characteristics of the cohesive fraction of the samples. The consistency limit test includes; liquid limit, plastic limit and linear shrinkage test. The difference between the liquid and plastic limits gave the plasticity index, which is the range of moisture contents over which the soil remains plastic.

Standard Proctor Compaction and California Bearing Ratio (CBR) tests were conducted on the soil samples. Casagrande's equipment, Compaction mould and Penetration plunder respectively, and in accordance with the procedures of the British standard code for site investigations (BS 1377-2, 1990). The CBR test was performed under soaked condition for 48 hours. The soil samples were classified and rated based on the American Association of State Highway Transportation Officials (AASHTO) Standard Specifications for Transportation Materials and Methods of Sampling and Testing (2020), and the Federal Ministry of Works and Housing (FMWH) General Specifications for Road and Bridges (2016).

California Bearing Ratio (CBR) test, widely used to characterize and select sub-grade materials for use in road construction was carried out. The test was devised by the California Highway Association and it is simply the ratio of the load that cause a penetration of 2.5 mm or 5.0 mm material to a standard load that causes similar penetration on a standard California sample, notably 13.24 kN and 19.96 kN respectively.

$$C\,B\,R = \frac{\underline{\text{Load that caused a penetration of } 2.5/5.0\text{ mm} \times 100\,\%}}{13.24/19.96\ (\text{kN})}$$

Both soaked and unsoaked CBR tests were carried out and swelling of samples was carefully monitored during the 96 h of soaking period to assess the likely effect of water ingress on the swelling of base material. The samples were compacted at the modified AASHTO level as described under procedure for compaction test in a standard *CBR* mold.

1.5 Results and Discussion

1.5.1 Electrical resistivity interpretation

The VES computer model consists mainly of A, H, HA and KH 3-5 geo-electric layers (Table 1).

Sub Surface Layers and Geoelectric Characteristics

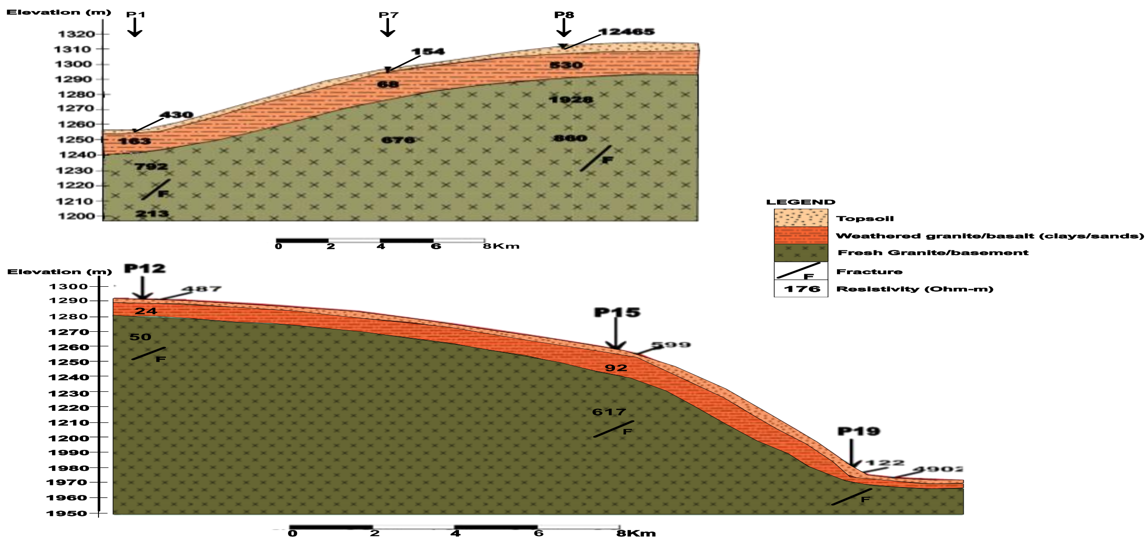
Interpretation of the VES data obtained from the field data shows 3-4 geo-electric layers or lithologic units (Table 1). The interpreted geo-electric layers/lithologic units consist of - Topsoil, weathered layer, partially weathered/fractured layer and fresh bedrock.

The interpreted sub-surface geo-electric layering of the VES locations are correlated in 2D sections (Fig. 7) in N-S direction.

The topsoil is composed of sand/lateritic sand with resistivity variation of 127–1738 Ωm and the thickness of 0.6–7.5 m. The second geo-electric layer is weathered layer with resistivity variation from 42 to 1847 Ωm while layer thickness varies from 5.4 to 39.9 m. The weathered layer is typically composed of clays/sands. The third layer is partially weathered/fractured basement. The layer resistivity varies from 30 to 473 Ωm while the layer thickness varies from 11.5 to 25.8 m. The fourth layer is fresh basement with resistivity varies from ≥762 Ωm.

Table 1: Interpreted VES results in the Study Area.

VES No.	Depth (m) d1/d2-----dn	Resistivity (ohm-m) ℓ1/ ℓ2------- ℓn	Curve type
P1	2.2/12.8/24.2/∞	697/291/835/277	HK
P2	2.2/4.1/∞	200/137/715	HA
P3	0.5/2.2/∞	3030/1847/2057	H
P4	0.9/5.6/20/∞	195/95/727/300	HK
P5	3.5/9.8/35/∞	948/360/678/463	HK
P6	0.9/19.9/∞	175/78/351	H
P7	1.8/6.5/∞	828/501/708	H
P8	1.6/39.8/∞	234/127/391	H
P9	0.6/20/46.2/∞	85/33/391/236	HK
P10	3.0/21.5/45/∞	113/97/235/167	HK
P11	2.6/22.2/55/∞	335/42/196/93	HK
P12	1.4/16.5/∞	1738/185/197	H
P13	0.5/2.8/7.6/11.9/∞	1688/146/88/102/5174	H
P14	0.6/1.3/10.7/29.8∞	785/531/103/143/638	H
P15	1.6/18/29.5/∞	241/200/991/281	HK
P16	1.4/16.5/∞	1738/185/197	H
P17	1.5/31.2/∞	1029/578/1242	H
P18	7.5/29.5/∞	1488/1175/1478	H
P19	2.5/40.2/∞	127/414/162	K
P20	0.7/5.5/7.1/∞	245/756/43/234	KA



Figures 7 (a) & (b): Geo-electric section along traverse.

This suggests, among other things, two (2) possible scenarios. The first scenario is that the study area is characterized by the occurrence of fresh bedrock at a very shallow depth with fractures at the near surface as observed in the VES P19 (Fig. 7b) can undermine the integrity of the proposed road if not properly handled during construction. Secondly, some parts of the investigated area underlain by weathered layer which are composed of clays can also increase the vulnerability of the road to failure particularly if the depth to the weathered layer is shallow with the evidence of water saturation.

Lineament Analysis and its significance.

In the study area, the major structural trends are in N-S, NNE-SSW, NW-SE, and E-W. As shown in Figure 4, several of the linear features cut across the area around failed segments of the highway. At depths ≥5m, geoelectric resistivity soundings interpretation on VES P5, P9, P10, P11, P15, P19 and P20 revealed subsurface structures (Table 1 and Fig. 7).

It is suspected that such subsurface structures may be one of the factors responsible for the perpetual failure of the failed segments of the highway especially were they occur at the shallow surface, since linear features are characteristic zones of weakness.

1.5.2 Geotechnical Results

Table 2: Geotechnical Results

Samples	Moisture Content	Consolidation Test			Compaction Test			Description of sample
		Specific Gravity GS	Coefficient of consolidation Cv (mm2/min)	Coefficient of volume change Mv (m2/kN)	MDD (Kg/m3)	OMC (%)	CBR (%)	
Heipang 1	17.60	2.69	6. 83	0.0013	1885	12.80	27.40	Laterite
Heipang 2	19.30	2.38	6.31	0.0210	1795	8.50	34.10	Laterite
Kassa	12.30	2.69	6.28	0.0210	2585	9.60	33.40	Laterite
Chit	11.30	2.64	6.26	0.0600	1900	12.20	35.70	Laterite
Bet Zat	17.60	2.00	0.20	0.0001	1915	12.20	28.30	Laterite
Wereh	9.10	4.20	0.52	0.0001	1885	14.30	35.10	Laterite
Dorowa	13.70	2.27	0.31	0.0013	1930	11.40	23.40	Laterite
Bwom	12.00	4.20	0.52	0.0008	1880	9.70	36.10	Laterite
Mairana	12.00	2.38	6.31	0.0214	1900	12.60	36.40	Laterite
Mangu Halle	11.20	2.66	2.05	0.0071	1820	11.80	30.70	Laterite

Moisture Content

The moisture content of the analyzed soil samples from both the control stable and failed segments of the road are shown in Table 2. The moisture content of soils from the road segments ranges from 9.10 -19.30%, indicating failed segments.

The entire soil samples have moderate to high moisture content. The ranges of moisture content of the soil samples taken from the stable and failed segments of the road show significant overlap. It is therefore difficult to explain the causes of failure of the failed road pavement from the point of view of the moisture content of the soils only.

Consistency Limit

As shown in Table 3, the Liquid Limit of the soil samples from the failed segments range from 28-47.50%.

Table 3: Summary of the Consistency Limit Test.

Samples	Aterberg Limit Test (%)			Plasticity index	Colour
	Liquid Limit LL	Plastic Limit PL	Shrinkage Limit LS		
Heipang 1	43.00	31.60	4.00	11.40	Dark Brown
Heipang 2	45.00	30.60	10.00	14.40	Brownish
Kassa	47.50	24.40	10.70	13.10	Dark Brown
Chit	31.10	15.00	5.00	14.40	Dark Brown
Bet Zat	34.40	21.10	8.60	13.10	Brownish
Wereh	32.00	19.10	7.10	13.90	Brownish
Dorowa	43.80	28.00	3.60	15.80	Dark Brown
Bwom	42.50	31.60	7.90	18.90	Dark Brown
Mairana	28.00	14.40	8.60	13.60	Brownish
Mangu Halle	32.80	19.10	7.90	13.70	Brownish

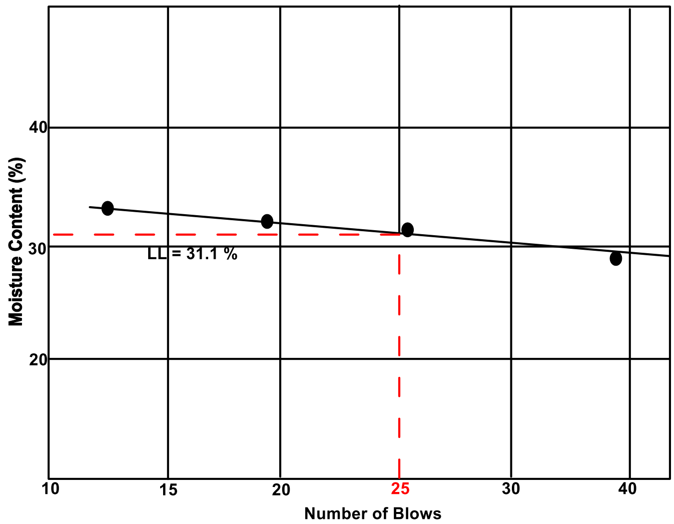


Figure 9: Liquid Limit Graph for Soil Sample taken from Chit Failed Segment

Typical Liquid Limit curve is shown in Figure 9. The Plastic Limit of the soils ranges from 14.40 - 31.60 % for failed segments of the road. The Plasticity Index of soils beneath the failed segments ranges from 11.40 – 18.90 %. Most of the soils (stable and failed localities) investigated fall within the FMWH (1972) specification (Table 3), and are suitable sub-grade materials. Using the America Association of State Highway and Transportation Officials (AASHTO) (1962) classification system, the soil samples from both the stable and failed segments belong to group A-4 to A-7, hence the soil can be rated to be of fair to poor sub-grade materials. The Cassa grande (1948) Chart generally indicate medium plasticity for both the soils beneath the stable and failed segments, hence, the soils are expected to exhibit low to medium swelling potential (Ola, 1982).

Table 4: Threshold Geotechnical Properties for Highway

% Finer (0.075mm)	Liquid Limits	Plastic Limits	Plasticity Index	Linear Shrinkage	Unsoaked CBR
< 35	50 (Maximum)	30 (Maximum)	20 (Maximum)	8 (Maximum)	80 (Minimum)

Source: Extracted from FMWH, (1972).

The Linear Shrinkage of the soils beneath the failed segments ranges from 3.60 -10.70 % (Table 3). The Linear Shrinkage of most of the soils are recommended by Brink *et. al.* (1982) and Madedor (1983) for highway sub-grade, hence the soils are good sub-grade materials. The consistency limits of the soil beneath the stable and failed segments of the studied highway did not exhibit significant difference and generally fall within the recommended specification by FMWH, 1972. Both soils are rated poor to fair for pavement materials, hence, the cause of failure along the failed segments of this road may be difficult to explain from the point of view of consistency limits only.

Compaction Characteristics

The Optimum Moisture Content (OMC) and the Maximum Dry Density (MDD) obtained from the tested soils are presented in Table 2 while Figure 10 shows typical compaction curve. The Optimum Moisture Content (OMC) ranges from 8.50-14.30 % for soil of the failed segments. The Maximum Dry Density (MDD) ranges from 1795 - 2585 Kg/m for soil of the failed segments.

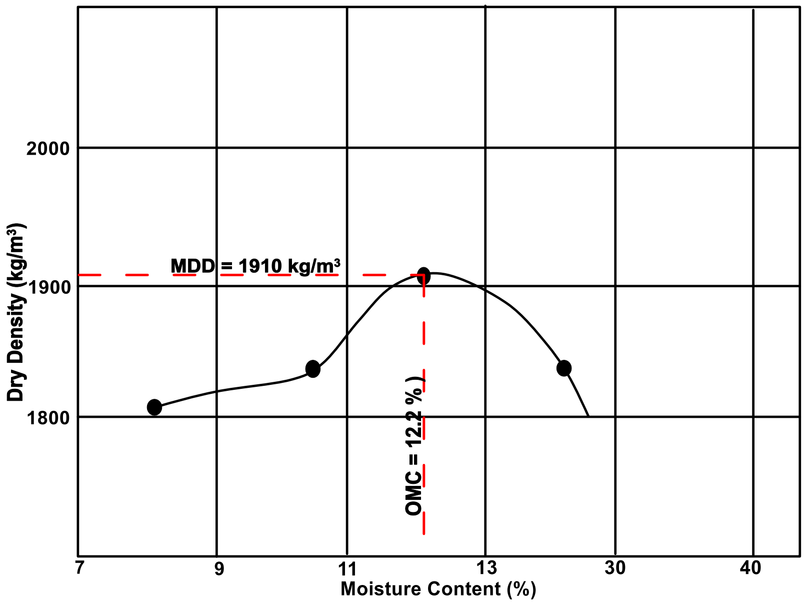


Figure 10: Compaction Curve for Soil Sample Mangu Halle failed Segment

There is significant overlap between the values of compacted parameters of the soils beneath the failed segments; hence compaction characteristic of the soils may not be a good index for the highway pavement stability.

California Bearing Ratio (CBR)

The unsoaked California Bearing Ratio (CBR) values for soil beneath the failed segments range from 23.40 –36.40 % (Table 1). Figure 10 shows the CBR curve for soil sample taken from Kassa failed segment. The unsoaked CBR value stipulated by the FMWH standards (1972) for sub-grade soils is 80% minimum (Table 4). None of the soil samples has a CBR value within this specification. Thus, there is no clear-cut correlation between the values of CBR of sub-grade soils and the degree of stability of the road. The cause of perpetual failure of the failed segments along this road may therefore not be related to the CBR. From the above, there seems to be a poor correlation between the geotechnical properties of soils and the road stability.

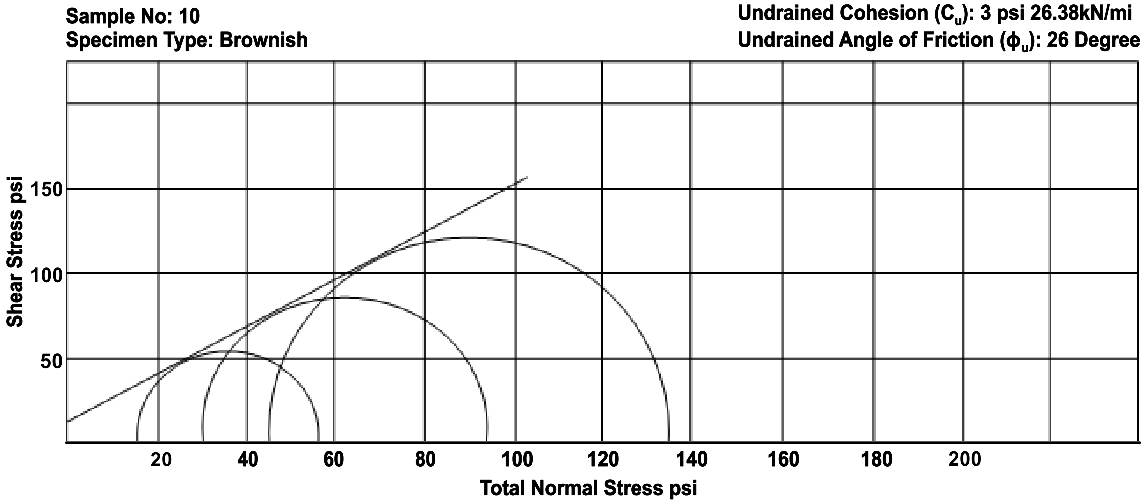


Fig. 11: Typical CBR Curve for Soil Sample taken from Kassa Failed Segment

Finally, the integrated results of geophysical and geotechnical investigations have enabled the development of the road failure vulnerability section of the studied road. Engineering geologists are responsible for identifying the geological factors that could affect construction projects. They analyze ground materials to assess their risk factors and advise on the best procedures for developments and the suitability of construction materials.

Conclusion

In this study, integrated geophysical methods were used to investigate the causes of incessant highway pavement failure along segments of Vwei (Mararaban Jama'a) – Mangu Road around Plateau State Basement Complex terrain of North Central Nigeria.

It can be concluded that the possible causes of incessant highway pavement failure along the studied highway are:

i. Thick and low resistive substratum (<100 Ωm, clayey sub-grade soil). The soil type absorbs water, swells and collapses under imposed traffic load stress and thereby leading to road pavement failure.

ii. The suspected near-surface lineament features such as fractures, fault or lithological contacts which act as zones of weakness that enhances the accumulation of water leading to pavements failure.

iii. Poor drainage of runoff from adjacent topographic highs at the toe of road pavement leading to its ponding as observed along traverse 3

Funding

This research work was funded by Tertiary Education Trust Fund (TETFund) intervention to the Plateau State Polytechnic Barkin Ladi.

Acknowledgements

The authors acknowledge the Civil Engineering Department, Plateau State Polytechnic Barkin Ladi and Ministry of Works Yard Plateau State for the Provision of their Laboratories and equipments for the Study.

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