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INTEGRATED RESISTIVITY, INDEX, AND STRENGTH CHARACTERISTICS OF SUBGRADE SOILS: IMPLICATION FOR HIGHWAY PAVEMENT FAILURE STUDIES IN NORTH-CENTRAL NIGERIA

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ABSTRACT

Integrated geophysical and geotechnical studies have been carried out to determine the geological cause(s) of the failure of sections of Ajaokuta – Anyigba Highway, North-central Nigeria. Forty-eight (48) Vertical Electrical Soundings (VES) were conducted on failed and stable sections of the highway. Also, twenty-one (21) subgrade soil samples close to VES stations from the unstable and stable sections of the highway were subjected to laboratory geotechnical analyses which include grain size distribution, Atterberg limits, compaction (Optimum Moisture Content, OMC, and Maximum Dry Density, MDD) and California bearing ratio (CBR) at soaked and unsoaked states following American Society for Testing and Material (ASTM) standards as appropriate. The geophysical results show that low resistivity (10–100 Ohms-m) inferred as clay/silt of low competence characterizes the subgrade soils of the unstable segment. While higher resistivity (148–272 Ohms-m) interpreted as sandy-clay/silt with moderate competence was obtained for the subgrade soils of the stable segment. Results of Geotechnical tests show that the subgrade soils of the unstable segment have geotechnical properties that generally fall below required standard specifications. Strong correlations of R = 0.86, 0.9, and –0.88 were obtained between CBR and sand, resistivity, and the amounts of fines, while a fairly strong correlation of R = –0.67 was obtained for the plasticity index. The high level of correlation implies that CBR can be predicted from geophysical data and other geotechnical parameters. The study has revealed that the advanced weathering of the underlying Mica-Schist to clayey/silty subgrades with unsuitable geophysical and geotechnical properties is a major contributor to the instability of the highway.

Keywords: Atterberg limit; CBR; Grain size distribution; Resistivity; Subgrade soil.

1 INTRODUCTION

The importance of roads in driving national, societal, and community development cannot be overemphasized [1]. In developing countries, such as Nigeria where means of transportation including underground tubes, rails, and water systems are mostly underdeveloped, flexible road pavement is the most common means of transportation [2]. Evaluation of existing highways is paramount to proposing better designs that can help to curtail the alarming frequency of failed roads all over the world [3]. In addition to the fact that it is common knowledge in Nigeria that highway pavement failure is a norm, most highways rarely attain their design age before failure [4]. A highway can be said to have failed when it can no longer perform its primary purpose or when it has structural defects. Thus, the failure of highways can be structural and/or functional. Research in [5] suggested that the perpetual failure of some sections of roads in southeastern Nigeria has not only negatively impacted human and vehicular movements but also incurred huge financial loss arising from incessant reconstruction and rehabilitation. The highway is an important engineering structure that consists of subsoil layers that include the subgrade, subbase, and base course in flexible pavement designs. The subgrade soil is the foundation soil and a product of the in-situ weathering of the basement rock (in the Basement Complex environment). It is therefore a product of geology and the climatic factor of an environment. Unfortunately, geological factors are rarely considered in the construction and rehabilitation of several highways in Nigeria even though the subgrade soil is a product of geology [6]. The causes of continuous failure of highways in Nigeria are not adequately investigated before rehabilitation [6]. Sources of highway failure have always been attributed to the use of poor construction materials, traffic loading, and poor design [7–9]. Similar causes of failure are often proposed irrespective of the geology of the area. A few studies have, however, shown the influence of geology on the stability and otherwise of highways. Research by [10 and 11] revealed that highway failure could be precipitated by geological features which include faults and clayey subgrades.

Despite the widespread failure of highways nationwide, most highway pavement failure studies in Nigeria concentrate on the southwestern parts of the country. Many failed highways (such as Kabba-Isanlu-Egbe, Ekirin – Omuo, and Ajaokuta-Ayingba highways in North-Central, Nigeria) are common with little or no effort to study the factors responsible for the incessant failure of highways in this part of the country. Therefore, there is a need to evaluate the causes of highway failure in other parts of the country. This could aid in comparative purpose and regional management strategies of highways in Nigeria.

An integrated geophysical technique (resistivity surveys) and geotechnical tests involving index (grain size distribution and Atterberg limits) and strength (compaction test and California bearing ratio) properties have been employed in this study. The combination of these methods helped to decipher the nature of the subgrade soils underlying the road alignment as well as provide a useful relationship between them. Several authors have shown the success of these methods or other integrated geological methods in highway subsoil evaluation. The geotechnical method was adopted by [12] and [6]; while authors [13–15] used the geophysical method. A study by [16] employed the geophysical and geotechnical methods and [17] adopted the mineralogical, geochemical, and geotechnical methods. In addition, [18] applied the use of Multichannel Analysis of Surface waves in highway subsoil evaluation. These studies showed the relevance of understanding the characteristic nature of the geology and subgrade soils before highway pavement construction.

The highway under study is a dual carriageway and an important road that can drive Nigeria's economy as it links several states in Nigeria. It is situated in Kogi State, North-central Nigeria, and is delimited within 6°40'30"–6°48'0" N and 7°22'30"–7°30'0" E (Figure 1). The highway is underlain by Mica-Schist and Granite Gneiss [19] which are Precambrian in age [20]. While the failed portion of the highway is underlain by subgrade soils obtained from Mica-Schist, the stable section rests on the weathered products of Granite-Gneiss. Both dual carriageways have failed and are characterized by the extensive and complete rippling of the asphalt layer as well as depressions. Although very recently, an arm of the carriageway has been rehabilitated, some sections have started failing again. This is because the possible cause(s) of this failure is (are) yet to be studied and rehabilitation exercise without an understanding of the nature of geology would continue to lead to huge financial loss and wastage of resources through repeated failures. The present study combines geophysical and geotechnical investigation to characterize the subgrade soils of the highway and establishes relationships between geophysical and geotechnical parameters.

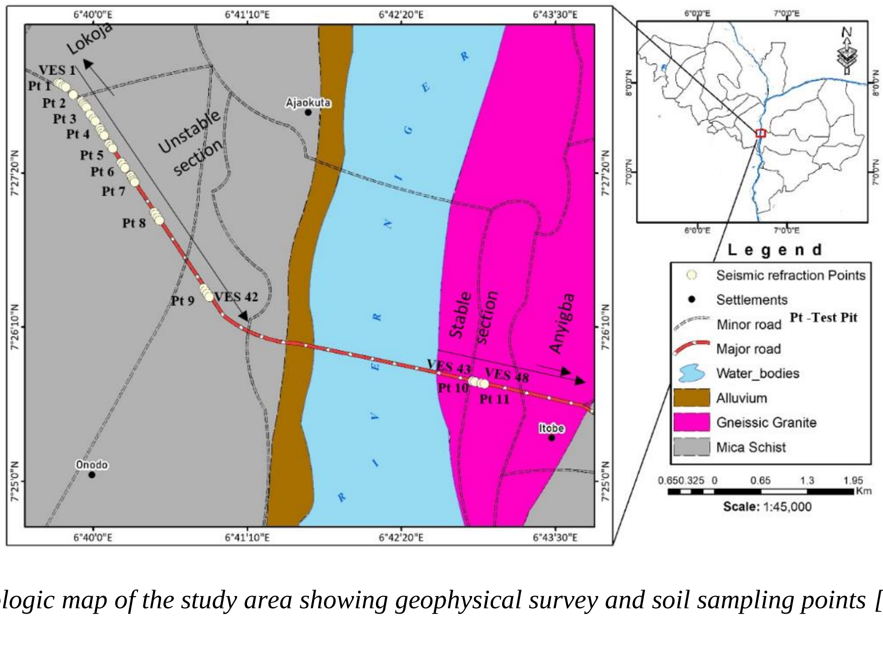

Figure 1. *Geologic map of the study area showing geophysical survey and soil sampling points [mod. from 19]*

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2 METHODOLOGY

Both geophysical and geotechnical techniques were deployed in this study. The locations of the geophysical survey points and the test pits from which the samples were obtained were dictated by the geology of the area and the stability or otherwise of the sections. The electrical resistivity technique using the Schlumberger array was adopted for the geophysical study. Forty-eight Vertical Electrical Soundings (VES) were conducted with total current separation (AB/2) between 65–100 m). Forty-two VES were on the unstable section and six on the stable section to serve as the control. The procedures for the acquisition, processing, and interpretation of the VES have been well documented by [21 and 22].

Twenty-one (21) soil samples formed from the in-situ weathering of the parent rocks were obtained; eighteen from the failed section and three from the stable section for control. The samples were obtained from eleven test pits dug (0.1–1 m depth) close to VES stations. This was to enable the integration of geotechnical and geophysical data as well as to establish relationships between the two methods through regression analysis. The subgrade soil samples were subjected to geotechnical index or classification tests which include, grain size distribution and Atterberg limit (Plastic and Liquid limit); strength test which involves compaction (standard proctor) and California bearing ratio (CBR) in unsoaked and soaked conditions. All the tests were performed according to the appropriate codes of the American Society for Testing and Materials (ASTM). A slight modification was however made in the determination of the grain size distribution which involved adding about 2% Calgon (Sodium hexametaphosphate) to soil solution for about 24 hours to effectively separate the clayey fragments from the sands. This is documented in [23 and 24].

3 RESULTS AND DISCUSSION

3.1 Resistivity results

The results of the resistivity values of the subgrades soils are summarized by the Box and Whisker plot (Figure 2). The resistivity of subgrade soils can be used to predict their competence. This is because the resistivity of a material is largely dependent on the fluid content and matrix or mechanical properties. A study by [26] proposed soils' competency rating based on resistivity range (Table 1). This was used to classify the soils. The resistivity of the subgrade soils of the unstable and the stable sections ranged from 10–100 Ohms-m and 148–272 Ohms-m. The subgrade soils of the unstable section, therefore, classify as soils with incompetent ratings while those of the stable section are moderately competent. The very low resistivity values of the subgrade soils of the unstable section indicated that the parent rock (Mica-Schist) has undergone intensive and extensive weathering to form weathered products dominated by very fine grain soils which are silty and clayey. These clayey/silty soils are inimical to the stability of the highway due to their low bearing capacity when subjected to the axle loads of vehicles and trucks that ply the highway regularly. By contrast, the relatively higher resistivity of the subgrade soils of the stable section reflects the preponderance of coarse-grained soils (sand/gravel) over the fine-grained soils (clay/silts) formed from the weathering of the underlying granite-gneiss. Field macroscopic studies of the Mica-schist and Granite-Gneiss showed that the mineralogy of the former is essentially made up of micas (biotite and muscovite), feldspar and quartz and possess schistosity. The schistosity describes the ease with which this rock can be split along a plane. The dark mica (biotite) however appears to dominate, making the rock entirely dark in colour. A little amount of quartz is visible in the parent rock. Biotite and Feldspar disintegrate relatively easily to form clay while the muscovite forms more silt upon weathering as evidenced by the shining lustre of the sampled weathered products. Quartz in the parent rock is the most resistant to weathering and remains virtually unchanged but forms a minor amount of sand and thus offers a minimal contribution to the stability of the highway. On the other hand, the granite gneiss is dominated by quartz and feldspar with minor amounts of micas. Although feldspar and micas weather to clay and silt, the high resistance of quartz to weathering makes the disintegration product of the parent rock to be dominated by sand, which offers a significant contribution to the stability of the highway due to higher bearing capacity.

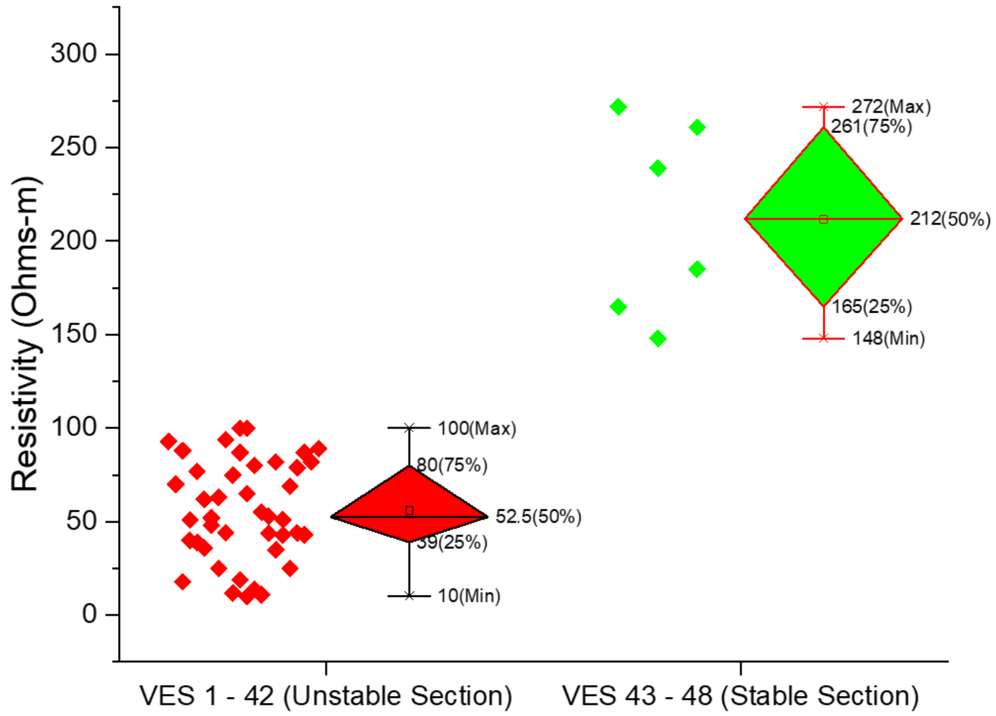

Figure 2. *Box and Whisker Plots of resistivity of the clayey (weathered) subgrade soils of unstable and stable sections*

Table 1. *Classification of resistivity values based on competence rating [25]*

Apparent resistivity range (Ohm-m)	Lithology	Competence rating
< 100	Clay	Incompetent
100 – 350	Sandy clay	Moderately Competent
350 – 750	Clayey sand	Competent
>750	Sand/laterite/bedrock	Highly Competent

3.2 Geotechnical results

3.2.1 Grain size distribution

Figure 3 shows the grain size distribution of the subgrade soil samples. Samples "S1–S9" are from the unstable section while "S10–S11" are from the stable section. Figure 4 shows the AASHTO classification plot of the fine-grained fractions subjected to the Atterberg limit test.

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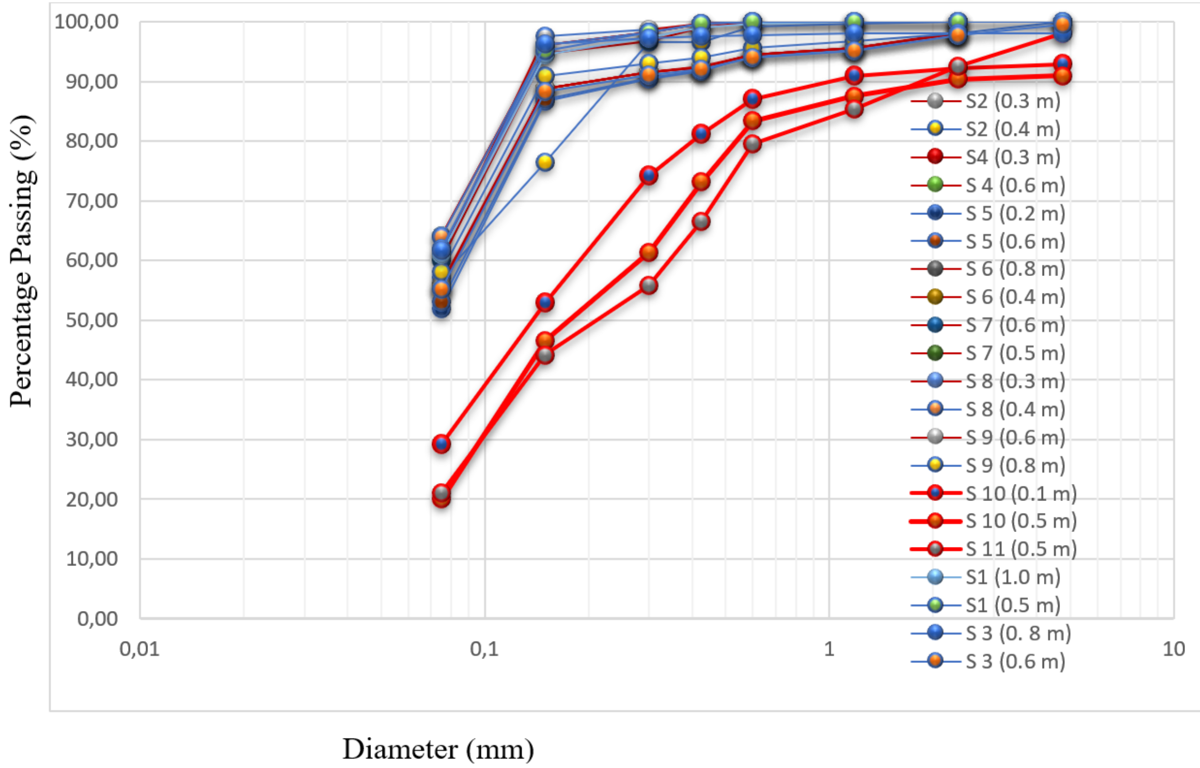

Figure 3. *Grain size distribution of subgrade soils*

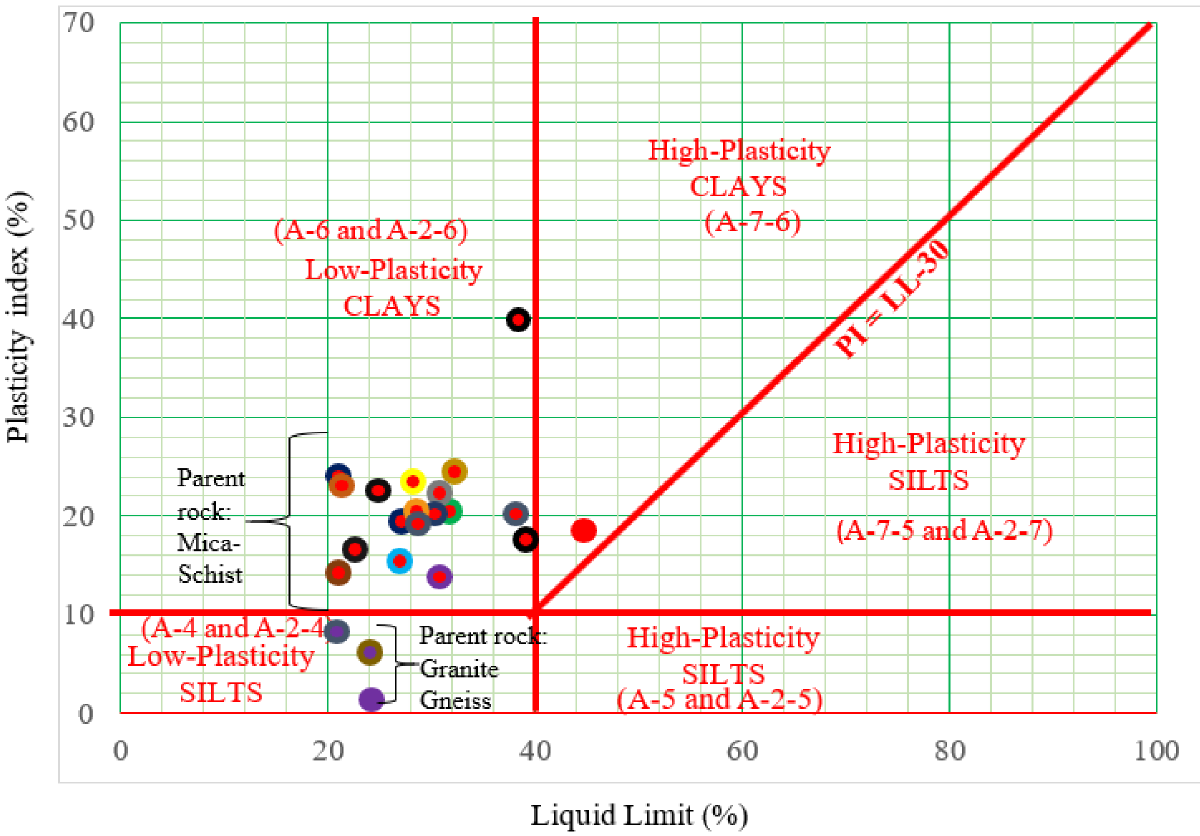

Figure 4. *AASHTO Soil Classification of subgrade soils*

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Table 2 summarizes the results of the grain size distribution and the Atterberg limit test. The percentage of sand in the subgrade soils of the unstable and stable sections ranged from 36–48% and 71–80%, respectively. While the amounts of fines (silt+clay) in the subgrades of the unstable and stable sections ranged from 52–64% and 20–29%, respectively. According to the Unified Soil Classification Scheme (USCS), soils with >50% sand are essentially coarse-grained while those below 50% are fine-grained. This implies that the subgrade soils of the unstable section are primarily fine-grained. This agrees with the result of the geophysical study as well as the geology. On the other hand, subgrade soils of the stable section are principally coarse-grained, and this confirms the findings of the geophysical study. In the same vein, the subgrade soils of the unstable section have poor to fair subgrade rating according to The American Association of State Highway and Transportation Officials (AASHTO) classification for subgrade soils with fines >35%. The subgrade soils of the stable section classify as good to excellent subgrade soils consequently.

The Atterberg limit test results (Table 2) and Figure 4 revealed that the subgrades of the unstable segment have Liquid limit and Plasticity index ranging from 21.3–46.0% and 12.7–38.8% While subgrades of the stable section respectively have 23.2–25.5% and 0.4–8.1%. The subgrades of the unstable section classify as A-7-6 and A-6 clayey soils with low to high plasticity while those of the stable section are A-2-4 silty soils with low plasticity. The plasticity index of subgrade soil provides clues on potential volumetric changes, permeability, and distress condition under unstable groundwater situations after construction [26]. Soils with low plasticity and liquid limit are often desired as subgrades soils for highway construction to prevent largely volume changes in soils that could trigger the failure of the highway during alternate wetting and drying of the soils in wet and dry seasons. The specifications in [27] stipulate that subgrade soils' Liquid limit and Plasticity Index should be less than 50% and 30%, respectively. The majority of subgrade soils of the unstable section however satisfy these criteria. This is attributed to the silty nature of the muscovite mica in the subgrade soils which reduces the plasticity effect. All the subgrade soils of the stable location meet these requirements.

Table 2. *Grain size distribution and Atterberg results of subgrade soils*

S/N	**Parent rock**	**Pit no.**	**Sampling depth (m)**	**Sand (%)**	**Amounts of Fines (%) AASHTO Standard (F≤35%)**	**Liquid Limit (LL)**	**Plasticity Index (PI) = (LL-PL)**	**AASHTO Classification**	**Geologic material**	**General Rating as Subgrade**
1	Mica-Schist	1	0.50	36	62	33.8	12.7	A-6	Clay/Silt	Fair to Poor
2	Mica-Schist	1	1.00	38	61	28.2	15.1	A-6	Clay/Silt	Fair to Poor
3	Mica-Schist	2	0.30	39	61	24.0	24.0	A-6	Clay/Silt	Fair to Poor
4	Mica-Schist	2	0.40	43	57	38.8	38.8	A-6	Clay/Silt	Fair to Poor
5	Mica-Schist	3	0.60	45	55	32.9	19.5	A-6	Clay/Silt	Fair to Poor
6	Mica-Schist	3	0.80	38	62	34.3	25.2	A-6	Clay/Silt	Fair to Poor
7	Mica-Schist	4	0.30	45	55	29.4	23.5	A-6	Clay/Silt	Fair to Poor
8	Mica-Schist	4	0.60	45	55	31.2	22.4	A-6	Clay/Silt	Fair to Poor
9	Mica-Schist	5	0.20	48	52	21.3	14.3	A-6	Clay/Silt	Fair to Poor
10	Mica-Schist	5	0.60	47	53	27.5	18.2	A-6	Clay/Silt	Fair to Poor
11	Mica-Schist	6	0.40	45	55	31.0	20.5	A-6	Clay/Silt	Fair to Poor
12	Mica-Schist	6	0.80	44	56	24.0	16.0	A-6	Clay/Silt	Fair to Poor
13	Mica-Schist	7	0.50	36	64	26.3	21.2	A-6	Clay/Silt	Fair to Poor
14	Mica-Schist	7	0.60	40	60	23.8	23.8	A-6	Clay/Silt	Fair to Poor
15	Mica-Schist	8	0.30	36	64	38.5	18.6	A-6	Clay/Silt	Fair to Poor

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16	Mica-Schist	8	0.40	36	64	46.0	17.4	A-7-6	Clay/Silt	Fair to Poor
17	Mica-Schist	9	0.60	45	55	40.0	17.5	A-6	Clay/Silt	Fair to Poor
18	Mica-Schist	9	0.80	42	58	30.0	18.4	A-6	Clay/Silt	Fair to Poor
19	Granite-Gneiss	10	0.10	71	29	23.2	8.1	A-2-4	Silty Sand	Excellent to Good
20	Granite-Gneiss	10	0.50	80	20	25.5	7.3	A-2-4	Silty Sand	Excellent to Good
21	Granite-Gneiss	11	0.50	77	21	25.0	0.4	A-2-4	Silty Sand	Excellent to Good

3.2.2 Strength characteristics

The compaction and California Bearing Ratio (CBR) results are presented in Table 3. The compaction curves are presented in Figure 5. Compaction of soil is a mechanical method utilized in improving soil geotechnical properties. The purpose of compaction is to enhance the density of the soil and reduce the pore space filled by air to increase the bearing or load-carrying capacity of the soil. Two key compaction parameters are evaluated which include the Optimum Moisture Content (OMC) and the Maximum Dry Density (MDD). Generally, the higher the MDD and the lower the OMC, the better the soil as highway foundation material. [27] proposed MDD values above 1.7 g/cm^3 while OMC should be less than 18% for soil to be suitable for subgrade. MDD and OMC values for subgrade soils of the unstable section ranged from 1.68–2.03 g/cm^3 (average of 1.77g/cm^3) and 9.62–18.7% (mean of 13.2%). The subgrade soils therefore generally meet the requirement except for one sample. This result suggests that if the subgrade soils are well compacted on the field with higher energy, the carrying capacity of the soil will be enhanced. The MDD and OMC values of the subgrade soils from the stable section however show better compaction characteristics than the former. MDD and OMC ranged from 1.89–2.69 g/cm^3 (mean of 2.2 g/cm^3) and 10.4–13.2% (average of 11.6%). The compaction curves show that further addition of water to the soils beyond the maximum dry density leads to a continuous decrease in the dry density. Thus, care should be taken during field compaction to avoid the addition of water beyond the OMC. This is because the infilling of water in the pore spaces will reduce soil grain to grain contact thereby lowering the frictional force between grains. This would rather cause the loosening and weakening of the soils and consequently, reduce the bearing capacity of the soils.

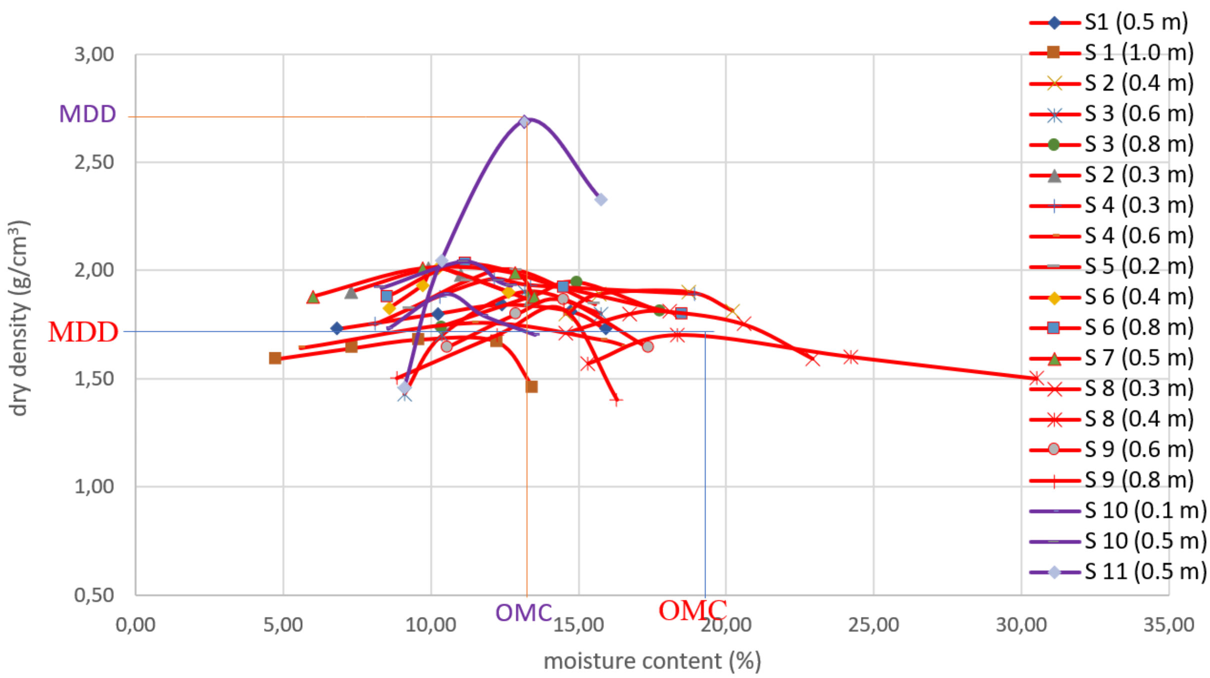

Figure 5. *Compaction curves of the subgrade samples*

The CBR of soil is different from the compaction test. The CBR is specifically a test designed to evaluate subgrade soils' resistance to penetration. The CBR is carried out both in unsoaked and soaked conditions to simulate the resistance to load penetration during dry and rainy seasons respectively. The soaked CBR gives an idea of the strength that would be lost during ingress of water in the subgrade soils which is the worst scenario. It is thus better for road pavement to be designed with the soaked results. Generally, the higher the CBR the better the soil is as subgrade material. [27] recommends a CBR value of not less than 10% for subgrade soils. The subgrade soils of the failed section have unsoaked and soaked CBR ranging from 4–9% and 2–4%, respectively. The percentage loss in strength as a result of soaking ranged from 20–78%. This implies that there will be an appreciable loss in the subgrade strength during rainfall or ingress of water. A drainage system is therefore required as the obvious absence of this could have contributed to the failure of the highway. The CBR in unsoaked and soaked conditions falls below the required specification. This suggests that the subgrade cannot withstand the penetration caused by axle loads of vehicles and trucks that regularly ply the highway. The subgrade soils would therefore require stabilization. The subgrade soils of the stable section have CBR unsoaked and soaked ranging from 15–17% and 7–10%. While the unsoaked CBR satisfies the criteria, the soaked falls just below the recommendation. The contractors however had taken care of this by providing drainages at the flanks of the stable section which help to keep the subgrade soils well-drained and unaffected by the ingress of water. It is thus recommended that the unstable section should not only be stabilized but also be provided with drainages at its flank as these are absent.

Table 3. *Compaction and CBR results of subgrade soils*

Sample no.	Pit no.	Parent rock	Optimum Moisture Content (OMC) %	Maximum Dry Density (MDD) g/cm^3	*CBRu	*CBRs	Strength Loss (%)
1	1	Mica-Schist	9.62	1.68	5	3	40
2	1	Mica-Schist	14.7	1.81	6	2	67
3	2	Mica-Schist	9.95	2.01	9	3	67
4	2	Mica-Schist	18.7	1.9	4	3	25
5	3	Mica-Schist	13.2	1.9	6	2	67
6	3	Mica-Schist	14.9	1.95	6	2	67
7	4	Mica-Schist	12.1	1.96	8	3	63
8	4	Mica-Schist	11.4	1.76	8	4	50
9	5	Mica-Schist	12.9	2.0	4	3	25
10	5	Mica-Schist	11.9	1.8	5	3	40
11	6	Mica-Schist	10.2	2.01	9	2	78
12	6	Mica-Schist	11.2	2.03	8	3	63
13	7	Mica-Schist	9.72	2.01	6	4	33
14	7	Mica-Schist	11.07	1.82	5	4	20
15	8	Mica-Schist	18.1	1.81	6	4	33
16	8	Mica-Schist	18.3	1.7	8	4	50
17	9	Mica-Schist	14.5	1.86	6	3	50
18	9	Mica-Schist	14.5 (*Av 13.2)	1.82 (Av. 1.77)	7	4	43
19	10	Granite-Gneiss	10.4	1.89	15	7	53
20	10	Granite-Gneiss	11.1	2.04	17	8	53
21	11	Granite-Gneiss	13.2 (*Av. 11.6)	2.69 (Av. 2.2)	16	10	38

**CBRu and CBRs are unsoaked and soaked California Bearing Ratio; Av: Average*

3.2.3 Relationship between resistivity, index properties, and California bearing ratio

The relationship between resistivity of the subgrade soils and the amounts of sand, fines, plasticity index, and California Bearing Ration was investigated (Figures 6–9). The CBR is the direct measurement of strength. It is however costly and time-consuming to determine CBR both in the unsoaked and soaked condition, especially when several samples are involved. The empirical relationship between CBR and these properties can be a reliable, less costly, and time-effective alternative for the determination of CBR. In this study, the unsoaked CBR was used for the correlation.

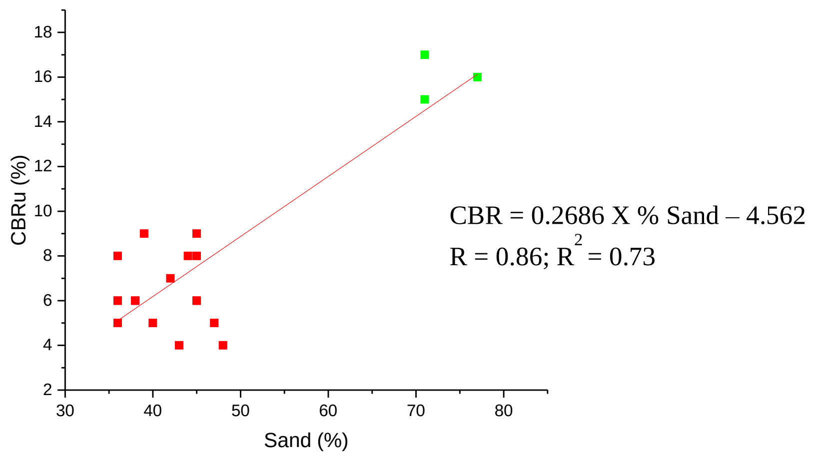

Figure 6. *Regression plot of CBR and percentage sand*

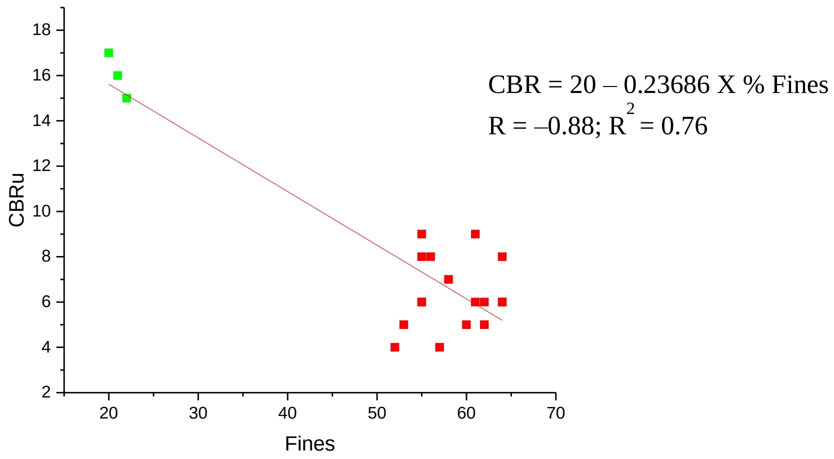

Figure 7. *Regression plot of CBR and percentage fines*

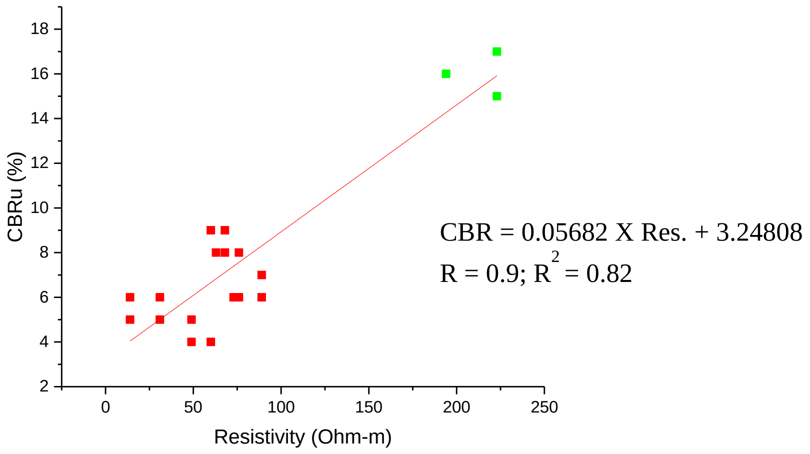

Figure 8. *Regression plot of CBR and resistivity*

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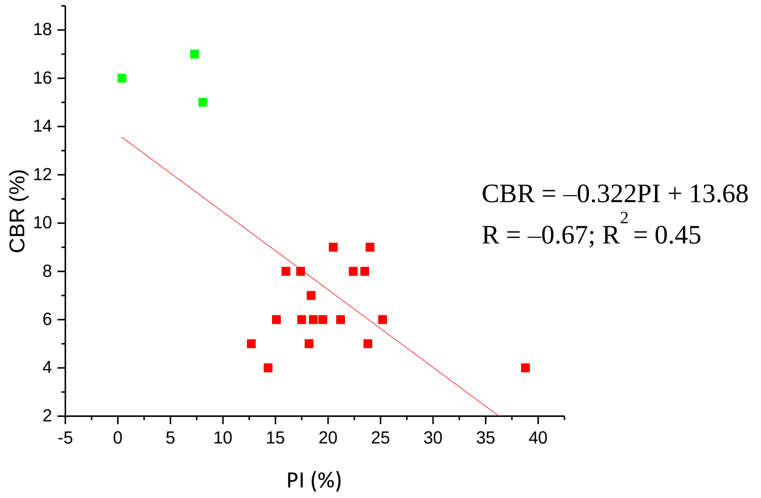

Figure 9. *Regression plot of CBR and percentage PI*

There is a strong correlation between CBR with Sand, Fines, and Resistivity: R = 0.86, –0.86, and 0.9 respectively. There is a fairly strong correlation between CBR and PI, R = –0.67. However, CBR has a negative relationship with fines and Plasticity index (PI). This implies that CBR is expected to decrease with an increase in fines and plasticity index. Also, the R^2 showed that 73%, 76%, 82%, and 42% of the variation in CBR data is accounted for by the amount of sand, fines, resistivity, and plasticity index respectively. The regression equations (except for the equation between CBR and PI) could therefore be reliably used to estimate CBR in areas with similar geology.

4 CONCLUSION

The low resistivity of the subgrades of the unstable section revealed that they are clayey/silty and of incompetent rating. The high percentages of fines and low amounts of sands of the subgrades of the unstable section classified them as fair to poor subgrade soils. The liquid limit and plasticity index indicated that they are of low-high plasticity. The Optimum Moisture Content and Maximum Dry Density revealed that most of the soils have good compaction characteristics but have very poor California Bearing Ratios. The high level of correlation implies that CBR can be predicted from other geotechnical and geophysical parameters. The weathering of the underlying Mica-Schist to clayey/silty subgrades with unsuitable geophysical and geotechnical properties has been determined to be a major contributor to the instability of the highway thus requiring a form of soil improvement as well as the incorporation of adequate drainage systems.

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CONFLICTS OF INTEREST

The authors have no conflicts of interests to declare that are relevant to the content of this article.

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