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GEOPHYSICAL AND GEOTECHNICAL INVESTIGATION, AND LIME STABILIZING EFFECT ON SUBGRADE FAILURE AROUND A SECTION OF LAGOS-ABEOKUTA HIGHWAY

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ABSTRACT

An electrical resistivity survey and geotechnical investigations were employed to examine the factors responsible for incessant roadway failure along sections of the Lagos-Abeokuta highway in Southwestern Nigeria. A study on the efficacy of lime as a stabilizing agent on failed subgrade soil was also conducted. Electrical resistivity imaging and laboratory geotechnical tests with the aid of grain size analysis, Atterberg's limits, the standard proctor compaction test, the CBR (unsoaked and soaked), and the compression test (UCS) were carried out for this study. The 2D resistivity images reveal incompetent subgrade layers with resistivity values ranging between 12– 88.3 Ω.m, and values ranging between 21.5 and 127.3 Ω.m at a depth of approximately 2 m for the VES investigation. The laboratory geotechnical tests' results along the failed segments also reveal poor subgrade soils which are composed of clay, clayey sand, and sandy clay; with plasticity indices ranging between 29.4 and 31.5, and average values of 74.5 KN/m2 and 1.69 g/cm3 for the UCS and maximum density, respectively. However, with lime admixture to the soil samples in 2%, 4%, 6%, 8%, and 10% proportions, there was a progressive improvement in the shear strength of the clayey soil samples, owing to the reduction in the plasticity and increase in the MDD and UCS.

Keywords: Geotechnical investigation; Geophysical survey; Roadway failure; Stabilizer; Subgrade.

1 INTRODUCTION

Subgrade failure in highway pavement structures is prevalent around tropical regions with high plasticity lateritic soils and medium to high rainfall, which affects road durability due to factors such as erosion, groundwater movement, and vehicular activity. Pavement is a structural material above a subgrade layer [1]. Asphaltic pavement, which is the most common in Nigeria, is typically a multi-layered system comprising a subgrade (support), subbase, base course, and surfacing. Its essence is to transmit the load received from the traffic via the layers to the subgrade [2]. Deteriorating road pavements mostly arise as a result of structural defects such as potholes (Fig.1a), gullies (Fig.1b), cracks, depressions, and ruts [2, 3, and 4]. These often result in an increase in travel time, cost of transport, unnecessary usual vehicle maintenance, etc.

Most roads and highways are constructed without due consideration of the type of subgrade soil on which they are laid, thereby failing soon after their construction because soil types vary based on their geotechnical properties. For instance, clay soils are problematic for geotechnical purposes because of their high-rate swelling and shrinkage when mixed with water.

Many major roads are subjected to undue pressure from the weights of different categories of vehicles, ranging from small vehicles to heavy-duty trucks, because road transport is the most developed transport sector, thus making roads the most common means whereby people and goods are conveyed from one part of the country to the other [5]. Many Nigerian roads fail within a short period of their construction, before their estimated lifespan, owing to many factors such as insufficient/lack of subgrade soil investigation, bad construction materials [6], and lack of proper drainage [7].

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Flexible pavements failures often result in the gradual weakening of the road pavements which occur as a result of the following: Poor subbase/subgrade soil properties [8], road maintenance negligence, poor adequate pavement design and without good workmanship, water intrusion into the subbase/subgrade layer [3], and stress from heavy vehicles [4].

Figure 1. *(a) Failed portion of the highway caused by pothole (b) Section of the highway damaged by erosion*

Geophysical methods, such as electrical resistivity techniques, have been widely used in recent times to investigate the subsurface lithology of highway pavements, especially with Schlumberger and Wenner electrode configurations, which have been established as effective methods for subsurface investigation. For instance, [9] conducted research along the Lagos-Badagry highway using two-dimensional (2D) electrical resistivity imaging with a Wenner configuration. They observed variations in subsurface resistivity profiles caused by a mixture of clay-rich subgrade soil and waterlogged sands from the results, which suggest part of the reasons responsible for the road pavement failures.

This method was also employed by integrating geophysical and geotechnical methods to investigate the reasons for consistent road pavement failures on one of the Lagos roads in southwest Nigeria [10]. Through Electrical Resistivity Imaging (ERI) with the Wenner configuration, they were able to provide a detailed delineation of subsurface factors affecting roadway integrity. The aim of subsurface characterization for road failure rehabilitation along sections of the Osogbo-Iwo highway [11], which was achieved through Vertical Electrical Sounding (VES), was used to estimate laterite deposits necessary for effective highway maintenance, thereby emphasizing the importance of understanding subsurface geological layers.

These studies show the effectiveness of electrical resistivity methods in investigating and understanding how subgrade conditions cause highway failure, especially in tropical regions such as the southwestern part of Nigeria.

Lime stabilization has become an established technique used in recent years for the improvement of the mechanical properties of clayey soils to ensure road pavement durability, because the efficiency and durability of subgrades generally depend on three major basic features: load bearing capacity, moisture content, and shrinkage and/or swelling capability.

Over the years, studies have shown the ability of lime to improve the unconfined compressive strength (UCS) and California bearing ratio (CBR) of clayey soils and have generally identified an optimal lime content between 6% and 8%, which maximizes strength and durability improvements without causing brittleness [12, 13,14]. However, these improvements have been observed to be a result of the chemical processes that usually involve the formation of cementitious compounds, as revealed by microstructural analysis [15].

Therefore, subgrade soil stabilization by lime has been proven to be an effective technique for enhancing the geotechnical properties of subgrade soils, especially in areas characterized by high-plasticity lateritic soils.

Several studies have been done to investigate the application of lime as mechanisms for the improvement in the subgrade soil's strength. [12] carried out an investigation to stabilize lateritic soils along sections of the Sagamu–Papalanto road, southwestern Nigeria, using lime. From their results, lime percentage ranging between 0% and 20% can significantly increase the Unconfined Compressive Strength (UCS) and California Bearing Ratio (CBR)

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of the soils, with optimal improvements observed at 6% to 8% lime. The study shows the effectiveness of lime in reducing soil plasticity, which leads to an improvement in the mechanical strength of subgrade and subbase soils.

[14] also carried out an investigation on lime effects on the strength of subgrade soil using lime percentages of up to 10% on soils around the same region of Southwestern Nigeria, which also confirms that 6% lime application optimally improves soil strength properties, thereby improving the strength.

To investigate the cause of road failure in the study area, it became necessary to carry out proper laboratory geotechnical tests to investigate the nature of the subgrade soil materials, whose results were integrated with the results of Electrical Resistivity Investigation (ERI) for better inference of the area of investigation subgrade layers. From the results of the electrical resistivity survey and laboratory geotechnical tests conducted on the subgrade soil samples, it was observed that the majority of the study area was underlain by incompetent clayey layers.

The essence of stabilizing soil is to change its characteristics for long-term strength and stability [16], which involves the mixing of stabilizing agents (such as lime) with soil to increase its strength and durability. Improving the engineering properties by stabilization results in increasing the soil strength (that is, its shearing resistance), stiffness, durability, and reduction in its swell potential [17]. Generally, fine-grained clay soil with a minimum of 25% passing No. 200 sieve (which is equivalent to a mesh sieve of 75μm by American Society for Testing Materials standards) [22], and a plasticity index greater than 10% is considered okay for stabilization [16].

Lime, as a stabilizing agent, was used in this investigation to stabilize the soil samples with the aim of improving their mechanical strength, with a progressive gradual increase in strength with increasing percentage content of lime ranging between 2% and 10%. Soil with a high percentage of clay content, when treated with lime, improves the soil strength by reducing its liquid limits, plastic limit, and plasticity index while increasing its optimum moisture content (OMC) and maximum dry density (MDD). [18]. This makes lime a suitable stabilizer for poor subgrade soil (with a high clay content) and is thereby recommended for the improvement of clayey subgrade soil for the construction of more durable roads [19].

Study Area

The study area is situated in the southwestern region of Nigeria, specifically between longitudes 3.2008E and 3.2157E and latitudes 6.7400N and 6.7511N. It is within the Ado/Odo Ota local government area of Ogun state, Nigeria, bounded by Ifo LGA to the north, Yewa south LGA to the north western axis, Ipokia LGA to the west, and Lagos to the south, with an annual temperature of 26.5 °C. It lies within coastal plain sands (Benin) formation sharing boundary with the underlying Ilaro formation which consists of both marine, continental massive yellow and poorly consolidated sandstones [20]

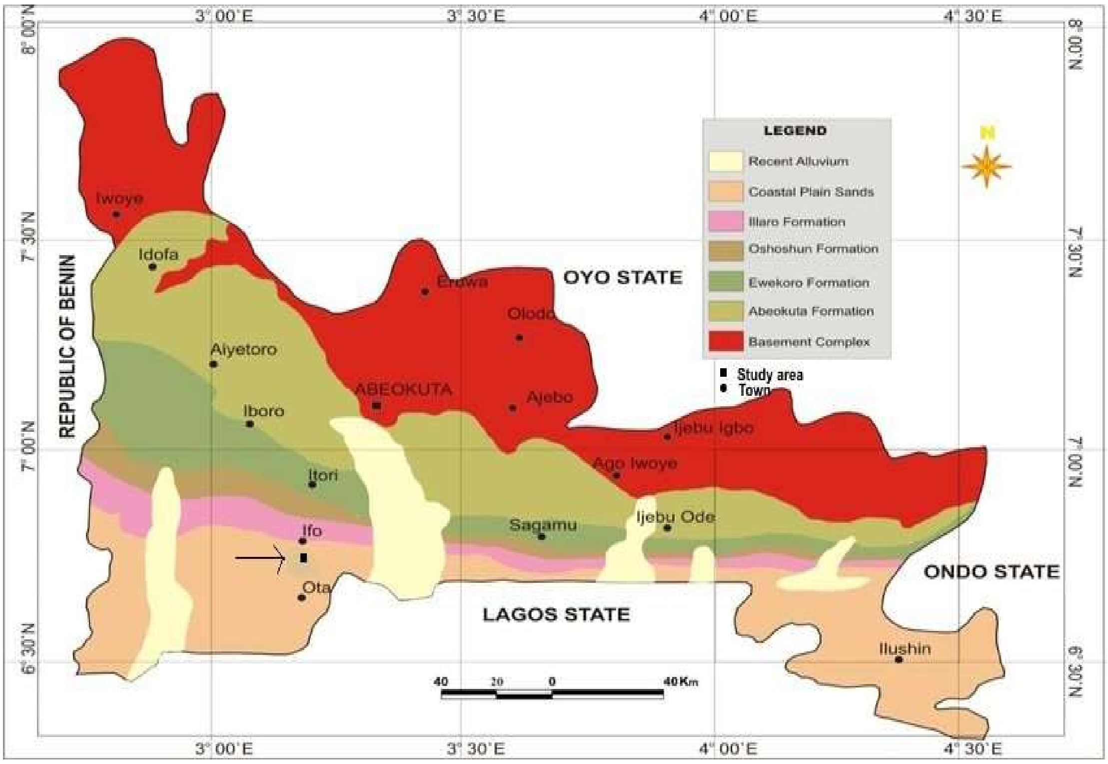

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Figure 2. *Ggeological map of Ogun state; showing the location of the study area [21]*

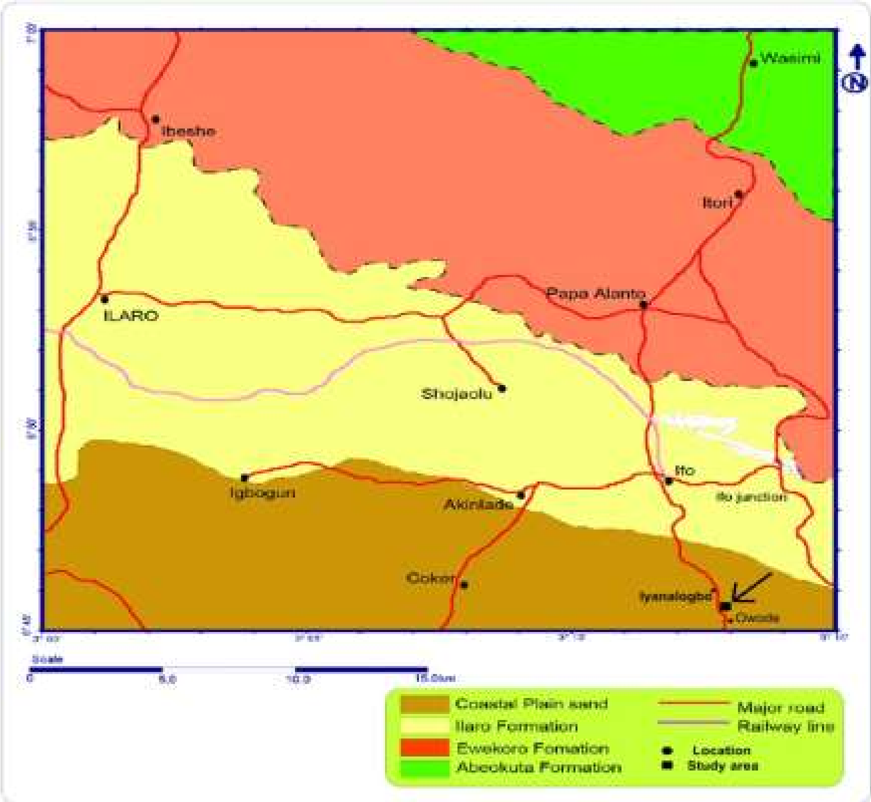

Figure 3*. Geological map of the study area and environs (modified after [20])*

Ado/Odo Ota LGA is strategically located as a link between Ogun state and Lagos state, with a major highway (Lagos-Abeokuta highway) connecting Lagos to Abeokuta, which has significantly boosted economic activities along sections of the highway over the years. The study area falls within the tropical climate zone with an average annual rainfall of 1718 mm with a rainy season between April and November and dry season between November and April.

The major communities along the study area are Owode and Iyanalogbo, with the road stretching more than 1 km in a south northern direction from Lagos towards Abeokuta.

The road profile runs through a relatively flat terrain with a gentle slope increasing gradually in elevation in the northern direction (Figure 4).

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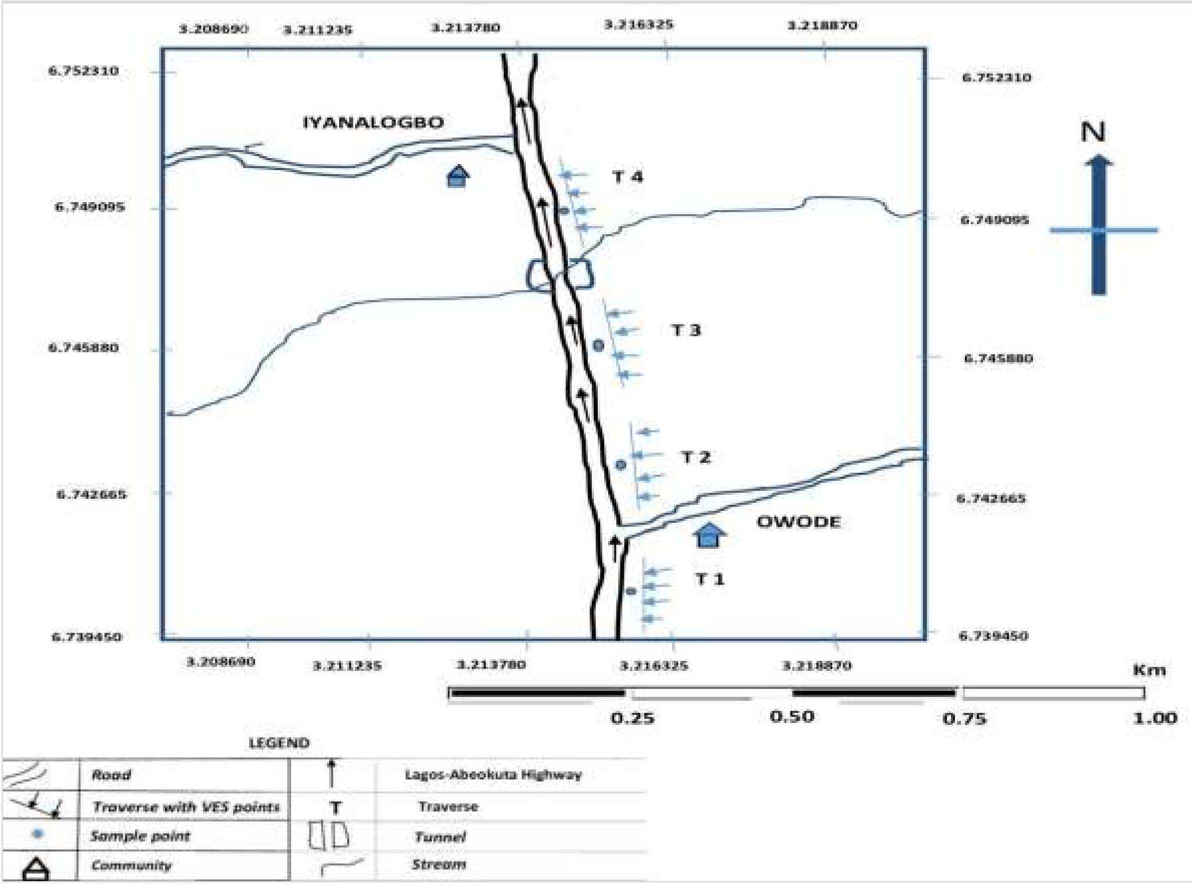

Figure 4. *Base map of the study area*

2 MATERIAL AND METHOD

2.1 Geophysical Investigation

Geophysical investigations which involved electrical resistivity profiling using Wenner array, and vertical electrical sounding using Schlumberger array configuration, were carried out to produce electrical resistivity imaging (ERI) of the subsurface within the four traverses probed, to produce the 2-D pseudo sections and geoelectric sections of the four traverses respectively.

Electrical resistivity imaging for the four profiles under study was produced from the resistivity values obtained by the Wenner array and Schlumberger array configurations for the 2D pseudosections and geoelectric sections, respectively. This same geophysical method was adopted by [20] to image the subsurface lithology for his investigation.

Wenner array configuration entails moving the current electrodes and potential electrodes equally along a straight line after every reading so that the spacing between the electrodes remains equal and takes on certain prescribed values. The outer two electrodes were current electrodes, whereas the two inner electrodes were potential electrodes (Figure 5).

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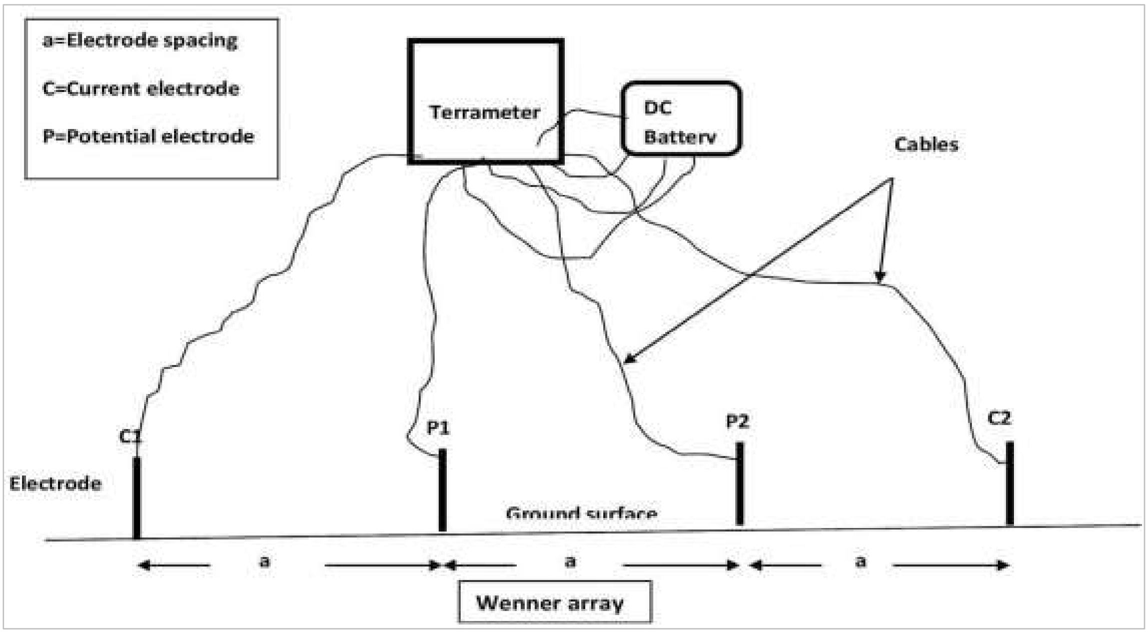

Figure 5. *Thematic sketch of Wenner array set-up*

The apparent resistivity of the medium measured with this array is given by:

$$\boldsymbol{\rho}\mathrm{a} = 2\pi a(\boldsymbol{\Delta}V/I) = 2\pi aR \qquad \textbf{\textit{(1)}}$$

where:

$\boldsymbol{\rho}\mathrm{a}$ apparent resistivity (Ω.m),
I current (A,)
V potential difference (V),
R resistance (Ω),
A spacing between the electrodes (m).

The Palsi resistivity meter recorded the subsurface resistance values, which were multiplied by a K-factor to obtain the apparent resistivity values.

Resistivity survey using Wenner array was carried out on each of the five traverses of 100 m each with electrode spacing a=2 m, 4 m, 6 m, 8 m and 10 m for each depth of investigation, therefore determining the lateral and vertical variation in the apparent resistivity values of the subsurface to a maximum depth of 10 m from the ground surface.

An electrical resistivity imaging software known as DIPRO (version4) was then used to produce the 2D resistivity structures of the traverses surveyed (Figure 7). This electrode configuration system was employed for the 2D imaging of two major Lagos roads to delineate the subsurface subgrade layers [9,10].

The Schlumberger array is also similar to the Wenner array in its configuration, but the difference is in the length between the potential electrodes, which is less than one-fifth of the distance between the current electrodes (Figure 6). It is utilized for vertical electrical soundings to probe the subsurface for the vertical variation in resistivity values and is less labour-intensive.

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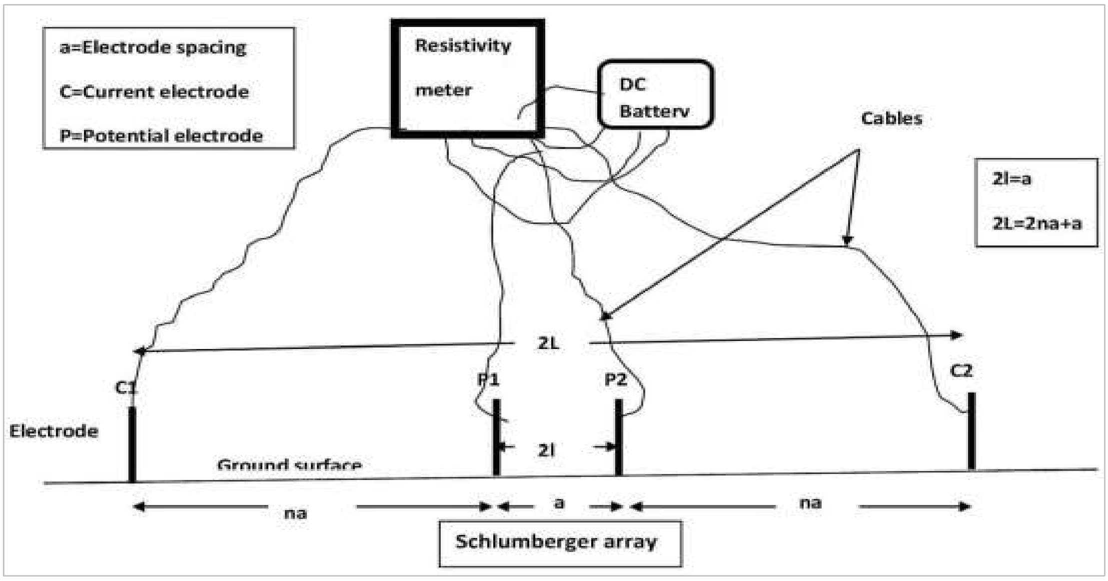

Figure 6. *Thematic sketch of Schlumberger array configuration*

The apparent resistivity for Schlumberger electrode array configuration is given by:

$$\rho a = \pi R(2L/2)^2/\mathbf{2l} \qquad \textbf{(2)}$$

where:

ρa apparent resistivity (Ω.m),
2l distance between potential electrodes (m,)
2L distance between current electrodes (m),
R resistance (Ω).

Surfer 13 software was then used to generate 2-D geoelectric sections of the four traverses using the resistivity values of the lithologic units and their corresponding thicknesses.

2.2 Laboratory geotechnical soil tests

The four bulk subgrade soil samples taken at about 1.4 m using excavator and shovel within each of the four profiles of about 100 m with Global Positioning Stations (GPS) of latitude/longitude: 6.7395179/3.2155710, 6.7427015/3.215441, 6.7443840/3.2151204 and 6.7498861/3.2148599 for profile 1, 2, 3 and 4 respectively. The soil samples were carefully put in sample bags for laboratory geotechnical investigation at Julius Berger's Soil Laboratory Test Department, Shagamu, Ogun State, Southwest Nigeria, and at the Department of Civil Engineering Soil Laboratory, Lagos, Nigeria.

The wet sieving method was adopted for the grain size analysis, which requires that the soil sample is first soaked in water for 24 hours before being sieved and washed through sieve #200 (mesh size 0.075 mm) under running water to eliminate the clay and silt portions.

The samples (1 kg) were oven-dried at 105 °C for 24 hours, weighed on a weighing scale, and pulverized with a pestle and mortar to dislodge clusters in the soil samples. The samples were then soaked in water for 24 hours after which they were washed through sieve #200 under a running tap to remove the silt and clay content. The oven-dried samples were then subjected to grain size analysis according to the ASTM D6913 standard [22]. The fraction that passed through 75 μm was subjected to liquid limit and plastic limit tests, according to ASTM D4318[22].

Specific gravity of soil, which is the ratio of the mass of a given volume of soil to the mass of an equal volume of water at 4 °C, was determined using pycnometer bottle in accordance to ASTM D854-02 [22].

Standard proctor compaction test, following ASTM D1557 [22] was performed on the soil samples to obtain the samples' maximum dry densities at their respective optimum moisture contents.

California bearing ratio (CBR) tests were carried out on the soil samples, compacted at their Optimum moisture content (OMC) for a penetration test according to the standard set by [22]. The soaked CBR test was performed after soaking the samples for 48 hours, following ASTM D1883 [22]. This was performed to ascertain the effect of water ingression on the strength of the subgrade soil. The unconfined compressive test was then carried out on the samples as a compression test to investigate their shear strength; following ASTM D2166-00 [22].

The effect of lime as a stabilizing agent was subsequently investigated by repeated tests for the liquid limits, plastic limits, compaction tests, and compression tests with an admixture of lime with 2%, 4%, 6%, 8%, and 10% replacement, respectively.

3 RESULTS AND DISCUSSION

3.1 Result Presentation

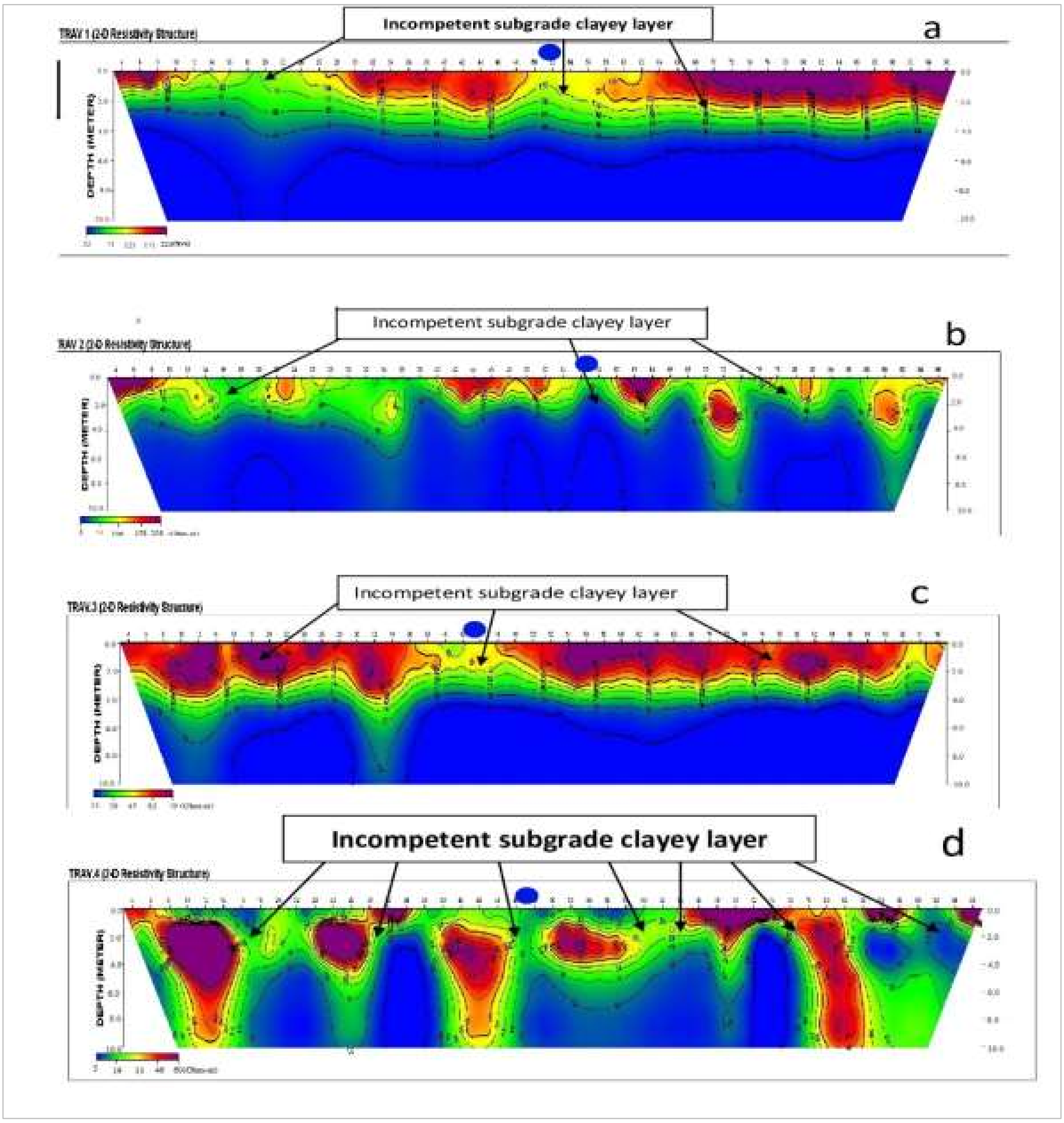

Figure 7. *(a) 2-D pseudo section for traverse 1;(b) 2-D pseudo section for traverse 2; (c) 2-D pseudo section for traverse 3; (d) 2-D pseudo section for traverse 4; with their respective sampling locations as shown*

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Table 1. *Summary of the resistivity values of the subsurface soil for the four traverses*

Traverse	VES No.	No. of layers	Resistivity (Ω.m)	Thickness (m)	Inferred lithology	Curve type
1	1	4	279.3/45.6/8.4/40.2	1.0/7.1//41.3/--	Top soil/ Sandy clay/ Peat/ Clay	QH
	2	4	154.6/41.0/5.1/63.4	1.8/10.3/16.7/--	Top soil/ Sandy clay/ Peat /Sandy clay	QH
	3	4	168.4/47.9/3.2/5.8	1.6/6.1/25.3/--	Top soil/ Sandy clay/ Peat/ Peat	QH
	4	4	331.3/127.3/6.8/357.6	1.0/4.1/16.2/--	Top soil/ Sand/ Peat/ Sand	QH
2	5	4	268.6/50.6/4.0/34.5	1.2/9.3/10.5/--	Top soil/ Sandy clay/ Peat/Clay	QH
	6	4	309.6/79.3/17.6/131.7	1.0/6.6/26.3/--	Top soil/ Clayey sand/ Clay/ Sand	QH
	7	4	313.1/116.3/12.4/124.3	1.3/4.9/30.5/--	Top soil/ Clayey sand/ Clay/ Sand	QH
	8	4	303.9/61.7/7.3/135.4	0.8/2.8/22.7/--	Top soil/ Sandy clay/ Clay/ Sand	QH
3	9	4	94.3/24.8/4.1/35.4	1.6/8.6/24.7/--	Top soil/ Clay/ Peat/ Clay	QH
	10	4	104.2/21.5/5.2/58.2	0.9/5.2/12.1/--	Top soil/ Clay/ Peat /Sandy clay	QH
	11	4	75.3/25.9/181.6/618.8	1.6/22.5/10.7/--	Top soil /Clay/ Sand	HA
	12	4	88.8/65.4/12.1/122.3	0.7/2.5/37.1/--	Top soil/ Sandy clay/ Clay/ Sand	QH
4	13	4	37.3/94.3/21.2/229.8	0.4/2.1/39.2/--	Top soil/ Clayey sand/ Clay/ Sand	KH
	14	3	103.8/36.4/342.4	2.5/41.4/--	Top soil/ Clay/ Sand	H
	15	5	155.8/26.8/244.8/26.1/147.8	0.6/1.9/3.4/18.6/--	Top soil/ Clay/ Sand/ Clay/ Sand	HKH
	16	5	19.6/37.8/275.9/9.1/387.7	0.8/1.3/4.5/21.7/--	Top soil/ Clay/ Sand/ Clay/ Sand	AKH

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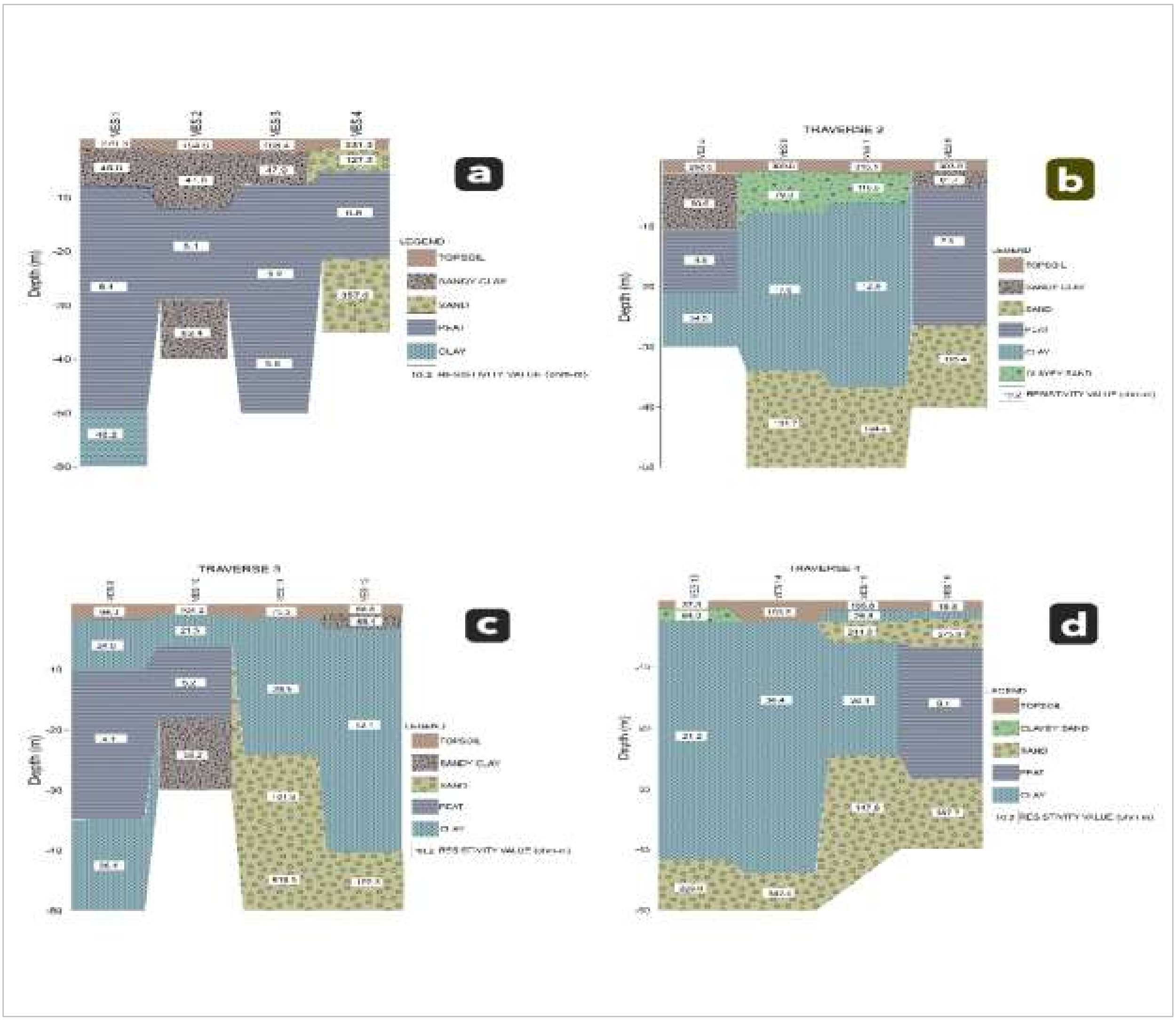

Figure 8. *(a)2-D geoelectric section for traverse 1; (b) 2-D geo-electric section for traverse 2; (c) 2-D geo-electric section for traverse 3; (d) 2-D geo-electric section for traverse 4*

Table 1 shows the summary of the VES resistivity values used to generate the geoelectric sections (Figure 8a–8d)) for the four traverses. The curve types generated using win resist software are QH, HA, KH, H, KQ, HKH and AKH with QH having the predominant curve type (68% of the total curve types).

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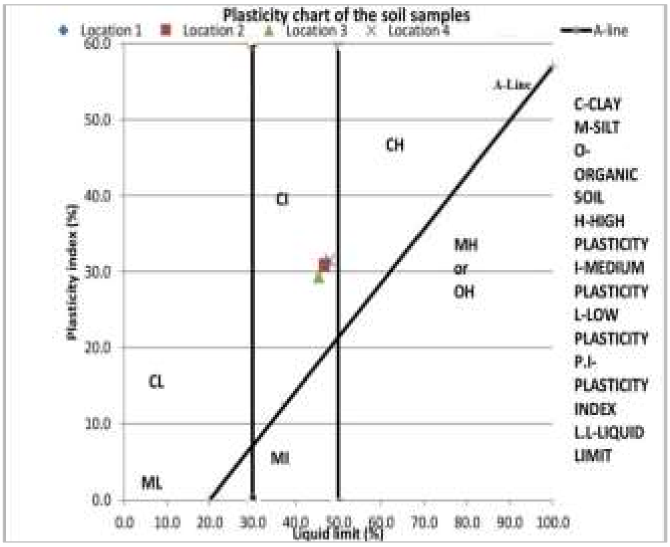

Figure 9. *Casagrande's chart showing the plasticity class of the soil samples*

Table 2. *Geotechnical characteristics of the soil samples*

Sample No.	NMC	LL	PL	PI	Passing #200	MDD	OMC	CBR Unsoaked	CBR Soaked	UCS	S. G
	(%)	(%)	(%)	(%)	(%)	(g/cm^3)	(%)	(%)	(%)	KN/m^2	
1	13.7	47.0	16.0	31.0	45.7	1.7	16.0	27.5	18.0	75.0	2.7
2	13.9	46.8	16.0	30.8	45.0	1.7	16.1	26.2	19.0	74.0	2.7
3	12.9	45.4	16.0	29.4	44.5	1.7	16.0	27.0	21.0	75.0	2.6
4	13.2	48.0	16.5	31.5	41.5	1.7	16.1	26.0	21.0	74.0	2.7

(NMC: Natural moisture content; LL: Liquid limit; PL: Plastic limit; PI: Plasticity index; MDD: Maximum dry density; CBR: California bearing ratio; UCS: Unconfined compressive strength; S.G: Specific gravity)

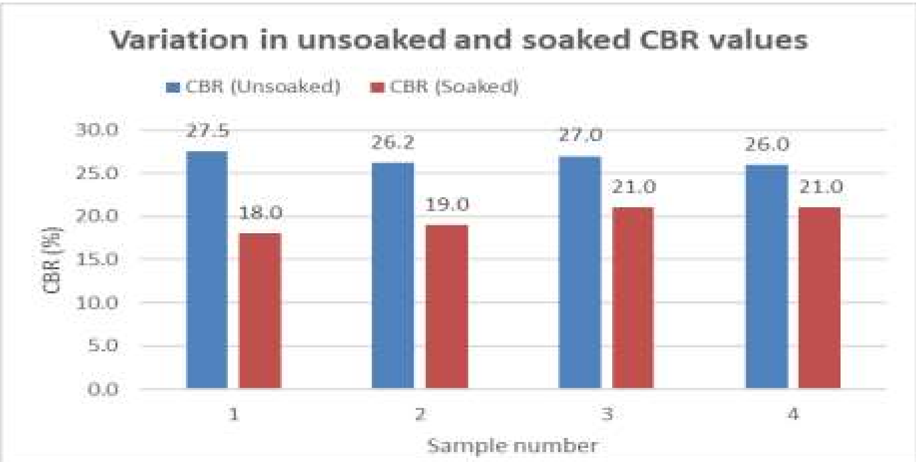

Figure 10. *Variation in unsoaked and soaked CBR values for the soil samples*

Table 3. *Average percentage reduction with lime percentage replacement for the soil samples*

	2%L, 98%S	4%L, 96%S	6%L, 94%S	8%L, 92%S	10%L, 90%S
Liquid Limit (%)	4.4	8.1	11.1	15.7	18.8
Plastic Limit (%)	5.8	9.1	12.0	15.1	17.2

Plasticity index (%)	3.6	7.5	10.4	16.0	19.6

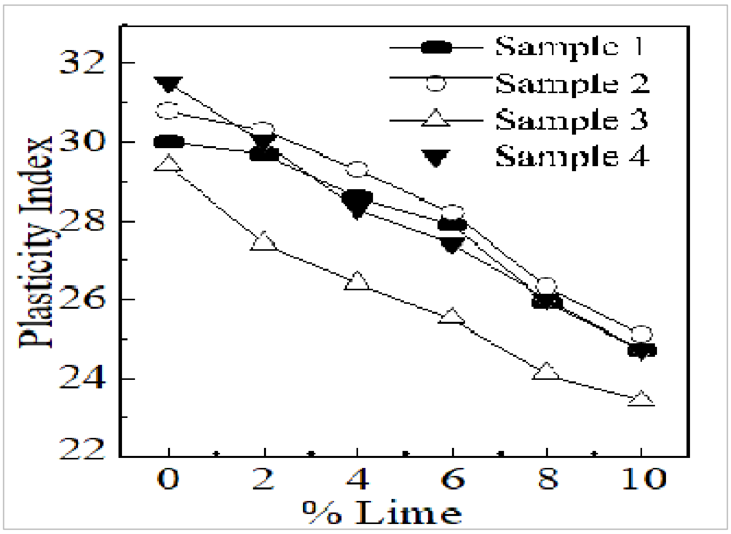

Figure 11. *Variation in PI with % replacement*

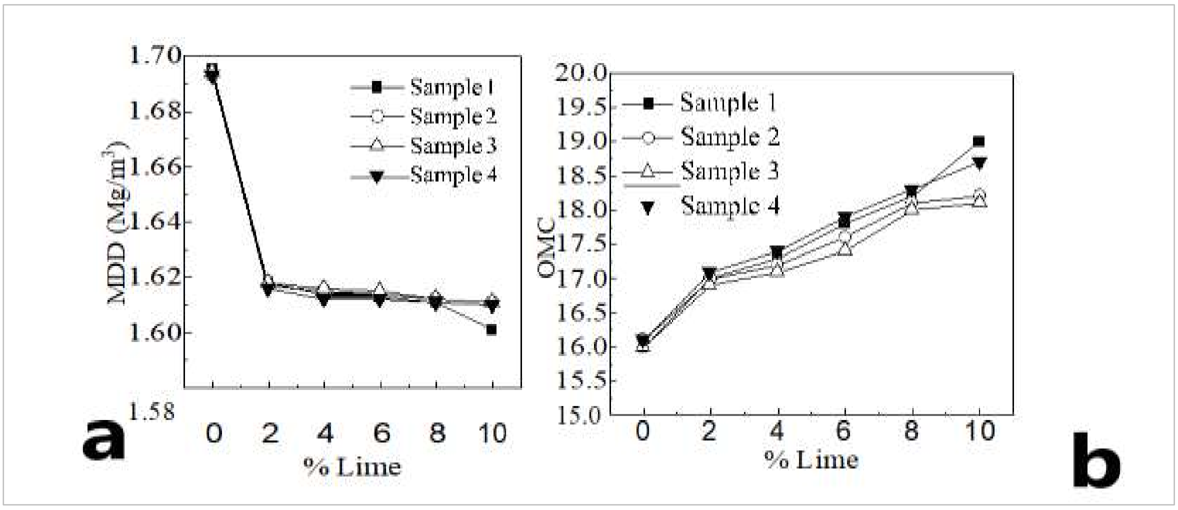

Figure 12. *(a) Variation in MDD with % replacement of lime; (b) Variation in OMC with % replacement of lime*

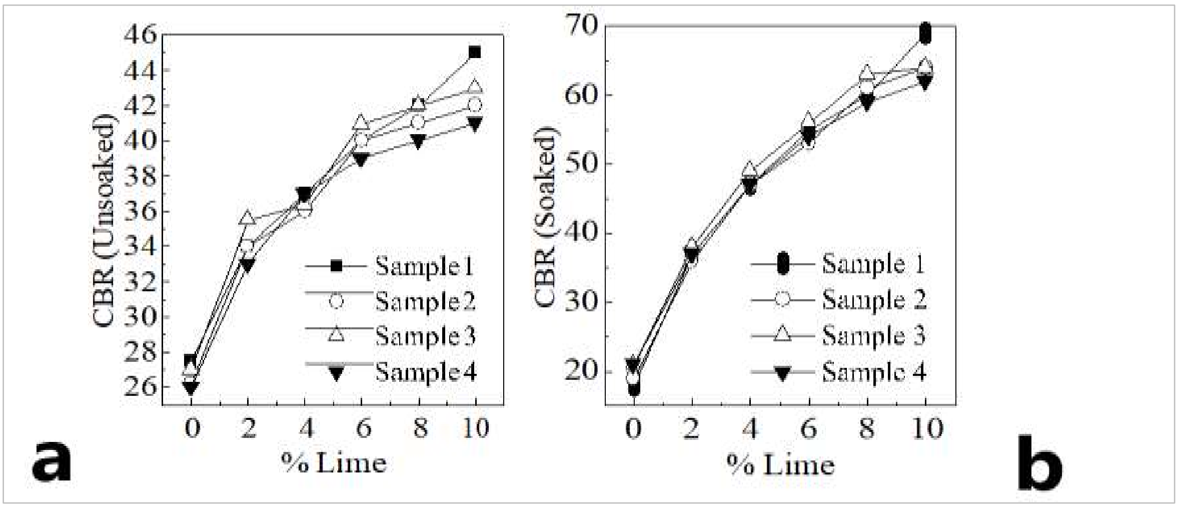

Figure 13. *(a) Variation in CBR (unsoaked) with % replacement of lime; (b) Variation in CBR (soaked) with % replacement of lime*

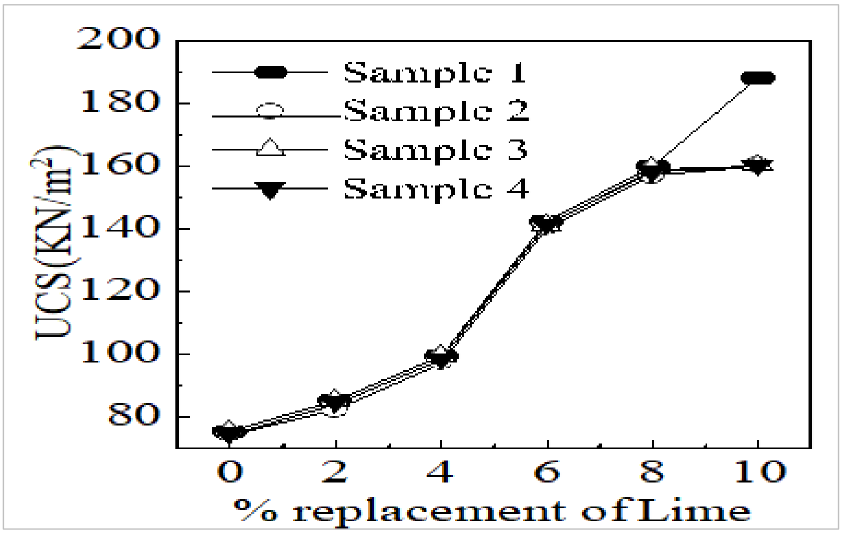

Figure 14. *Variation in UCS with % replacement of lime*

Table 4. *Average percentage increment in the CBR and UCS values with percentage lime replacement*

	2%L, 98%S	4%L, 96%S	6%L, 94%S	8%L, 92%S	10%L, 90%S
CBR (unsoaked) (%)	28.0	37.3	49.9	54.7	60.2
CBR (soaked) (%)	88.1	141.4	177.1	208.9	230.0
UCS (KN/m^2)	12.7	31.9	89.3	112.8	124.1

3.2 Discussion

3.2.1 Electrical resistivity investigation

Because the effects of external factors are mostly determined by the nature and strength of the subgrade soil within 2 m of the ground surface [23], the focus was on the nature of the subsurface lithology within a depth range of 0–10 m.

The 2-D pseudosection along traverse 1 (Figure 7a) shows three lithologic layers with resistivity values of the first layer (topsoil) ranging between 133–177 Ω.m within 27– 49 m along the profile and 144–221 Ω.m within 64–100 m to a depth of approximately 1.8 m which depicts the sand layer.

The underlying second layer's resistivity values range between 66 and 110 Ω.m to the depth of about 2.2 m extending towards the ground surface within 12–25 m and 50–62 m along the profile, depicting clayey sand which is incompetent for engineering road construction.

The third layer with the resistivity values range between 22–44 Ω.m from the depth of 2.2 m depicting clayey soil, which is prone to alternate swelling and shrinkage when it gets mixed with water. This thus makes it unsuitable as a subgrade soil for a durable road construction.

The integrated geoelectric section of the traverse (Figure 8a) reveal similar poor subgrade layer with the first layer having resistivity values ranging between 154.6 and 331.3 Ω.m to a depth of about 1.6 m while the underlying layer have the resistivity value between 41.0 and 47.9 Ω.m across the lateral distance of 0–75 m and to the depth of about 10 m, which depict clay. Traverse 2 was also observed to be of poor subgrade material as shown in the 2-D pseudosection (Figure 7b), which comprises three lithologic layers with the top (first) layer having resistivity values ranging between 142 and 209 Ω.m along 0–8 m, 41–46 m and 60–64 m to the depth of about 1.8 m depicting

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sand while other sections of the top layer are of resistivity values of between 72 and 95 Ω.m, which shows the layer to be clayey sand.

The second layer ranges between 27 and 72 Ω.m to the depth of 2.5 m depicting sandy clay which is not suitable as subgrade layer for geotechnical road construction. The third layer's resistivity values range between 3.0 and 27 Ωm from the depth of 2.5 m and depict peat/clay.

The geoelectric section (Figure 8b.) also reveals the incompetence of the subgrade soil as a good road construction material with the resistivity values between 50.6 and 116.3 Ω.m from the depth of 1.8 m, depicting sandy clay/clayey sand to an average depth of 7 m.

For traverse 3, the three lithologies delineated by the 2-D pseudo section (Figure 7c) are clayey materials, with the resistivity that ranges from 12 to 79 Ω.m. The topsoil with values between 64–79 Ω.m represents sandy clay/clayey sand, the second layer with resistivity values from 27 Ω.m to 42 Ω.m at the depth of about 1.7–5.3 m is composed of clay material which is as well not appropriate for engineering road construction. The bad subgrade soil was also confirmed from the 2-D geoelectric with the top layer's resistivity layer ranging between 75.3 and 104.2 Ω.m to the depth of about 1.8 m which depicts clayey sand/sand. The underlying second layer's resistivity is between 12.1 and 25.9 Ω.m to the depth of about 10 m within the lateral distance of 0–50 m.

The 2-D pseudosection along traverse 4 (Figure 7d) also shows three lithologic layers with top layer having resistivity values from 12 to 44 Ω.m to a depth of about 1.4 m, depicting clay.

The second layer's resistivity values ranging between 15–60 Ω.m to a depth of about 5.5 m depicting clayey soil materials. The traverse's integrated 2-D geoelectric section (Figure 8d) was observed to correlate with the pseudo section, especially at the second layer with resistivity values which range between 21.2 Ohm-m and 94.3 Ω.m. This confirms the profile to be underlain by clay materials subgrade, hence not suitable for the construction of a lasting roadway. The results of the resistivity values from the pseudosections are similar to the ones obtained by [9 and 10] which inferred poor subgrade layers.

The QH dominant curve type in the study area is characterized by sandy clay, clay, peat, and sand with the resistivity values raging between 3.2 and 357.6 Ω.m. (Table 1)

3.2.2 Geotechnical laboratory investigation

The soil samples were discovered to be unsuitable for road construction as subgrade material because they all have clay contents of more than 35% passing through sieve #200. The relatively high fines contents (with more than 35% passing through sieve #200) in the soil samples from traverses 1–4 suggest part of the reasons for the road way failure, which is similar to the findings by [25] who reported higher clay contents in soil samples from a failed road subgrade as part of the reasons for road failure under study.

From the results of the Atterberg's limits determination, the PL, LL and PI are of the average values of 16.1%, 46.8%, and 30.7% respectively across all the soil samples indicating the soil samples are of medium plasticity (Table 2, Figure 9). The LL of the soil samples all fall above 35% which is the acceptable limit for soil that can be used as subgrade or fill during construction of roads in Nigeria, in accordance to specifications by [24]. The liquid limits for the soil samples are also within the range reported by [26] from a proposed road within the Southwestern part of Nigeria. Thus, confirming the subgrade layer to be of poor subgrade soil for road construction, having liquid limit above the threshold of 35%. The MDD of the soil samples compacted at British Standard level of compaction range between 1.693 g/cm3 and 1.695 g/cm3 with an average OMC of 16.05%. (Table 2), which is similar to the low range of maximum dry density (MDD) and optimum moisture content (OMC) values observed by [27] as part of the factors responsible for a failed portion along a major highway within Southeastern part of Nigeria.

[28] classified samples with high value of MDD and low OMC as best suitable for subgrade materials; this therefore reveals the soil samples' MDD and OMC values to be of poor to fair subgrade soil for construction, hence subgrade soil, is therefore not suitable as a competent construction material for a durable road construction.

The CBR results for the unsoaked and soaked test values for the unstable sections vary between 26.2% –27.5% and 18.0% – 21.0%. It shows a range of percentage reduction in the soaked CBR values of between 19.2% and

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34.6%. (Table 2). The CBR values obtained are also similar to the range of values got by [25] for the laterites of a failed road.

The effect of soaking can be seen in the results, as the CBR values for the 48 hour-soaked samples were significantly lower than those of the unsoaked ones due to the addition of moisture dropping the strength of the soil (Figure 10). [29] suggested that soils in their natural form, water might still percolate into the soil's interstitial spaces, weakening it. This suggests that the strength of the soil will significantly reduce by the ingress of water to the subgrade soil.

The UCS of the untreated lateritic soil range between 74–75 KN/m^2 (Table 2), which is similar to the observation of [25] for the laterite of a failed road within the southwestern part of Nigeria.

The specific gravity (SG) of the soil samples varies between 2.63 and 2.69 which fall within the range of lateritic soils in accordance to the standard set by [24], as also reported by [30] with values between 2.65 and 2.68 for the lateritic subgrade and sub base soil used for road construction within the southwestern part of Nigeria. All the specific gravity (SG) values fall above the minimum value for lateritic soil, which confirms that the road is underlain by laterite.

3.2.3 Stabilization effects with lime replacement in 2%, 4%, 6%, 8% and 10%

Effects on Consistency Limits

The addition of lime in varying proportions reduced the soil’s plasticity correspondingly (Figure 11), which causes an improvement in its mechanical strength. The addition of 10% lime reduced the liquid limit by the average percentage values of 18.8%, 17.2%, and 19.6% for the liquid limits, plastic limits and plasticity indices respectively (Table 3).

[32] inferred that the reason for this was because of the decrease in swell potential, the swell potential is reduced as a result of cation exchange, which happens when Ca^{2+} ions from the lime replace weaker cations in the soil, resulting in greater void sealing through particle agglomeration.

Effects on Compaction levels

The results of the MDD and OMC of soil samples mixed with lime showed an inversely proportional relationship between the OMC and MDD (Figure 12). As the OMC increased, the MDD reduced, and as various percentage of stabilizer was added, the resulting moisture content increased causing further reduction in the dry density.

The addition of between 2 % and 10 % by weight of lime resulted in a gradual reduction in the maximum dry density. With the addition of 10 % by weight of lime, the MDD reduced from an average of 1693.75 kg/m^3 (untreated sample) to an average of 1608.25 kg/m^3 as the moisture content increased, showing an average percentage decrease of 5.05 % across all four samples. The results showed that adding lime increased the OMC and reduced the amount of MDD progressively with the increase of stabilizer addition (Figure 12).

The inverse relationship between MDD and OMC with lime increment (decrease and corresponding increase in MDD and OMC respectively) was also reported by [35]. This is also similar to the work of [36], who observed a progressive increase in OMC and decrease in MDD with percent increase in lime; which resulted in the achievement of lower comp active effort of the laterite.

Effects on California bearing ratio (CBR) values

All the soil samples' unsoaked and soaked CBR values increased significantly after stabilization with varied proportions of lime. With lime admixture of 10% replacement, the unsoaked and soaked values were increased by the average percentage values of 60.2% and 230.0%, respectively (Table 4).

The higher percentage increment for the soaked CBR values indicate the bearing strength of lime to increase in the presence of water compared to the absence of water.

This indicates that the stabilizing mix boosted the soil's load bearing capacity. The availability of calcium from the lime for the cementation’s reaction with the silica and iron oxide from the lateritic soil could explain the rise in both wet and unsoaked CBR. The significant rise in values of the soaked sample indicates how sensitive the samples are to moisture. According to [27], the recommended CBR values (after 24 hours of soaking) for subbase soils shouldn't be less than 30% while for subgrade though specifications are silent but 3–10 % is acceptable. The addition of lime beats the benchmark for both subbase and subgrade soil.

Effects on the unconfined compressive strength (UCS)

The response of the different samples to initial addition of lime yielded continuous increase in unconfined compressive strength as shown in figure 14. Addition of lime at 10 % significantly increased the soil shear strength property with the average percentage increase of 124.1% for the soil samples, and the effect of the addition of lime is seen all through the addition, with similar results being obtained by [29].

The rise in UCS values when soil samples were mixed with lime as a stabilizing agent was also reported by [31,33 and 34]. Overall, a significant progressive improvement in mechanical strength was observed in clayey soil with lime increments of 2%, 4%, 6%, 8%, and 10% respectively.

4 CONCLUSION

Many lives and properties have been lost to road accidents as a result of incessant road failures due to incompetent subgrade pavement layers, which also usually lead to increased journey time, traffic gridlock, and high vehicle maintenance cost. This is usually due to the prevalence of high plasticity lateritic subgrade soil, which is unsuitable for road construction because of its susceptibility to internment swelling and shrinkage after the ingression of water.

The lithology of the subsurface along sections of the failed portions of the study area was inferred using both Wenner and Schlumberger arrays; indicating the presence of clayey soil with resistivity values ranging between 17–70 Ωm within the depth of 10 m from ground surface, typical of the soil nature underlying the road section of the study area.

Laboratory tests also showed the presence of clay contents for the samples, with % passing 0.075mm >35% (which is the benchmark for soil suitable for road construction by [24]).

Due to clay's intrinsic properties of high porosity and low permeability with the susceptibility of high swelling rate in the presence of water and shrinkage when dry, and from the average values of 30.7% plasticity index and 44.2% passing through #200 sieve of the soil samples, it is a poor subbase and subgrade material for road construction, which suggests part of the reasons for the incessant road failure of the study area.

Lack of a proper drainage system was also observed to be responsible for water-logged potholes and the cause of erosion in parts of the road, especially during the rainy season.

Adding lime as a stabilizing agent, however, boosted the mechanical strength of the soil samples stabilized. With increasing percentages of lime used as additives, the UCS, CBR (unsoaked) and, CBR (soaked) improved progressively with percentage values increment of 124.1%, 60.2%, and 230.0% respectively at 10% lime admixture, thereby improving the strength of the soil.

Further research is suggested to determine the effectiveness of other stabilizing agents in comparison to lime, so as to determine the one that yields a better result.

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