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Contributions of Engineering Geological Properties of Subgrade Soils to Premature Failure of Major Highways in Southwestern Nigeria: A Case Study of Akure-Ikere Ekiti Highway

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Abstract: Geotechnical analyses were carried out to examine the contributions of engineering geological properties of subgrade soils to the failure of the Akure-Ikere Ekiti road, Southwestern Nigeria. Field observations revealed that the road is in a very poor state of serious deformation and disrepair as most parts of the road alignment have failed. The alignment of the studied road is predominantly underlain by Granite, Charnockites, and Migmatites. Laboratory tests results showed that the natural moisture content ranges from 10.98 to 21.4%, liquid limit from 22.8 to 47.7%, plastic limit from 19.2 to 24 .6, plasticity index 3.6 to 26.3%. The grain size analysis revealed that the amount of fines ranges from 15.9 to 49%. Others are linear shrinkage, between 1.4 and 10%, free swell between 25 and 46%, maximum dry density from 1593 to 2016 kg/m, and CBR between 5 and 48%. The specific gravity ranges from 2.64 to 2.74. With reference to AASHTO classification, 5% of the samples was classified as A-4, 15% classified as A-2-4, 40% classified as A-6, while 40% classified as A-7-6. The dominance of fair-to-good California bearing ratio, fair to good maximum dry density, high linear shrinkage and A-7-6, A-6, and A-2-4 soil groups have combined to give fair-to-good geotechnical properties to the studied soils. Generally, the fair to good geotechnical properties of soil of the road under study is an indication that the contribution of subgrade soil to the failure of the highway is negligible. The total breakdown of the road can be traced to substandard engineering specifications which are complemented by a poor drainage system.

Keywords: Subgrade, geotechnics, failure, compaction, Akure, Ikere-Ekiti.

1. Introduction

Economic importance of roads in Nigeria has been on the increase over the last decades as a result of its affordability and accessibility. Road transportation is complementary to other modes of transportation. However, the deterioration of these roads began soon after construction. Some of the roads in Nigeria do not last up to a few years before the failure becomes pronounced. Road failure has been defined in different ways by various authors. According to Fadaka (1989) [1], a road stretch is said to have failed when part or all parts of the road surface or any part of the road structural section or the entire road prism or pavement at any point along the highway is deformed and rendered non-motorable. Highway failure occurs when the road no longer performs its traditional function of providing a continuous smooth surface with minimal frictional movement of vehicles. When such failure occurs before the anticipated design life, it is best described as premature [2]. Highway premature failure occurs within the highway pavement structure or the road foundation or both. It may also be as a result of problems beside the highway with consequent effects on the pavement and or foundation. Failure of a highway pavement structure or foundations manifests in the form of various categories and degrees of distress on the pavement surface. The failure of roads is a common occurrence all over the world. However, there has been an

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increase in the failure of roads in Nigeria in recent times and in Southwestern Nigeria in particular. Information about the subsoil materials upon which these roads are built are very important. The road under study is a major highway which is in a bad condition as it is characterized by various forms of deformation features that are responsible for pavement failure. These deformations develop on this road pavement in various forms, magnitude, and frequencies. They often cut deep into the structural section of the highway thereby limiting and inhibiting human and vehicular movement. These regular occurrences of failures have not justified the large amount of money spent on its construction. The key to proper maintenance of asphalt pavements is to understand the types and causes of failure and the actions needed for correction before any repair work is done. Consequently, the focus of this study is to determine the engineering geological properties of subgrade soils in the study area, their contributions to road's failure, and possible information that will improve the design and re-construction of the road [3].

1.1 Description of Study Area

The studied road is located within the southwestern part of Nigeria. The road (Fig. 1) is a 25 km stretch which connects Akure to Ikere Ekiti and serves as a link to several other parts of the country. The study area (Fig. 2) is located between longitudes 5°13′E to 6°13′E and latitudes 7°16′N to 7°29′N. The area enjoys a prevalent humid tropical climate marked by alternating wet and dry seasons. The wet season begins in April and ends in October, while the dry season spans from October to March. The mean annual rainfall is about 1375 mm and the rainfall pattern is bimodal in nature, with an annual maximum in June and a secondary maximum in September [4]. Average yearly temperature also varies from 22°C (wet season average) to 30°C (dry season average), while the humidity varies from 40% (December average) to 80% (July average). The entire zone of the study has damp air which comes from the Atlantic Ocean most of the year.

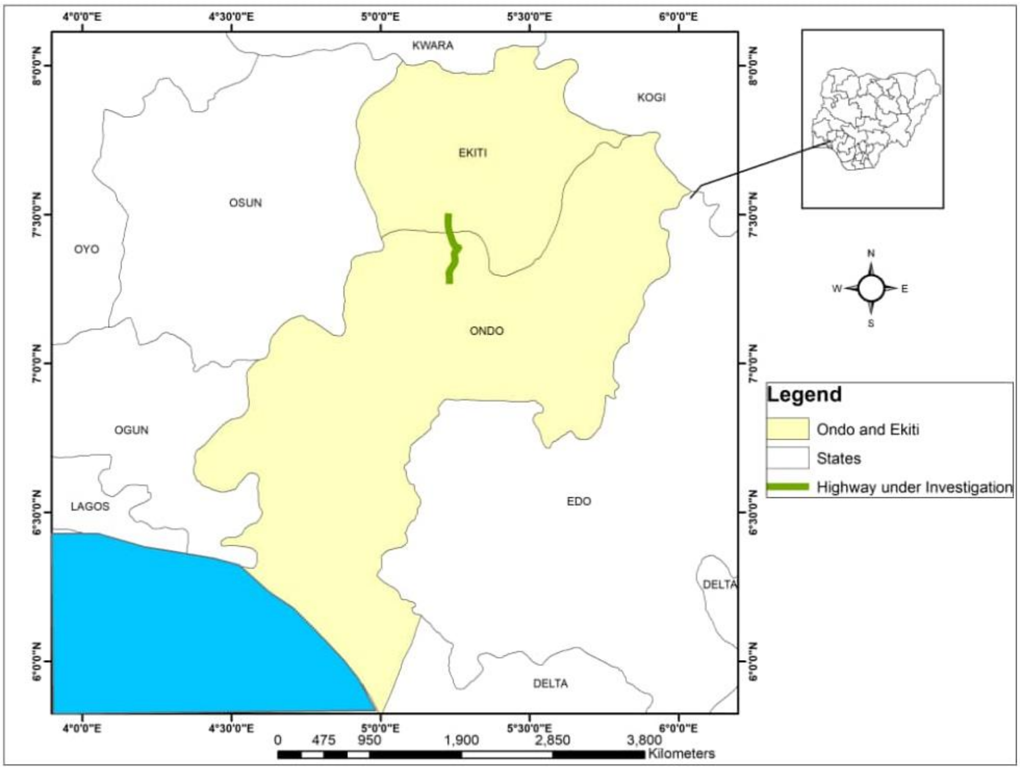

Fig. 1 A Map of Southwestern Nigeria showing the Akure-Ikere-Ekiti Road.

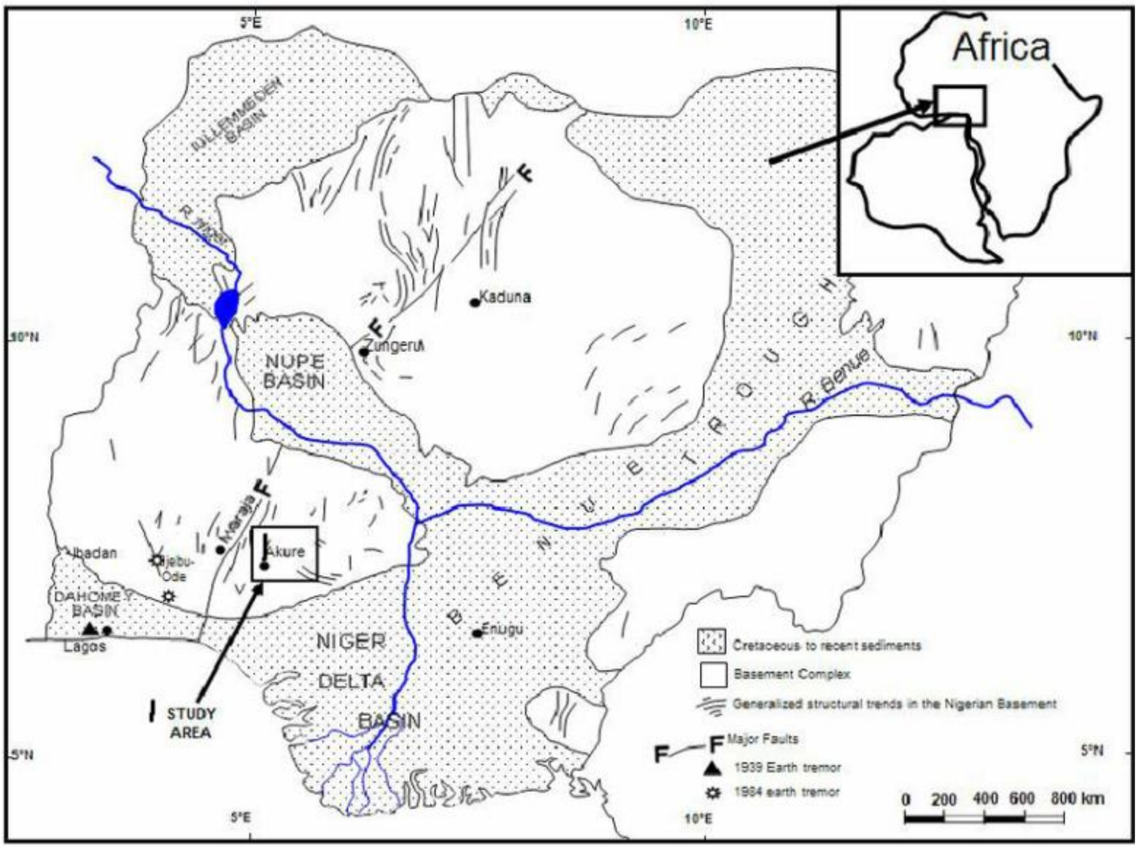

Fig. 2 Geological Map of Nigeria showing location of the Study Area.

1.2 Geology and Soils of the Study Area

The study area is underlain by rocks of the Basement Complex (Precambrian). The area consists mainly of igneous rocks such as granite and charnockites. The granitic rocks occupy a large part of the study area and are porphyritic in nature as shown in the geological map. The area is underlain mainly by Granites, Charnockites, and Migmatites (Fig. 3). These Migmatites generally outcrop sparingly around the Ikere-Ekiti axis of the road. The Migmatites occur as inselbergs and a range of hills [5]. The soils in the study area are residual lateritic materials composed of greyish to reddish brown, loose medium to coarse grained mineral matter with some clayey materials. The soils are however, greyish to reddish brown colour in areas with vegetation cover, probably as a result of decomposing flora and fauna. The relief is of relatively irregular and undulating terrain of crystalline basement rocks (Fig. 4). There are many

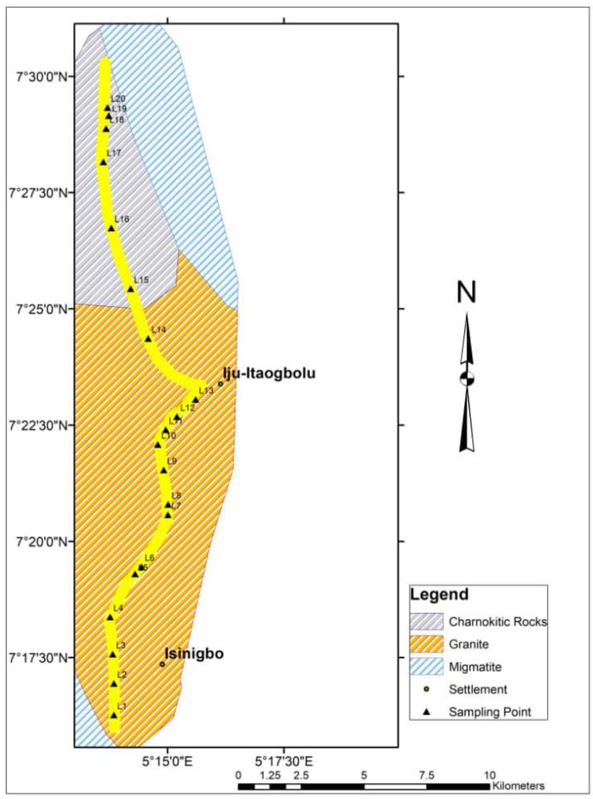

Fig. 3 Geological map showing the study area.

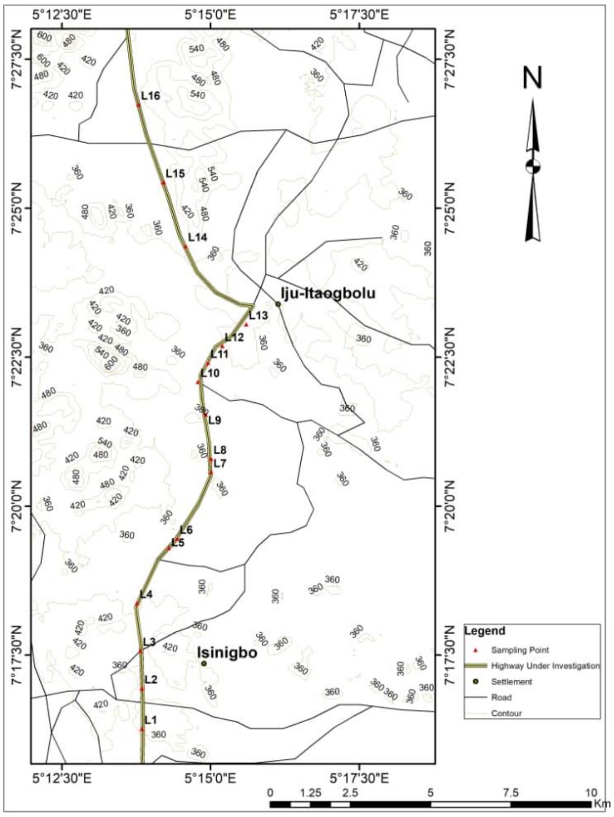

Fig. 4 Topographical Map of the Study Area showing sampling points.

ridges in the study area, in some cases, the rocks occur as inselbergs and have heights varying from 150 m in some places to 570 m in others (Federal Surveys of Nigeria, 1996). The study area is dominated by a dendritic drainage patterns and this suggests a fairly homogeneous resistance of the underlying rocks to weathering.

2. Materials and Methods

During the field work, a visual reconnaissance survey was carried out to evaluate and assess the physical conditions of the highway pavement. The underlying rocks were identified and their structural trends were noted. Twenty bulk disturbed soil samples were collected into polythene bags from subgrade materials in borrow pits along the roadsides adjacent to the failed sections in such a way that the soil samples represent the different topographic conditions of the road alignment. The natural moisture content of each of the samples collected was determined immediately and it was taken to the laboratory. This was followed by air drying of all samples for a week to obtain fairly constant moisture content. The soil tests carried out are Atterberg Limits, linear shrinkage, and free swell. Others include grain size and hydrometer analysis, specific gravity, natural moisture content, compaction, and California Bearing Ratio (CBR). The tests were done following British Standard Institution methods of testing for civil engineering purposes [6].

3. Results and Discussions

3.1 The Condition of the Road Pavement

The road under study is a flexible pavement which consists of the subgrade, subbase, base, and the wearing course. The subgrade being the natural soil represents the foundation of the road. This is directly overlain by the subbase which is made up of soil imported from designated sites. The base of the road is made of aggregates derived from basement rocks from the area while the wearing courses consist of bitumen. As at the time of the study, a major portion of the road alignment has failed (Fig. 5). Pavement distress manifested in various forms such as cracks, pot holes, corrugations, raveling, and rutting along the road alignment (Fig. 5(a), 5(b)). In some portions of the road, cracks, corrugations, and potholes often get to two metres wide and 0.8 m deep (Fig. 5(c)-5(d)). The various types of failure at different locations are described in Table 1. It was also noted that the road is poorly drained.

3.2 Geotechnical Properties of the Soil

Summary of the results of geotechnical tests is presented in Table 14. The values of in situ moisture content will vary depending on the depth of the soil sample, antecedent rainfall (seasons), and prevailing drainage conditions. The seasonal high variation in the moisture contents of soil will cause large volume changes in the clayey soil. Underwood L. B. (1967) [7] proposed that soil with natural moisture content of

(a) An intensely cracked portion of the studied road at Igoba.

(b) A completely broken down portion of the road alignment at Isinigbo.

(c) A completely broken down and corrugated section of the studied Itaogbolu.

(d) An intensely eroded surface exposing the subbase and subgrade at Ikere-Ekiti.

Fig. 5 Major failed portions of the study area.

Table 1 Failure types and their locations on the studied road.

S/N	Failure Type	Location	Remarks
1	Cracks	1, 3	Intensely cracked with the bituminous layer completely damaged.
2	Potholes	18, 19, 20	Bowl shaped defect with average extensive width in some places cutting as deep as 0.8 m in some of the locations. It retains a large amount of rain water and serves as an easy route for the ingress of rainwater.
3	Corrugation and raveling	4, 5	Sections are completely broken down with the structural components of the road completely damaged and removed.
4	Rutting	10, 11, 12	Extensive depressions on the road, retains rain water and aids its ingress into the structure of the road.

5%-15% are suitable engineering material while soil with natural moisture content values ranging from 20% to 35% are unfavorable engineering materials. The values of moisture content obtained for the soil samples range from 10.98 to 21.4 (Table 2). A comparison of the test results with the Underwood proposition indicates that only nine samples have unfavourable to marginally favourable natural moisture content values. The result of the grain size analysis of the samples is presented in Table 3. The dominant grain sizes vary from sandy silt to clayey sand. The clay fraction expectedly exerts a dominant influence on the mass behaviour, even when present in small proportions in any soil sample for engineering use. A comparison of the test results with specification which requires subgrade soil to possess less than 35%

Table 2 Results of moisture content.

Moisture Content (%)			
Sample number	Test 1	Test 2	Average (%)
1	13.35	13.50	13.43
2	10.95	11.01	10.98
3	17.15	17.63	17.39
4	21.15	21.66	21.41
5	11.35	11.65	11.50
6	13.55	13.72	13.64
7	11.75	11.83	11.79
8	14.45	14.66	14.56
9	13.75	13.91	13.83
10	20.15	20.65	20.40
11	14.55	14.75	14.65
12	15.45	15.69	15.57
13	17.75	18.20	17.98
14	16.65	16.94	16.80
15	14.55	14.79	14.67
16	15.55	15.90	15.73
17	11.35	11.39	11.37
18	17.75	18.16	17.97
19	13.50	13.48	13.49
20	15.55	15.74	15.65

Table 3 Results of grain size analysis and hydrometer analysis.

S/no	% clays	Silt %	% fines	% sand	% gravel	Soil group
L1	19.4	16.7	36.2	61.9	2.0	Clayey Sand
L2	14.5	1.4	15.9	82.1	2.1	Clayey Sand
L3	27.7	20.5	48.2	42.0	9.9	Clayey Sand
L4	29.0	18.2	47.2	51.7	1.1	Clayey Sand
L5	26.5	17.6	44.2	52.7	3.1	Clayey Sand
L6	22.7	19.2	41.9	55.0	3.1	Clayey Sand
L7	9.4	14.7	24.1	73.9	2.0	Silty Sand
L8	22.7	14.5	37.2	53.4	9.3	Clayey Sand
L9	21.7	15.3	37.0	59.4	3.6	Clayey Sand
L10	30.4	18.6	49.0	48.0	3.0	Clayey Sand
L11	18.6	19.3	37.9	55.0	7.1	Silty Sand
L12	20.4	19.6	40.0	58.0	1.4	Clayey Sand
L13	26.6	16.5	43.1	53.0	3.9	Clayey Sand
L14	22.8	15.3	38.0	59.9	2.1	Clayey Sand
L15	20.7	17.5	38.2	56.7	5.1	Clayey Sand
L16	25.9	17.1	43.0	50.0	7.0	Clayey Sand
L17	12.1	15.0	27.0	61.0	12.0	Silty Sand
L18	22.3	15.5	37.9	45.1	17.1	Clayey Sand
L19	21.2	14.6	36.1	52.8	11.0	Clayey Sand
L20	22.4	15.7	38.1	57.8	4.0	Clayey Sand

Table 4 Consistency limit results.

Sample code	Liquid limit, W_L	Plastic limit, W_P	Plasticity index, P_I	Arbitrary division by A-line	Plasticity according to liquid limit, WL	BSCS classification code
1	30.3	19.4	10.9	Inorganic clay	Low plasticity	CL
2	24.3	19.3	5.1	Inorganic clay	Low plasticity	CL-ML
3	47.5	21.2	26.3	Inorganic clay	Intermediate plasticity	CI
4	47.7	24.6	23.1	Inorganic clay	Intermediate plasticity	CI
5	36.4	21.2	15.3	Inorganic clay	Intermediate plasticity	CI
6	34.2	22.3	12.0	Inorganic clay	Low plasticity	CL
7	27.3	20.2	7.2	Inorganic clay	Low plasticity	CL-ML
8	34.5	20.6	13.0	Inorganic clay	Low plasticity	CL
9	32.0	21.2	10.8	Inorganic clay	Low plasticity	CL
10	47.2	23.5	23.70	Inorganic clay	Intermediate plasticity	CI
11	32.1	19.4	12.75	Inorganic clay	Low plasticity	CL
12	34.2	19.7	14.50	Inorganic clay	Low plasticity	CL
13	44.1	19.8	24.30	Inorganic clay	intermediate plasticity	CI
14	36.1	19.8	16.30	Inorganic clay	Intermediate plasticity	CI
15	35.0	19.7	15.35	Inorganic clay	Intermediate plasticity	CI
16	41.0	19.9	21.15	Inorganic clay	intermediate plasticity	CI
17	22.8	19.2	3.6	Inorganic clay	Low plasticity	ML
18	41.7	19.8	21.95	Inorganic clay	intermediate plasticity	CI
19	30.0	21.1	8.9	Inorganic clay	Low plasticity	CL
20	32.6	21.1	11.5	Inorganic clay	Low plasticity	CL

Table 5 Results of linear shrinkage.

S/No	LS=1-(Lf/L0)x 100
1	5.0
2	2.9
3	10.0
4	10.0
5	7.9
6	4.3
7	3.6
8	7.1
9	6.4
10	10.0
11	4.3
12	4.3
13	9.3
14	7.9
15	7.9
16	8.6
17	1.4
18	8.6
19	2.9
20	7.9

Table 6 Mean values of specific gravity results.

Sample No.	SG Water
1	2.67
2	2.69
3	2.66
4	2.68
5	2.65
6	2.64
7	2.65
8	2.64
9	2.65
10	2.66
11	2.64
12	2.65
13	2.64
14	2.64
15	2.74
16	2.71
17	2.66
18	2.69
19	2.69
20	2.68

fines, 15% of the samples met the requirement while 85% did not fulfill the requirement. The hydrometer analysis is a continuation of the sieve size analysis used for fine grain sizes of the samples. The result of the hydrometer analysis for the studied soils is shown in Table 3 as continuation of the grain size distribution curves. The percentage of clay ranges from 9.4% to 30.4% and of silt ranges from 1.4% to 20.5%. The consistency limits give much information on the behaviour of clay than the grain size data and it is thus an important factor in the selection of the subgrade and subbase materials. Excessive plasticity often leads to waviness, which is a road failure that results from the plastic flow of soil upon the application of an axle load. The liquid limit values range from 22.8 to 47.7% (Table 4). Soils with very high liquid limits tend to possess low bearing capacity. The specification for road materials recommends that for a material to be suitable as a subgrade, it should possess a liquid limit less than 40%, only 30% of the studied samples do not satisfy this specification, while the other 70% satisfy the specification. The plastic limit values range from 19.2% to 24.6% while the plasticity index ranges from 3.6% to 26.3%. Only 30% of the studied samples do not satisfy the Nigerian specification that recommends a plasticity index of less than 20%. The rest of the samples meet the standard recommended. According to Ola (1983a) [8], the soils would exhibit low to medium swelling potential as they possess plasticity index values lower than 25%. The results were plotted on the plasticity chart, all soil samples plots in the field of inorganic clays. 55% of the samples possess low plasticity while the rest 45% falls within the field of medium plasticity. According to Cassagrande (1947) [9], soil samples with low, medium, and high plasticity will have low, medium, and high compressibility, respectively. The characteristics of the soil groups are summarized in Table 14. Results of the soil classification show that only 15% of the tested soils classify as A-2-4 soils, while the rest 85% classify as A-7-6, A-4, A-6 and A-7 soils (Table 13). The linear shrinkage values of the soils range from 1.4 to 10.0%. Soils having linear shrinkage values greater than 8% will be active and have critical swelling potential and are not good foundation materials [10], 30% percent of the samples have linear shrinkage values greater than 8% and are expected to be unsuitable subgrade materials (Table 5). Gidigasu (1973) [11] noted that soils possessing linear shrinkage greater than 10% will pose a field compaction problem, hence 15% of the samples may pose field compaction problems. Specific gravity is known to correlate with the mechanical strength of lateritic aggregates and may be used as a basis for selecting suitable highway pavement construction materials particularly when used with other pavement construction materials [12]. The values of the specific gravity of the studied soil in water range from 2.64 to 2.74 (Table 6). Activity of the soil was obtained using this procedure by combining Atterberg limits and clay content into a single parameter. The degree of colloidal activity is expressed by the ratio of plasticity index to the percentage of the soil fraction finer than two microns. Skempton (1953) [13] suggested three classes of clays based on their activity viz; inactive clays with activity values less than 0.75, normal clay with activity values ranging between 0.75 and 1.25 and the active clays with activity values greater than 1.25. The activity values (Table 7) ranged from 0.35 to 0.98, which suggest normally active to inactive clays; hence they possess low to medium expansion potential as obtained in the activity chart.

The free swell values (Table 8) of the tested soils fall in the range of illite and kaolinite. Hence, they possess low swelling potential [14].

Soil having free swell index > 200	Very high
Free swell index between 100 and 200	High
Free swell index between 50 and 100 …	Medium, and
Free swell index < 50	Low.

The maximum dry density obtained from compaction tests ranges from 1593 to 2016 kg/m and the optimum moisture content ranges from 11.4 to

22.4% for the soil samples (Table 9). Based on Wood (1938) [15] proposition, the soil samples have fair to good foundation characteristic. This relatively fair to good maximum dry density value is expectedly not a contributory factor to the frequent occurrence of road failure (Table 10). The Federal Ministry of Works and

Table 7 Activity values of tested soils.

Sample code	Plasticity index (%)	Clay content (%)	Activity	Remarks
1	10.90	19.4	0.56	Inactive
2	5.05	14.5	0.35	Inactive
3	26.30	27.7	0.95	Normal
4	23.10	29.0	0.80	Normal
5	15.25	26.5	0.58	Inactive
6	11.95	22.7	0.53	Inactive
7	7.15	9.4	0.76	Normal
8	13.90	22.7	0.61	Inactive
9	10.80	21.7	0.50	Inactive
10	23.70	30.4	0.78	Inactive
11	12.75	18.6	0.69	Inactive
12	14.50	20.4	0.71	Inactive
13	24.30	26.6	0.91	Normal
14	16.30	22.8	0.71	Inactive
15	15.35	20.7	0.74	Inactive
16	21.15	25.9	0.82	Normal
17	3.60	12.1	0.30	Inactive
18	21.95	22.3	0.98	Normal
19	8.90	21.2	0.42	Inactive
20	11.50	22.4	0.51	Inactive

Table 8 Free swell values of the studied samples.

Sample no	Free swell (%)
1	34
2	46
3	26
4	34
5	33
6	36
7	37
8	36
9	41
10	30
11	38
12	41
13	25
14	36
15	41
16	30
17	40
18	34
19	40
20	37

Table 9 Compaction classification [15].

Maximum dry density (kg/m^3)	General value as a foundation
Over 2082.6	Excellent
1922.4-2082.6	Good
1762.2-1922.4	Fair
1602-1762.2	Poor
1121.4-1602	Very poor

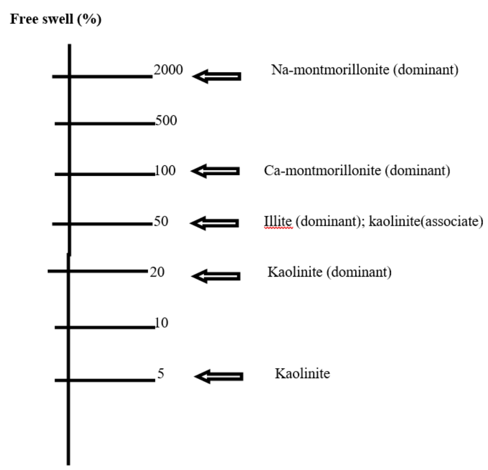

Fig. 5 Free swell value of clay minerals [16].

Table 10 Compaction characteristics of the soil samples.

Sample no	Maximum dry density Kg/m^3	Optimum moisture content (%)
1	1898	14.5
2	2016	11.4
3	1746	18.4
4	1593	22.4
5	1988	12.1
6	1902	14.4
7	1968	12.3
8	1864	15.0
9	1910	14.2
10	1632	21.3
11	1852	15.3
12	1816	16.2
13	1746	18.4
14	1980	12.3
15	1882	15.3
16	1844	16.3
17	1980	12.3
18	1746	18.4
19	1906	14.3
20	1824	16.4

Table 11 Results of the CBR test.

Sample no	CBR%	Remarks
1	16	Suitable
2	48	Suitable
3	7	Unsuitable
4	5	Unsuitable
5	34	Suitable
6	30	Suitable
7	38	Suitable
8	28	Suitable
9	32	Suitable
10	6	Unsuitable
11	24	Suitable
12	23	Suitable
13	8	Unsuitable
14	33	Suitable
15	29	Suitable
16	28	Suitable
17	44	Suitable
18	9	Unsuitable
19	29	Suitable
20	26	Suitable

Housing recommends a California Bearing Ratio of greater than 10% for subgrade materials (Table 11). The laboratory test result shows that the California bearing ratio values of only five of the soils are lower than this value. The relatively fair to good California bearing ratio value for the studied soils are not responsible for the failure of highway pavement in the study area (Table 12).

3.3 Relationship Between California Bearing Ratio, Maximum Dry Density and Optimum Moisture Content

The relationship between California Bearing Ratio, Maximum Dry Density and Optimum Moisture Content values from sample results showed that L3, L4, L10, L13 and L18 with poor to fairly good California Bearing Ratio rating possess high Optimum Moisture Content and low Maximum Dry Density (Table 10). Samples with good to excellent California Bearing Ratio rating have relatively high Maximum Dry Density and low Optimum Moisture Content values. California Bearing Ratio decreases with the increase in the Optimum Moisture Content of soil samples but increases with the increase in the Maximum Dry Density (Figs. 6 & 7).

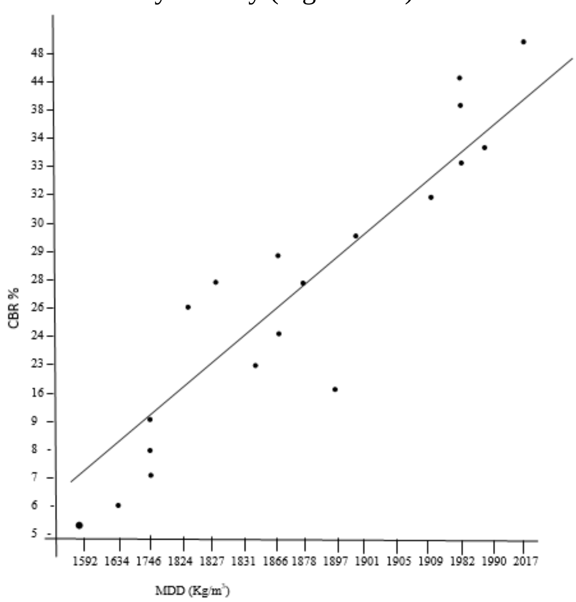

Fig. 6 Effect of MDD (Kg/m^3) on CBR values.

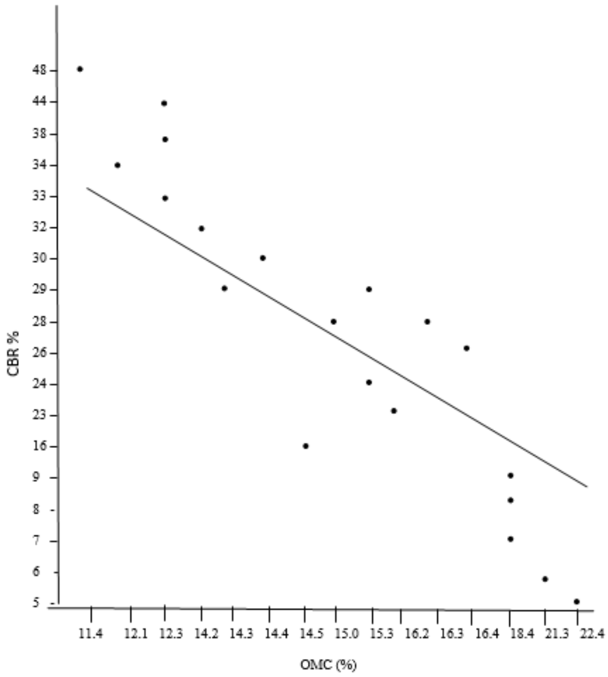

Fig. 7 Effect of OMC (%) on CBR values.

Field observations and various tests carried out on the disturbed soil samples collected from the failed sections of the road revealed that road failures in the study area are due to the following reasons:

(i)The lack of provision for drainage of the highway led to a reduction in the strength characteristics of the subgrade as a result of ingress of water.

(ii) Linear shrinkage values of most of the soils are less than 10% as this means that there will not be shrinkage problems.

(iii) Based on AASHTO classification, the majority of the soils classify as A-4, A-6, and A-7-6 soil which have fair to good subgrade properties.

(iv) Fair to good CBR values of the tested samples cannot be the major cause of failure in the sections with CBR values higher than the 10% limit by the Federal Ministry of Works and Housing 1974. Only 25% Samples have CBR values less than 10%.

4. Conclusions and Recommendations

4.1 Conclusion

Table 12 CBR values and general rating.

S/N.	Sample code	Optimum Moisture content (%)	Maximum dry density (kg/m^3)	CBR (%)	General rating	Use
1	L1	14.5	1897	16	Fair to good	S5 Subgrade
2	L2	11.4	2017	48	Good	Sub-base/S6 Subgrade)
3	L3	18.4	1746	7	Poor	S3 Subgrade
4	L4	22.4	1592	5	Very poor	S3 Subgrade
5	L5	12.1	1990	34	Good	Sub-base/S6 Subgrade)
6	L6	14.4	1901	30	Excellent	S5 Subgrade
7	L7	12.3	1982	38	Good	Sub-base/S6 Subgrade)
8	L8	15.0	1878	28	Excellent	S5 Subgrade
9	L9	14.2	1909	32	Good	Sub-base/S6 Subgrade)
10	L10	21.3	1634	6	Poor	S3 Subgrade
11	L11	15.3	1866	24	Excellent	S5 Subgrade
12	L12	16.2	1831	23	Excellent	S5 Subgrade
13	L13	18.4	1746	8	Poor	S4 Subgrade
14	L14	12.3	1982	33	Good	Sub-base/S6 Subgrade)
15	L15	15.3	1866	29	Excellent	S5 Subgrade
16	L16	16.3	1827	28	Excellent	S5 Subgrade
17	L17	12.3	1982	44	Good	Sub-base/S6 Subgrade)
18	L18	18.4	1746	9	Fair to good	S4 Subgrade
19	L19	14.3	1905	29	Excellent	S5 Subgrade
20	L20	16.4	1824	26	Excellent	S5 Subgrade

Table 13 AASHTO (1945) soil classification.

General classification	Granular materials (35% or less passing No 200 sieve)										Silt-clay materials (more than 35% passing No 200 sieve)
Group classification	A – 1		A – 3	A-2				A-4	A-5	A-6	A-7 A-7-5 A-7-6
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
Sieve analysis, percentage passing: No 10 sieve No 40 sieve No 200 sieve	 50max 30max 15max	 50max 25max	 51max 10max	 35max	 35max	 35max	 35max	 36min	 36min	 36min	 36min
Characteristic of fraction passing No 40 sieve Liquid limit (%) Plasticity index (%)	 6max		 NP	 40max 10max	 41min 10max	 40max 11min	 41min 11min	 40max 10max	 41min 10max	 40max 11min	 41min 11min
Group index	0		0	0		4max		8max	12max	16max	20max
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand					Silty soils		Clayey soils
General rating as subgrade	Excellent to good					Fair to poor					
Ratings of the tested samples				2,7,17				19		1,5,6,8,9, 11,12, 20	3,4,10,13,14, 15,16, 18

Table 14 Summary of the geotechnical properties.

	Table 14 summary of soil properties obtained																		
S/n	% Clay	% Silt	% Fines	% Sand	% Gravel	% MC	Free swell (%)	% LS	Sg water	Liquid Limit	Plastic Limit	PI	Activity	MDD (Kg/m^3)	OMC	CBR %	USCS	AASHTO	Soil group
1	19.4	16.7	36.2	61.9	2.0	13.43	34	5.0	2.67	30.3	19.4	10.90	0.56	1898	14.5	16	CL	A-6	Clayey Sand
2	14.5	1.4	15.9	82.1	2.1	10.98	46	2.9	2.69	24.3	19.3	5.05	0.35	2016	11.4	48	CL-ML	A-2-4	Clayey Sand
3	27.7	20.5	48.2	42.0	9.9	17.39	26	10.0	2.66	47.5	21.2	26.30	0.95	1746	18.4	7	CI	A-7-6	Clayey Sand
4	29.0	18.2	47.2	51.7	1.1	21.41	34	10.0	2.68	47.7	24.6	23.10	0.80	1593	22.4	5	CI	A-7-6	Clayey Sand
5	26.5	17.6	44.2	52.7	3.1	11.50	33	7.9	2.65	36.4	21.2	15.25	0.58	1988	12.1	34	CI	A-6	Clayey Sand
6	22.7	19.2	41.9	55.0	3.1	13.64	36	4.3	2.64	34.2	22.3	11.95	0.53	1902	14.4	30	CL	A-6	Clayey Sand
7	9.4	14.7	24.1	73.9	2.0	11.79	37	3.6	2.65	27.3	20.2	7.15	0.76	1968	12.3	38	CL-ML	A-2-4	Silty Sand
8	22.7	14.5	37.2	53.4	9.3	14.56	36	7.1	2.64	34.5	20.6	13.90	0.61	1864	15.0	28	CL	A-6	Clayey Sand
9	21.7	15.3	37.0	59.4	3.6	13.83	41	6.4	2.65	32.0	21.2	10.80	0.50	1910	14.2	32	CL	A-6	Clayey Sand
10	30.4	18.6	49.0	48.0	3.0	20.40	30	10.0	2.66	47.2	23.5	23.70	0.78	1632	21.3	6	CI	A-7-6	Clayey Sand
11	18.6	19.3	37.9	55.0	7.1	14.65	38	4.3	2.64	32.1	19.4	12.75	0.69	1852	15.3	24	CL	A-6	Silty Sand
12	20.4	19.6	40.0	58.0	1.4	15.57	41	4.3	2.65	34.2	19.7	14.50	0.71	1816	16.2	23	CL	A-6	Clayey Sand
13	26.6	16.5	43.1	53.0	3.9	17.98	25	9.3	2.64	44.1	19.8	24.30	0.91	1746	18.4	8	CI	A-7-6	Clayey Sand
14	22.8	15.3	38.0	59.9	2.1	16.80	36	7.9	2.64	36.1	19.8	16.30	0.71	1980	12.3	33	CI	A-7-6	Clayey Sand
15	20.7	17.5	38.2	56.7	5.1	14.67	41	7.9	2.74	35.0	19.7	15.35	0.74	1882	15.3	29	CL	A-7-6	Clayey Sand
16	25.9	17.1	43.0	50.0	7.0	15.73	30	8.6	2.71	41.0	19.9	21.15	0.82	1844	16.3	28	CI	A-7-6	Clayey Sand
17	12.1	15.0	27.0	61.0	12.0	11.37	40	1.4	2.66	22.8	19.2	3.60	0.30	1980	12.3	44	ML	A-2-4	Silty Sand
18	22.3	15.5	37.9	45.1	17.1	17.97	34	8.6	2.69	41.7	19.8	21.95	0.98	1746	18.4	9	CI	A-7-6	Clayey Sand
19	21.2	14.6	36.1	52.8	11.0	13.49	40	2.9	2.69	30.0	21.1	8.90	0.42	1906	14.3	29	CL	A-4	Clayey Sand
20	22.4	15.7	38.1	57.8	4.0	15.65	37	7.9	2.68	32.6	21.1	11.50	0.51	1824	16.4	26	CL	A-6	Clayey Sand

(v) The fair to good maximum dry density values of the soil samples classify them as suitable highway foundation materials. This fairly good compaction characteristic cannot result in the failure of the sections of the road.

4.2 Recommendations

Results from laboratory tests carried out on the soils indicated fairly good compaction values of the studied soils. It is thus recommended that the use of heavy and more powerful compaction equipment for such soils to improve the dry density values of the soils in the field.

The fair-to-good strength characteristics of the soils as indicated by the fairly good CBR values recorded by some of the soil shows that the soil falls within the Nigerian standard of greater than 10% for subgrade materials, therefore stabilization of only the substandard soils is recommended.

Poor drainage facilities have also led to failure of the sections of the road stretch which led to the reduction of the strength of the soil as a result of water entering into the soil. Very good drainage and side ditches should be provided immediately after the construction of roads.

An engineering geological investigation prior to the construction of roads is also recommended as it helps to foresee any unforeseen problems and identify the problematic areas. Post construction maintenance schemes should also be encouraged so as to prolong the lifespan of the road. Improvements should be made in the area of engineering specifications.

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