GEOTECHNICAL INVESTIGATION OF SOME FAILED SECTIONS ALONG OSOGBO-AWO ROAD, OSUN-STATE, SOUTHWESTERN NIGERIA.

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ABSTRACT

This study aimed at investigating the possible causes of the persistent road failure along Osogbo – Awo road of Osun State, Southwestern Nigeria. Following a geological mapping of the study area, soil samples were collected from different locations where the road had failed severely. The samples were subjected to geotechnical analyses such as particle size analysis, natural moisture content, specific gravity, Atterberg limits, compaction and California Bearing Ratio (CBR). Results of the geological mapping revealed that the road pavement is underlain by schist with pegmatite intrusion, whose minerals have weathered into expansive clay. Laboratory analyses revealed that the soils around these area are wet, with most of them having natural moisture content of above 15%, fine content ranging from 11.2 to 86.4%, liquid limit of 32.37 to 75.0%, plastic limit of 22.71 to 56.00%, plasticity index of 9.63 to 31.13% and linear shrinkage of 1.43 – 27.86%. The soils fall within the A-2-7 to A-7-5 of the AASHTO classification scheme. The specific gravity of 2.20-2.73 showed a lower degree of laterization. The Maximum Dry Density, (MDD) ranged from 1.32– 2.15 Mg/m^3 while Optimum Moisture Content, (OMC) ranged from 13.01 - 27.98%, The California Bearing Ratio, (CBR) ranged from 6.10 to 8.21 for soaked and 8.93 – 18.05 for unsoaked samples. The study concluded that the clayey nature of the road pavement subgrade and the poor geotechnical properties of the soils which were generally below the specifications of the Federal Ministry of Works and Housing could largely be responsible for the persistent road failures.

Keywords: Geotechnical Analyses, Road Failure, Osogbo-Awo Road

INTRODUCTION

The widespread deterioration and failure of Nigerian roads have been attributed to the indiscriminate use of lateritic soils without full knowledge of their limitations (Gidigasu, 1976). Studies have shown that some lateritic soils due to their mode of formation (genesis), their geotechnical properties and field behavior could differ considerably from soils of similar particle-size distribution and plasticity characteristics developed from the same parent rocks. This emphasizes the need for proper knowledge of residual soils used in road construction, because the lithology of the underlying parent rocks play significant role in the geotechnical performance of the lateritic soil (Gidigasu, 1976; Adeyemi, 1992).

Some road pavements in the country were poorly constructed and were largely founded on problematic soils such as expansive clay soils leading to pavement failure. Rehabilitation/reconstruction of such failed road network has become a huge financial burden to the Government. Billions of naira are spent annually on reconstruction and rehabilitation of roads across the country.

Several investigations have been carried out on road failures in southwestern Nigeria and investigations are still being carried out on the geotechnical properties of the soils in this region.

Osinowo *et al.* (2011), carried out an integrated geophysical investigation of the failed portion of a selected roads in the Basement Complex of the Southwestern Nigeria. The authors identified anomalously low resistivity zones and delineated some major and minor linear features beneath the failed road pavement. The authors opined that the interaction of the sub grade and sub base soils with water from numerous fractures in the basement rock had greatly reduced the subsoil shear strengths leading to incessant failure of the overlying pavements.

Adewoye *et al.* (2004) observed that the flexible highway failures along the Oyo-Ogbomoso road, which manifested as waviness/corrugation rutting and potholes were due to environmental factors such as poor drainage, lack of maintenance and misuse of the highway pavement. They also observed that runoff, due to precipitation, largely found its way into the pavement structure damaging them in the process.

The Osogbo-Awo highway in Osun-State, Nigeria experiences persistent road pavement failure. This study intends to investigate the possible causes of the road failure.

Location and Geology of the Study Area

The studied highway is located within Latitudes $N07^{o}45^{1}$ - $N07^{o}48^{1}$ and Longitudes $E004^{o}25^{1}$ – $004^{o}29^{1}$. The road is about 45 km long and trends approximately NE –SW (Fig. 1). The road connects several towns, villages, farm settlements and markets. It serves as a link between Osogbo and Ibadan which makes the road very busy all year round.

The study area is underlain by Precambrian Basement Complex rocks which include; pegmatised schist, synite and quartzite (Fig. 2). The schists were intruded by pegmatite (pegmatised schist) and constitute the dominant rock type beneath the highway. Schist chemically weathers into clay which is a poor sub-grade/sub-base soils.

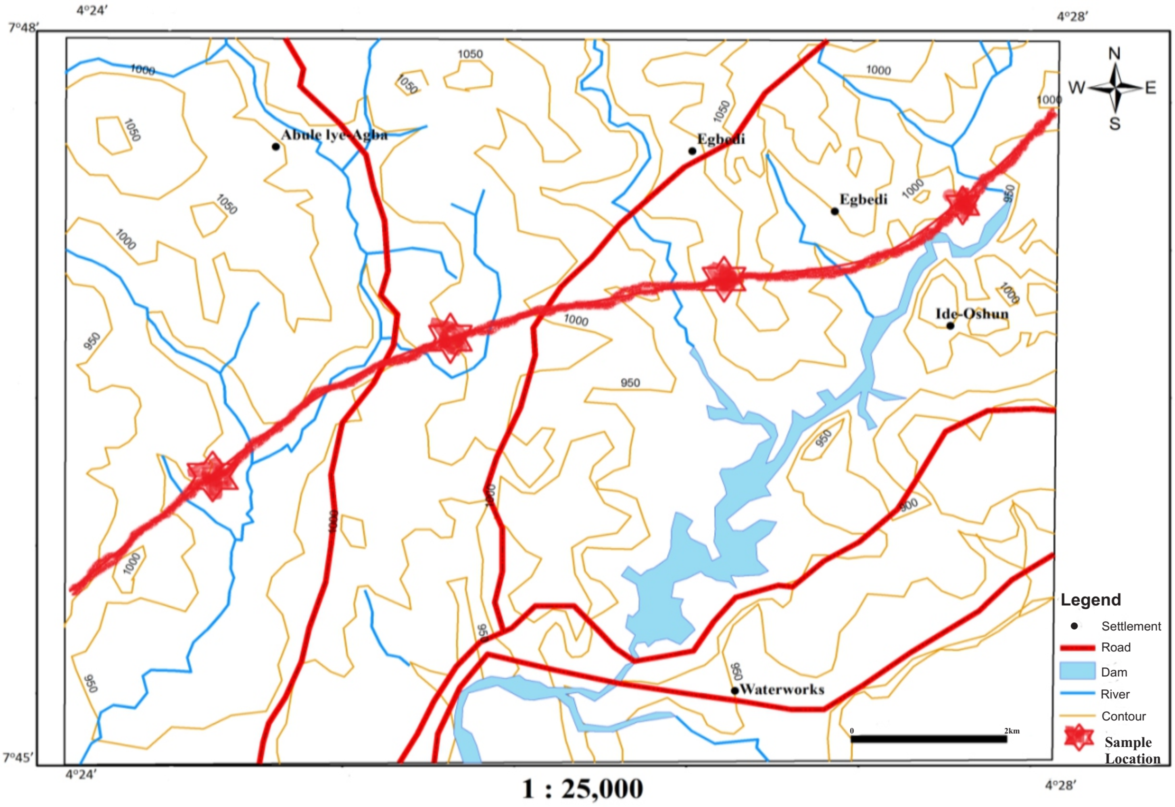

Figure 1: Topographical Map of the Study Area Showing the Investigated Highway. (Modified after Federal Surveys, Nigeria, 1967)

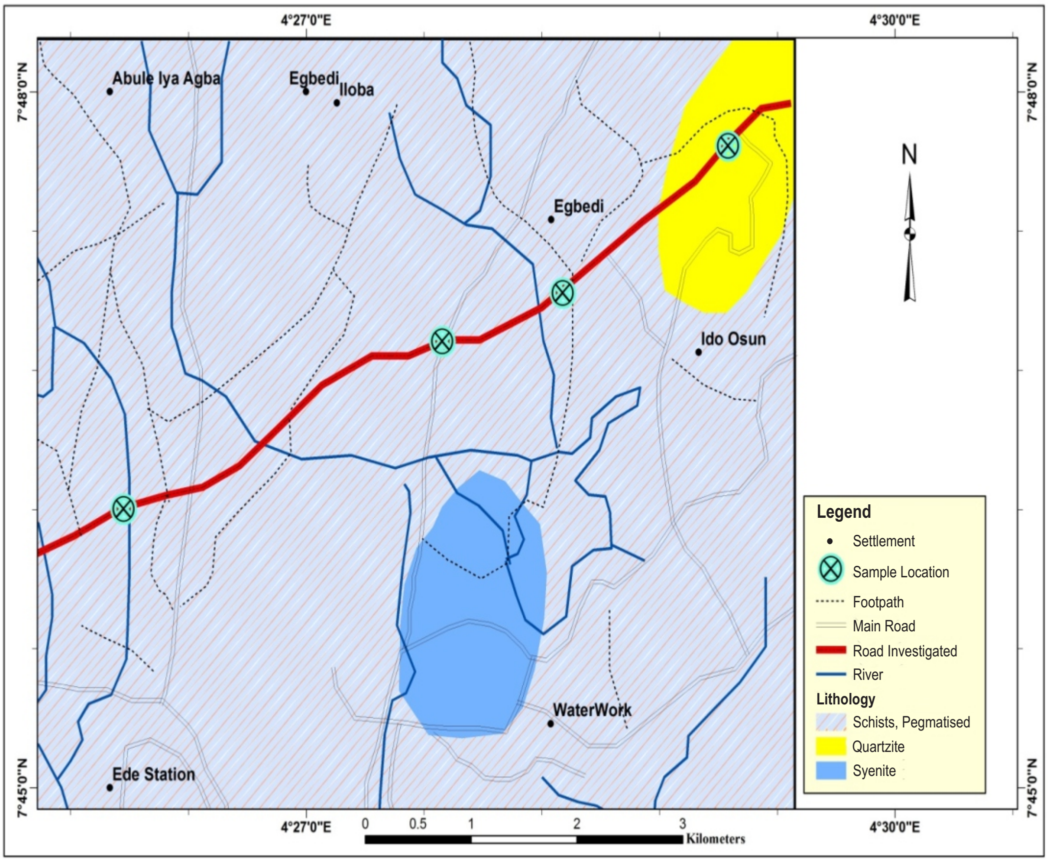

Figure 2: Geological Map of the Study Area, (Modified after the Nigerian Geological Survey, 1965)

METHOD OF STUDY

After a local geological mapping, twelve soil samples were collected from four different failed sections (See Fig. 2) earlier selected along the road. Samples were collected 5 m away from the paved road. Three soil samples were collected at each failed section based on colour change with a view to determining the characteristics of the various soil types present in the sub-grade and sub-base and even base course where possible. The maximum depth dug was 3.5 m below the asphaltic surface. The soils were suitably packed into an air tight sack and labeled L1-L4.

The samples were prepared in accordance with B. S. 1377 of 1990 with a little modification to suit the lateritic nature of the sample. Oven dried sample was poured into the sieve and the whole stack was transferred to the sieve shaker. After the stack was removed from the sieve shaker, the amounts retained on each sieve was weighed and the mass recorded. The percentage retained on each sieve was computed from the weight retained on each sieve relative to the original soil weight. The percentages passing were computed and these were plotted on a semi logarithm chart to obtain the grading curve.

The limitation of this process include

i. Some particles may break down as a result of prolonged shaking by the sieve shaker (Malomo, 1977)
ii. Not all particles are spherical in shape, the more angular the grains, the less likely for them to pass through the sieves of appropriate sizes.

Consistency limits which include the liquid limit (W^L) and plastic limit (W^P) were carried out in accordance to Nigerian Specification as stipulated in FMWH, 1997 guidelines. From this the plasticity index (PI) which is the amount of water required to change from its plastic limit to its liquid limit was estimated from the equation:

$$PI = W^L - W^P \quad (1)$$

The determination of linear shrinkage involved two palette knives, a hat glass plate, an oven, a 0.425 mm BS sieve and a linear shrinkage limit device. Soil sample weighing about 150 g was taken from the material passing the 0.425 mm BS sieves and thoroughly mixed with water using the palette knives until the mass became a smooth homogenous paste. The paste was put at the linear shrinkage device and was air dried for about 2 to 3 hours. The original length was measured and recorded and the device was then placed in the oven. It is necessary to air dry first in order to prevent the soil from cracking which may result from the rapid dehydration of the soil sample. The final length of the soil was measured after oven drying for about 24 hours. The difference in length was expressed as a percentage of the original length as expressed in equation:

$$S_L = ((L_O - L_F)\,100)/L_0 \quad (2)$$

Where S_L is linear shrinkage, L_O is initial length and L_F is the final length

The California Bearing Ratio (CBR) and Unconfined Compression Strength Test, (UCS) were carried out in accordance to the specification of the Federal Ministry of Works and Housing, 1997. Soils for the CBR were subjected to soaking according to the specification in order to replicate the natural conditions the soils undergo with the ingress of water. The soil was soaked for about 96 hours during which swelling was monitored adequately. In the case of UCS, a cylindrical mould (38 mm in diameter and 76 mm in height) cellophane, scale and an unconfined compression machine were employed for the test. Specimens were cast from samples that had been compacted. The samples were trimmed to specified dimensions of 38 mm diameter and 76 mm height for four test specimens of each compacted sample. The specimens were divided into two, one part was left uncured and the other cured for 48 hours. This was done by wrapping the sample in a polythene for sample moisture content determination.

RESULTS AND DISCUSSION

Sieve Analysis

The result of the particle size analysis contained in Table 1 indicates that the soil's clay content for most samples ranged from 37.4% to 86.4% except for samples 1B, 2B and 4B with clay content ranging from 11.2% to 33.2% (see Fig. 3 for the graph of some of the soils). The high clay content in the sample could be attributed to the weathering of mica and feldspar present in the underlying rock in the study area. According to the Federal Ministry of Works and Housing (1997) specification, the clay content for both sub-grade and sub-base and base materials must not exceed 35%. This high clay content could be responsible for instability of road pavement in the area.

Natural Moisture Content

Table 2, shows that soils in the study area have moisture content ranging from 6.80% - 29.18%. Samples with high moisture content are not suitable for road construction while those with low moisture content are most suitable for road construction. Samples 1A, 1B, 2B, and 3B are suitable soils while the rest are poor soils for road construction. This suggests a possible interaction of sub grade and sub base soils with water from the numerous fractures in the basement rock which increases the wetability of the soils. This condition is expected to greatly reduce the shear strengths of soil and therefore causes the incessant failure of the overlying pavements.

Table 1: Summary of Grain Size Analysis and Natural Moisture Content

Sample No.	%passing sieve 2mm	%passing sieve 425 µm	%passing sieve 75µm	AASHTO Classification	Bedrock Geology
1A	86.4	61.8	37.4	A-5	Pegmatised Schist
1B	80.8	49.6	20.4	A-2-7	Pegmatised Schist
2A	92.4	75.6	44.4	A-7-5	Pegmatised Schist
2B	84.0	45.6	11.2	A-2-6	Pegmatised Schist
3A	85.2	69.2	61.2	A-7-5	Pegmatised Schist
3B	97.6	89.6	78.0	A-7-5	Pegmatised Schist
3C	98.0	94.0	86.4	A-7-5	Pegmatised Schist
4A	62.0	44.4	37.6	A-7-5	Quartzite
4B	56.4	40.4	33.2	A-2-7	Quartzite
4C	76.0	69.6	63.2	A-7-5	Quartzite

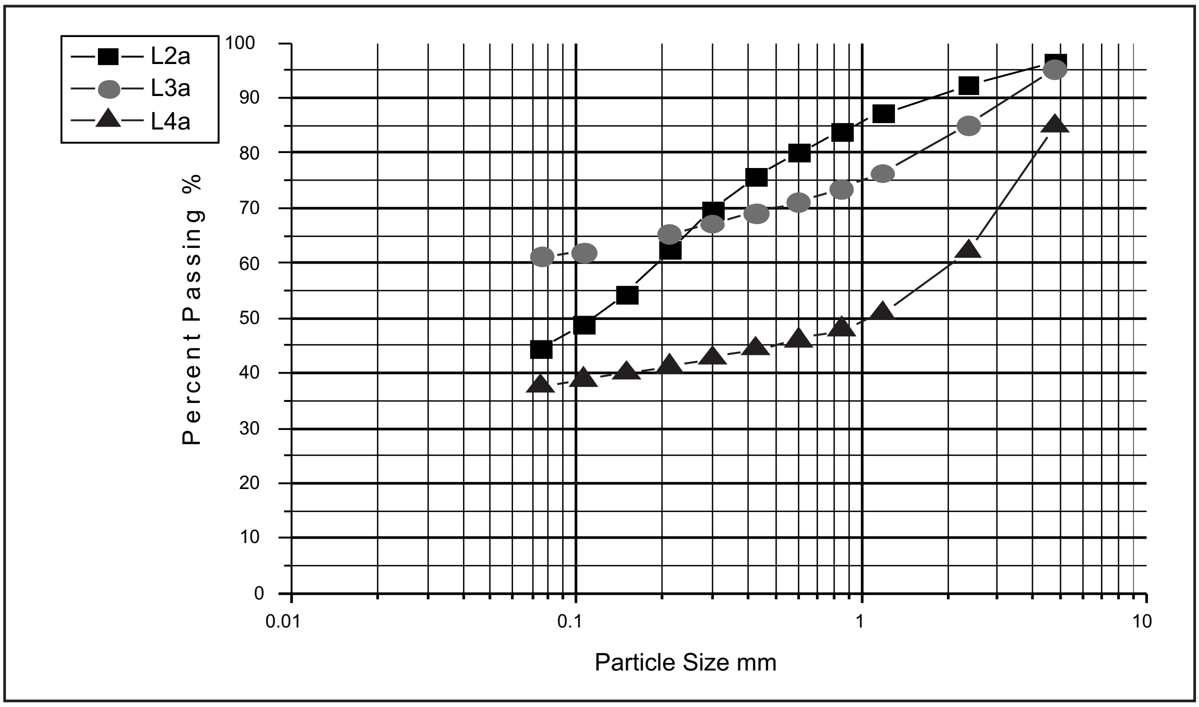

Figure 3: Sieve Analysis Graph for Selected Soil Samples from the Study Area

Atterberg Limits

Table 2 shows that the liquid limit of the soils ranges from 32.37% to 75.0% while the plastic limit ranges from 22.7% to 56.0%. Liquid limits of 40 - 60% and above are typical of clay soils while values of 25 - 50% are typical of silty soils as outlined in BS 1377 of 1990. Figure 4 shows the plasticity charts for soils in the study area. Soils with liquid limits <30% are considered to be of low plasticity and compressibility, those with liquid limits between 30% and 50% exhibit medium plasticity while those with liquid limits >50% exhibit high plasticity and compressibility. It follows therefore that, samples 1A, 1B, and 2B fall within the medium plasticity while 2A, 3A, 3B, 3C, 4A, 4B, and 4C fall within the high plasticity/compressibility of the plasticity chart. The Federal Ministry of Works and Housing (1997) specifications are liquid limits of 30% maximum, plastic limit of 30% maximum and plasticity index of 13% maximum for sub-base and base materials. None of the soil samples met these required specification. Based on high plasticity figures, the soils are expected to exhibit high swelling potentials and high shrinkage. According to FMWH these soils are not suitable for base and sub-base course materials based on liquid limit and plasticity index values <30% and <13% respectively.

Table 2 shows the result of liquidity index of the soil samples which ranged from -3.60 to -0.27. It implies that the soil samples are in semi-solid to solid states. This confirms the high degree of wetting of the soils.

From the AASHTO soil classification of soil samples for highway, A-1 and A-2 soils are excellent and good soils for highway with percentage passing Sieve No. 200 not more than 35% while A-3 to A-7 soils are fair to poor soils with percentage passing Sieve No. 200 greater than 35%. Based on this, Samples 1B, 2B, and 4B may be classified as good sub-grade soils while the rest are poor soils for both sub-grade and sub-base.

The linear shrinkage values for the soils ranged from 1.43 – 27.86. The high linear shrinkage values can be attributed to the high plasticity of the soils and this implies that the soils have the ability to swell when they absorb water. This is likely to pose a serious threat to the performance of the road, which is further complicated by the absence of drainage channels on the road. The Federal Ministry of Work and Housing (1997) recommended linear shrinkage of 8% maximum for highway soils and based on this, only samples 1A, 1B, and 2B falls within the specified range.

Table 2: Summary of Atterberg/Consistency Limit and Specific Gravity

Sample no	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Natural Moisture Content %	Liquidity Index	Specific Gravity
1A	32.37	42.0	9.63	3.57	7.33	-3.60	2.32
1B	47.0	24.13	22.87	5.71	6.80	-0.76	2.20
2A	54.0	30.47	23.53	14.29	18.37	-0.52	2.37
2B	37.90	22.71	15.19	1.43	7.49	-1.00	2.71
3A	63.0	40.9	22.1	17.86	21.06	-0.90	2.54
3B	74.0	47.7	26.3	20.71	21.45	-0.99	2.36
3C	75.0	56.0	19.0	21.43	23.31	-1.72	2.37
4A	54.15	33.23	20.92	19.29	10.28	-1.10	2.73
4B	56.00	36.97	19.03	25.00	8.27	-1.51	2.58
4C	68.50	37.37	31.13	27.86	29.16	-0.27	2.41

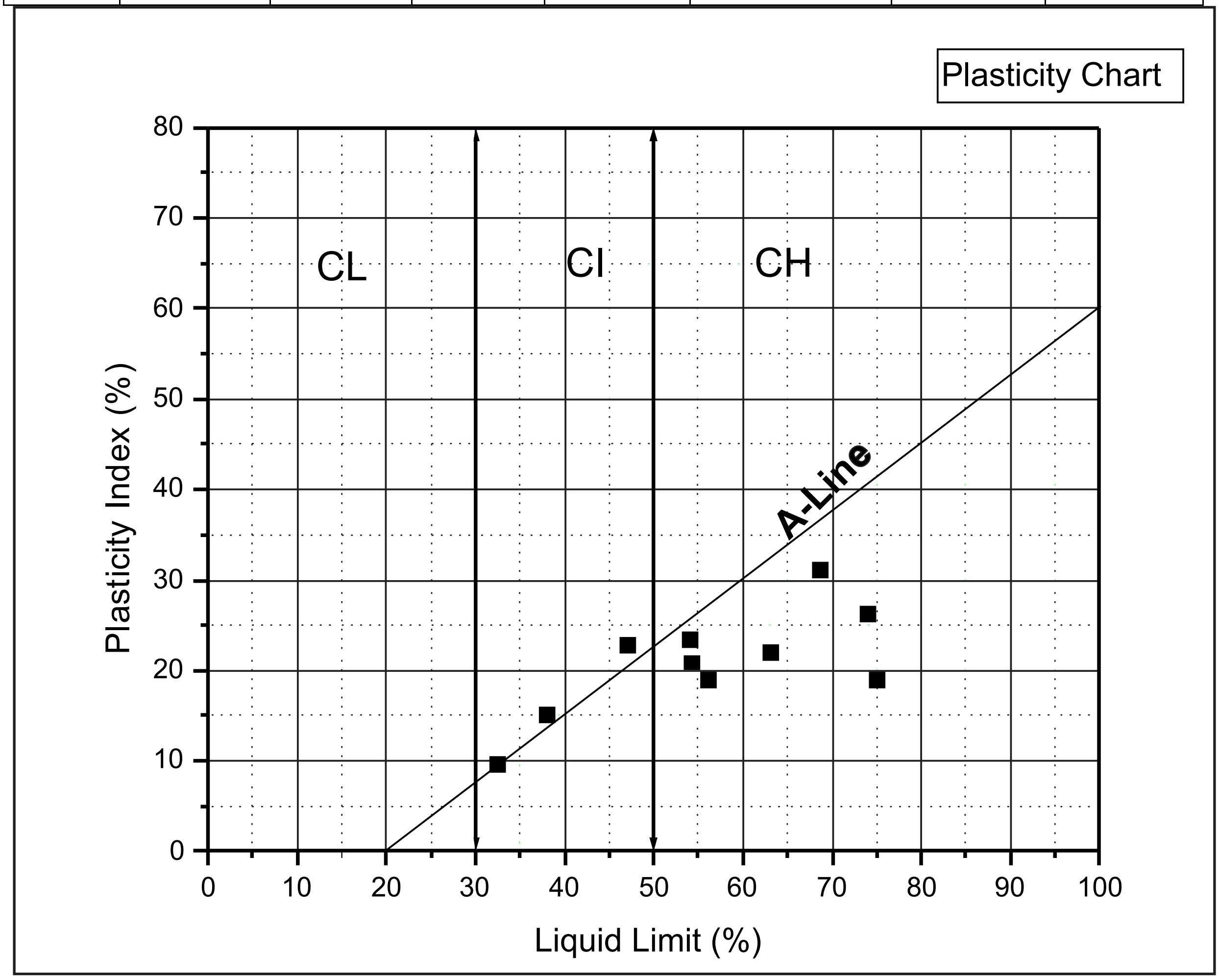

Figure 4: Plasticity Chart of Soils in the Study Area

Specific Gravity

From Table 2, the specific gravity of the soils ranged from 2.20 to 2.73. De-Graft Johnson (1969) observed that the specific gravity of lateritic soils fall within a range of 2.60-3.40, the higher the degree of laterization, the greater is the specific gravity, provided the soils are from the same parent material. However the specific gravity of clay ranges from 2.2 - 2.6. Based on this, the specific gravity of the majority of the soils falls within the clay group which shows that the soils are clayey and poorly laterized, except for samples 2b and 4a.

Compaction Test

Table 3, shows that the maximum dry density (MDD) of the soils ranged between 1.32 and 2.16 Mg/m^3, while the optimum moisture content (OMC) ranged from 13.01 to 27.98%. According to Bello *et al.*, (2007), samples characterized with high value of maximum dry density and low optimum moisture content is best suitable as sub-base and sub-grade materials. Also the Federal Ministry of Works and Housing, (1997) specified OMC less than 18% for both sub-base and sub-grade materials. Based on these specifications, samples 2A, 3C, 4A, 4B and 4C are not suitable as sub-base and sub-grade material. Sample 3B could be suitable with a minimal soil modification.

California Bearing Ratios

Table 3 shows that un-soaked CBR values ranged from 8.93% to 18.05% while soaked CBR values ranged from 6.10% to 8.21%. These values are generally less than 30% and 80% recommended for highway sub-base and sub-grade soils by the Federal Ministry of works. The soils yielded fair to poor CBR values. Such low values are not likely to provide a stable compacted sub-grade material. This deficiency could be attributed to high amount of clay present in the soil, ingress of water with a poor drainage and poor laterization of the soil used. Therefore, in terms of strength, the soils are grossly inadequate for use as a sub-grade or sub-base road construction material.

Table 3: West African Compaction Parameters and CBR at Optimum Moisture Content

Sample no	MDD (Mg/m^3)	OMC (%)	Soaked CBR (%)	Un-soaked CBR (%)
1A	1.72	17.38	12.65	12.46
1B	1.83	13.57	11.46	11.42
2A	1.58	21.61	10.00	11.42
2B	2.16	13.01	13.52	13.50
3A	1.64	16.24	6.31	8.93
3B	1.54	18.79	6.10	10.59
3C	1.32	27.98	6.16	11.01
4A	1.48	24.96	7.76	15.16
4B	1.38	26.02	8.21	18.05
4C	1.42	26.01	7.65	14.12

Conclusions

The limited but valuable tests carried out on the soil samples from failed sections in the study area showed that most of the soils in the area exhibit high natural moisture content which could be attributed to the poor drainage in the area. The soils are also clayey with percent passing sieve no 200 greater than 35%. Most of the soils possess high swelling potential which classifies them within A-5 to A-7 group of the AASHTO classification system which is indicative of fair to poor highway sub-grade and sub-base materials. The values of the MDD and OMC fall below specifications. The specific gravity result of the soils also showed that the sub-grade and sub-base soils in the study area are poorly laterized. The investigated road pavement is underlain by pegmatised schist some of which have weathered into clay this may be responsible for the poor engineering characteristics of the sub-base and sub-grade soil

The soils in the study area are wet, plastic and possess high linear shrinkage which indicates that the soils are susceptible to swelling which is evident on the waviness and potholes on the road. The study concluded that the persistent road failure could be attributed to the clayey subsoils and water ingress into the sub-grade soil due to lack of drainage channels at the shoulders of the road pavement.

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