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Geotechnical Properties of Subgrade Soils along Sections of the Ibadan–Ife Expressway, South-Western Nigeria

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Abstract
Subgrade soils beneath sections of Ibadan-Ife highway were studied with a view to identifying factor(s) responsible for the stability or failure of sections of the expressway.Bulk soil samples were collected from four stable locations on the Ibadan bound carriageway while four samples were collected from unstable locations on the Ife bound way. Specific gravity, grain size distribution, liquid limit, plastic limit, linear shrinkage, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) of the soils compacted at West African and Modified American Association of State Highway and Transportation Officials (AASHTO) levels were determined.The soils are essentially well graded with those from stable locations having between 25.42 & 56.89% amount of fines and 29.92 & 83.00% amount of fines in those from unstable locations. Variation in the degree of laterization of soil samples resulted in significant variation in the amount of fines in subgrades. Soils from the stable locations are predominantly medium plasticity soils while those in the unstable locations possess higher plasticity. Three soils from stable locations and two from unstable sections are good to fair subgrade soils while those from two unstable sections and one from stable location are poor subgrade/subbase materials based on the AASHTO classification system. The studied soils gave better compaction characteristics at modified AASHTO than at the West African level, with an Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) of 10.7-15.4 and 1850-2037KN/m^2 and 9.60 -14.0 and 1810-2076 KN/m^2, respectively for those from unstable and stable locations. Soaking of the soils compacted at both compaction levels resulted in over 60% reductions in strength measured in terms of CBR. Similarly at both levels of compaction, curing led to significant increase in the UCS, although, Modified AASHTO level generally gave higher values of UCS. Field observation showed that groundwater levels are generally higher in the vicinity of unstable locations than stable ones. The observed nature of the pavement is thus due mainly to the weakening influence of groundwater on subgrade soils.

Introduction

Over the years, significant efforts have been made in Nigeria in the development of roads and highways infrastructures, with the construction of over 20,000 km of road network nationwide. While little efforts have been made in the development of rail networks and waterways, the roads and highways have been undergoing continuous developments. However, large sections of these roads and highways fail shortly after construction and/or reconstruction. The roads are continuously reconstructed or rehabilitated without any adequate effort to identify factor(s) responsible for their continuous failure, in order to provide long lasting solution to the incessant highway failure. Apart from the huge costs expended on the reconstruction and/or rehabilitation of these roads, failed portion have continued to serve as death traps to people through fatal accidents

As noted by Gidigasu (1974), highway failure is a common phenomenon in the tropics, mostly occurring in form of rutting, pitting and waviness. The major contributing factors to highway failure in the tropics include misuse, over usage and poor construction (Ajayi, 1985). A number of studies by workers such as Adegoke-Anthony and Agada (1982), Adeniyi (1984), Anowai (1986), Teme and Anowai (1986), Olusola and Oloruntola (1998) have identified causes of road failure in Nigeria to include geotechnical factors, excessive haulage loads, poor construction, poor/inadequate drainage. However, studies by Gidigasu (1972), Mesida (1985), Ajayi, (1985), and Adeyemi (1992) have identified geotechnical factors as the cause of most road failures in the country. In many cases, subgrade and subbase materials on which the pavements are placed rarely meet the highway sub-grade/sub-base specifications. These situations are due largely to the fact that most times, road and highways are constructed without any, or sometimes with inadequate, geotechnical studies of the soil along the highways alignments and borrow pits, where materials for construction are won.

A ride along the Ibadan-Ife highway will show that the older Ibadan bound carriage way is more stable than the newer Ife bound carriage way. This study is thus aimed at evaluating the geotechnical properties of the sub-grade materials beneath some sections of the highway, with a view to determining the factors responsible for the failure of some sections of the highway. The Ibadan-Ife highway (Fig. 1) is an important continuation of the Lagos-Ibadan highway, which aids movement of goods and services from Lagos and adjoining towns and cities to Northern part of Nigeria. The highway was upgraded to a dual carriage way quite recently but as typical of most highways in Nigeria, some sections of the highway have failed, just a few years after construction. The highway was constructed with and on materials derived mostly from in-situ weathering of south-western Nigeria basement complex rocks. These rocks include migmatites, gneisses, metasediments, charnockite and dolerite dykes (Anowai, 1986).

Materials and Methods

Four failed sections (B1, B2, B3 and B4) with adjacent stable sections (A1, A2, A3 and A4) were identified (Fig. 2) and bulk soil samples were collected at the placement level of the flexible pavement, 5m from the centre of the road from each of the 8 sampling locations. The samples were collected at depth ranging from about 0.5m-1m below existing ground level, the depth adjudged to be the subgrade level in each section. The parent rock samples from which the soils were derived through in-situ weathering were also collected at the locations. Samples from the stable locations were labelled A1, A2, A3 and A4 while those from unstable locations were labelled B1, B2, B3 and B4.

The soil samples were air-dried and prepared for the different laboratory tests. These tests which were carried out following the British Standard procedure indicated in 1377 of 1975 for soil testing include classification tests (grain size distribution analysis and consistency limits), linear shrinkage, specific gravity, compaction at both West Africa and Modified AASHTO levels, Unsoaked and soaked California Bearing Ratio test and finally, uncured and cured unconfined compression test. The rock samples were thin-sectioned for petrographic studies.

Results and Discussions

Specific Gravity

Soils with specific gravity of between 2.60 and 3.40 have been classified by De Graft-Johnson (1969) as laterite soils. In addition to the use of specific gravity as the basis for identification of laterite soils, it has also been found as a useful identification and evaluation index of laterite aggregates for pavements construction, since it correlates well with mechanical strength of soil. These findings have made specific gravity an important parameter in evaluation of soils as subgrade materials, most especially in determining the degree of laterization.

The specific gravity of the studied samples is presented in Table 1. The values range from 2.60-2.72 for samples from both stable and unstable locations, the average values of the specific gravity are within the same range. Essentially, all the values are greater than 2.60 the minimum value for the specific gravity of laterite soil, invariably confirming that the pavement is founded on lateritic horizon at all locations.

Grading Characteristics

The results of the grain size distribution analyses of the soils stabilized at both West African and Modified AASHTO levels of compaction are presented in Tables 2a & 2b and the grading curves for the soils are presented in Fig 3a-3d. The grading characteristic is an important evaluation and classification parameter for highway subgrade and subbase materials because most road failures due to geotechnical factors result from the shrinking of clay as a result of withdrawal of water and expansion of clay when there is ingress of water. As a result, with other factors being constant, a soil largely made up of fines (clay and silt size particles) is likely to have worse geotechnical properties as highway subgrade or subbase than a soil largely made up of coarse particles (particles greater than 63 μm).

The soils are generally well graded with the exception of those from location B1. The soils in stable locations A1 and A2 have higher amount of coarse particles (73.39% & 59.71%) than the corresponding adjacent unstable locations B1 & B2 (17% and 48.87%) while there are fewer coarser particles at stable locations A3 and A4 (55.09% & 43.11%) than those of unstable locations B3 and B4 (61.25% & 70.08%). The significant variation in proportion of coarse to fine particles most especially in stable and unstable locations 1 (A1 & B1) and stable and unstable locations 4 (A4 & B4) which are adjacent locations, derived from weathering of the same parent rocks, can be attributed to variation in the degree of laterization or local variation in the properties of the parent rocks.

Consistency limits

These limits measure the moisture content at which soils change from one state to the other as water is added or withdrawn from a soil. It has been found useful in evaluation of sub-grade /sub-base materials. Teme and Anowaii (1987), Madedor (1983) and FMWH (1974) have specified standards limits for sub-grade/sub-base materials for highway pavements.

As shown in Table 3, the liquid limits of soil samples from the unstable locations range from 34.20-45.64% while those of the stable locations range from 23.69-48.30%. The plastic limits of the soils range from 25.09-28.92% and Nil-26.18% respectively for the unstable and stable locations respectively. The plasticity index of soils from the unstable locations range from 9.11-18.09 and 4.75-23.69 for those from the stable locations.

With the exception of location B1 (45.64%) and A2 (48.30%), the liquid limits of all the samples are within the recommended maximum liquid limit of 45% for base course material by Townsend et al (1982). Similarly, soils from only locations B1 (18.09) and A1 (23.69) have plasticity index greater than 15 recommended by Townsend et al (1982).

Soil Classification

Using the American Association of State Highway and Transportation Officials (AASHTO) classification system, samples A1 and B4 fall within the A-2-6 class for silty or clayey gravel and sands. This is a class for good sub-grade/ sub-base soils based on the AASHTO system. Soils in locations A3, A4, and B2 are fair silty soil materials in the A4 class of the classification system and materials beneath the flexible pavement at locations B1, B3 and A2 are poor sub-grade/sub-base materials based on the AASHTO system. The soils at location B3 is in the A-6 class while B1 and A2 are respectively in class A-7-6 and A-7-5.

With the exception of location B4, soils beneath the unstable locations are fair to poor as sub-grade/sub-base materials whereas only soil beneath location A2 is within the class of poor soil among all the stable locations. The Casagrande chart (Fig. 4) revealed that soils from stable locations (A1, A3 and A4 are all low plasticity soils while those from unstable locations B1, B2, B3 and B4as well as A2 have medium plasticity. In addition, all the samples from unstable location plotted below the A-line, which separates predominantly organic soils below from predominantly inorganic soils above. For the soils to be able to support stable pavements adequate drainage would be needed, in addition to chemical and/or mechanical stabilization of these soils.

Linear Shrinkage

Linear shrinkage is also an important parameter for evaluating Highway sub-grade/sub-base soils. Madedor (1983) recommended a maximum of 8% for highway sub-base and 10% for sub-grade materials. As presented in Table 3, the values of the linear shrinkage are within the range of 2.46-12.13, generally lower than the recommended value of 10%, with the exception of location A4 (12.13%). Most of the soils beneath the stable locations have lower plasticity than those from unstable sections.

With the exception of a location, all samples from stable locations have plasticity index and linear shrinkage that are better than those from unstable locations.

Compaction Characteristics

The soils were compacted at both West Africa and Modified AASHTO levels in order to determine the most suitable level of compaction for each of them. The optimum Moisture Content (OMC) and the Maximum Dry Density (MDD) obtained from the tests are presented in Table 4. The OMC at West African level of compaction, range from 11.00%-17.00% for samples from unstable locations and 10.55-16.60% for those from stable locations. The MDD at West African level range from 17.60 to 19.64 KN/m^3 and 17.80 to 19.40 KN/m^3 for samples from stable and unstable locations, respectively. The soils had better compaction parameters in stable locations A1 and A3 than the corresponding unstable locations B1 and B3. Conversely, the compaction characteristics beneath unstable locations B2 and B4 are better than their adjacent stable locations. There is thus no clear-cut correlation between the values of compaction parameters of subgrade soils and the degree of stability of pavement.

The Modified AASHTO level gave better compaction characteristics of all samples than the West African level. The range of OMC and MDD respectively for unstable locations is 10.70-15.40% and 18.50-20.37 KN/m^2 while the range for stable locations is 9.60-14.00% and18.10-20.76 KN/m^2 respectively for the OMC and MDD.

On a general note, all the soils are better compacted at modified AASHTO level because they generally had higher MDD and lower OMC, than those compacted at West African level.

California Bearing Ratio

The results of the unsoaked and soaked CBR are presented in Table 5 below. With the exception of sample A1, the CBR values of soils compacted at Modified AASHTO level of compaction are generally higher than those of the soils compacted at West African level. This trend invariably suggests that the soils are better compacted at Modified AASHTO level. The CBR are generally higher in the soils collected from unstable locations than those from stable samples. This suggests that the stability at these points may not be due to the adequate soil strength but other factors. Field observation has shown that groundwater table is likely to be higher around most of the unstable locations than stable ones. High strength of a soil may be irrelevant if such soil is in contact with water.

The Federal Ministry of Works and Housing (1974) specified minimum values of 10% and 15% respectively, for soaked and unsoaked CBR for sub-grade soils, compacted at the OMC and MDD using the BS proctor (Standard AASHTO) compaction.

Although the soils were not compacted at BS proctor, the CBR values are generally below the recommended values despite the fact that soils compacted at West African level would be expected to have values greater than the minimum of 10 and 15% respectively for soaked and unsoaked CBR. Soaking of the soils resulted in significant reduction in the CBR, with more than 90% of the samples compacted at both West African and Modified AASHTO having in excess of 60% reduction. This implies that the strengths of the soils can be significantly reduced by ingress of water into the sub-grade soils and pitting may occur. Sandy silty clays which form the bulk of subgrade soils in the study area are known to possess low permeability. Any surface water in contact with the soil will be retained for a long time, a situation that may result in weakening of the soils. This can also lead to rapid expansion of an existing particle.

Unconfined Compressive strength

The results of the uncured and cured Unconfined Compression tests are presented in Table 6 for both soils compacted at West African and Modified AASHTO levels. The range of the Unconfined Compressive strengths for soils compacted at West African level are 105.62-123.65 KN/m^2 and 298.62-391.04 KN/m^2 respectively for uncured and cured UCS of samples from unstable locations. For samples from stable locations, the uncured and cured UCS range from 50.92-135.80 KN/m^2 and 121.97-367.15 KN/m^2 respectively. The cured and uncured UCS at West African level is generally higher in all the soils samples from unstable locations compared with corresponding stable locations. Curing significantly increased the strength of the soil by as much as between 115-263.7%.

On the contrary, the uncured UCS for soil compacted at modified AASHTO level gave higher strength beneath the stable locations than the corresponding unstable locations. While the range at stable locations respectively for uncured and cured UCS is 142.7-192.4 KN/m^2 and 308.7-555.1 KN/m^2, the range for the unstable location is 114.4-149.6KN/m^2 and 320.6-572.7KN/m^2 respectively. As a result of compaction at this level, the UCS increased due to curing by 56.4% at location A1 to 316 % at location B4.

For all the soil samples compacted at modified AASHTO level , curing led to a significant increase in strength of the soils as shown in Table 6, whereas the increase in the Unconfined Compressive strength for locations A3, B3, A4 and B4 for the West African level of compaction.

As observed in the results of compaction characteristics, the soils are generally better compacted at Modified AASHTO because the strength of soils at this level are higher than those of soils compacted at West Africa level. In addition, samples B2, A3, A4 and B4 classify as excellent to good in the AASHTO classification system, all gave higher strengths than the other soils, rated as fair to poor sub-grade in the AASHTO system.

Influence of Groundwater

Field observations show that groundwater level is generally shallower on the Ife bound way than the Ibadan bound carriageway. This is responsible for the determined higher liquid limit of samples from Ife bound carriage way than those of samples from the more stable Ibadan bound carriageway.

The observed failure of the flexible pavement on the Ife bound carriageway is thus attributable to shallower depth to water table. Any soil in contact with water can have its strength reduced and/or its plasticity increased, all of which can promote failure of flexible pavement.

Conclusions

From the foregoing discussions, the following inferences can be drawn

Three soils beneath stable locations and two below unstable sections are excellent to good sub-grade/sub-base materials based on their index properties, whereas two of the samples below unstable sections and one from the stable locations are fair to poor materials. The variation in the properties at adjacent locations B3 and A3 may be due to local variation in the composition of parent rocks from which the soils at this location were developed.

The soils below unstable locations have slightly higher strengths (UCS) than those from the corresponding stable locations with the exception of the third location (A3 & B3). This probably suggests that the index properties may be a better parameter for assessing two soils with similar unconfined compressive strength.

The failure of the highway pavement at these spots is due partly to poor geotechnical properties and partly to some other factors such groundwater level and exposure to heavier axle loads.

Recommendation

Similar comparative investigations should be executed in as many parts of Nigeria as possible. In order to properly understand the hydrogeology of such environments, surface geophysical survey can be carried out. Such investigations are expected to shed more light on the influence of groundwater on the geotechnical properties of the subgrade soils.

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Table 1 . Summary of the average specific gravity of the studied soils

Sample Code	State of Pavement	Sample average
A1	Stable	2.7
A2	Stable	2.65
A3	Stable	2.66
A4	Stable	2.67
B1	Unstable	2.62
B2	Unstable	2.58
B3	Unstable	2.64
B4	Unstable	2.63

Table 2a Grain size distribution characteristics of soils compacted at Modified AASHTO

Sample Code	State of Pavement	Gravel	Coarse sand	Medium sand	Fine sand	Silt	clay	Amount of fines
A1	Stable	0.00	2.81	14.29	57.29	12.80	13.81	26.61
A2		0.00	14.49	34.97	10.25	22.10	18.19	40.29
A3		0.00	10.57	16.59	27.93	28.10	16.81	44.91
A4		0.00	0.39	5.98	36.73	30.60	26.29	56.89
B1	Unstable	0.00	2.43	5.45	9.12	13.50	69.50	83.00
B2		0.00	0.04	1.48	47.36	25.60	25.53	51.13
B3		0.00	2.53	12.27	46.44	24.70	14.05	38.75
B4		0.00	28.51	18.93	22.85	20.50	9.22	29.92

Table 2b. Grain size distribution characteristics of soils compacted at West African Level

Sample Code	State of Pavement	Gravel	Coarse sand	Medium sand	Fine sand	silt	clay	Amount of fines
A1	Stable	0.00	2.64	14.45	57.49	12.70	12.72	25.42
A2		0.00	15.07	34.69	9.46	20.80	19.98	40.78
A3		0.00	12.88	16.27	27.74	28.70	14.41	43.11
A4		0.00	0.43	5.71	36.67	30.00	27.18	57.18
B1	Unstable	0.00	2.55	5.55	8.97	14.20	68.74	82.94
B2		0.00	0.05	1.59	47.88	25.60	24.88	50.48
B3		0.00	2.85	12.50	46.01	23.10	15.54	38.64
B4		0.00	28.84	18.32	20.61	18.20	14.03	32.23

Table 3. Results of the consistency limit tests on the studied soils

Locations	State of Pavement	Liquid limit (%)	Plastic limit (%)	Plasticity Index	Casagrande chart Classification	Linear Shrinkage (%)
A1	Stable	23.69		23.69	LP	7.28
A2		48.30	26.18	22.12	MP	
A3		25.65	20.9	4.75	LP	9.32
A4		29.06	20.73	8.33	LP	12.13
B1	Unstable	45.64	27.65	18.09	MP	8.25
B2		34.20	25.09	9.11	MP	3.50
B3		41.03	26.38	14.65	MP	4.21
B4		40.92	28.92	12.00	MP	2.46

LP-Low Plasticity MP-Medium Plasticity

Table 4.:Summary of the Compaction values of the parameters of the studied soils

Sample Code	State of Pavement	West African		Modified AASHTO		% Decrease in OMC	% Increase in MDD
		OMC	MDD	OMC	MDD		
A1	Stable	16.60	1795	13.00	1850	21.69	3.06
A2		15.60	1780	14.00	1810	10.26	1.68
A3		10.55	1895	10.10	1936	4.27	2.16
A4		11.00	1940	9.60	2076	12.72	7.01
B1	Unstable	17.00	1760	12.40	1855	27.05	5.40
B2		16.60	1828	15.40	1850	7.23	1.20
B3		13.00	1830	12.00	1855	7.69	1.37
B4		11.00	1964	10.70	2037	2.73	3.72

Table 5: Summary of the California Bearing Ratio of the studied soils compacted at OMC

Sample Code	State of Pavement	West African Level			Modified AASHTO		
		Unsoaked	Soaked	% Reduction	Unsoaked	Soaked	% Reduction
A1	Stable	10.83	3.32	69.3	6.35	6.35	0
A2		10.16	5.50	45.9	17.56	6.51	62.9
A3		21.00	6.18	70.6	22.85	6.07	73.4
A4		9.85	2.26	77.1	11.68	2.75	76.4
B1	Unstable	13.36	5.50	58.8	30.35	8.37	72.4
B2		12.63	10.82	14.3	30.48	8.25	72.9
B3		21.61	6.18	71.4	24.14	7.69	68.1
B4		10.05	2.70	73.1	12.30	3.37	72.6

Table 6. Summary of the cured and Uncured Unconfined Compressive strength of compacted samples.

Sample Code	State of Pavement	West African Level			Modified AASHTO		
		Uncured	Cured	%Increase	Uncured	Cured	%Increase
A1	Stable	50.92	121.97	139.5	197.41	308.68	56.4
A2		135.8	292.34	115.3	142.71	386.01	170.5
A3		85.5	302.4	253.7	139.57	476.54	241.4
A4		103.74	367.15	253.9	164.72	555.13	237.0
B1	Unstable	123.85	298.62	141.1	149.63	320.63	114.3
B2		106.25	316.86	198.2	114.42	465.23	306.6
B3		105.62	384.12	263.7	127.63	451.39	253.7
B4		114.42	391.04	241.8	137.68	572.72	316.0

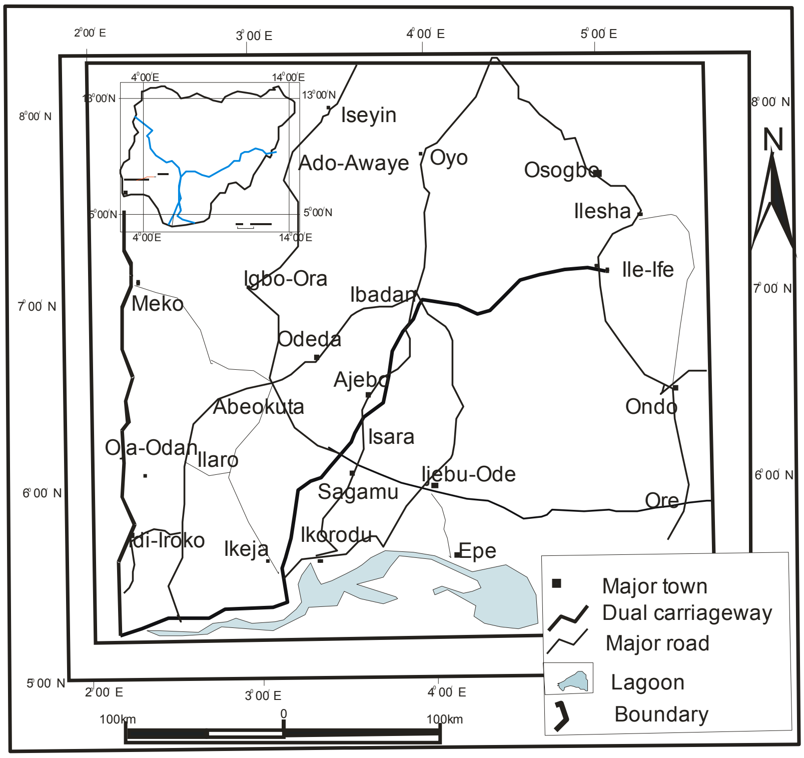

Fig. Location map of the study area

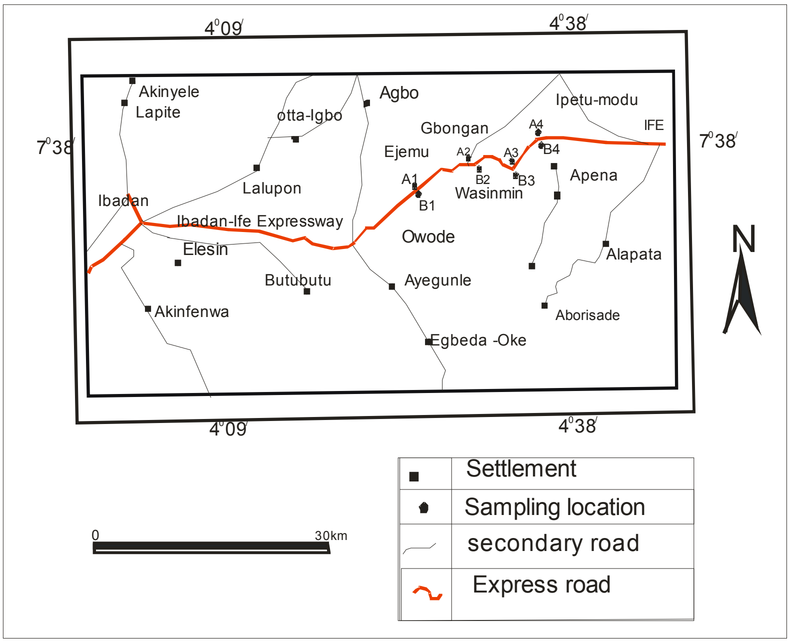

Fig.2 Sample Locations along Ibadan-Ife highway

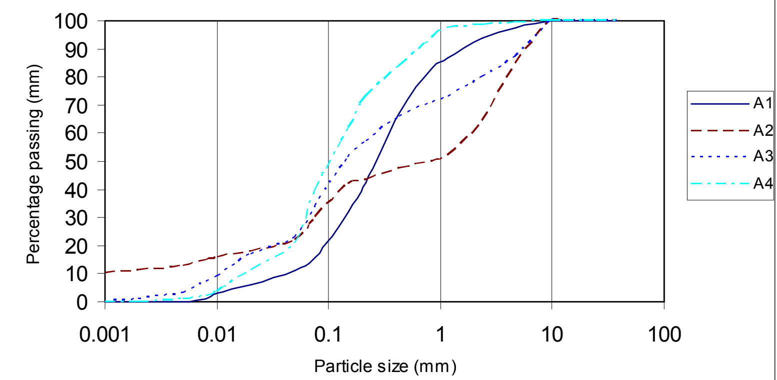

Fig3a. Grain size distribution characteristics of soils compacted at West African Level (Stable location)

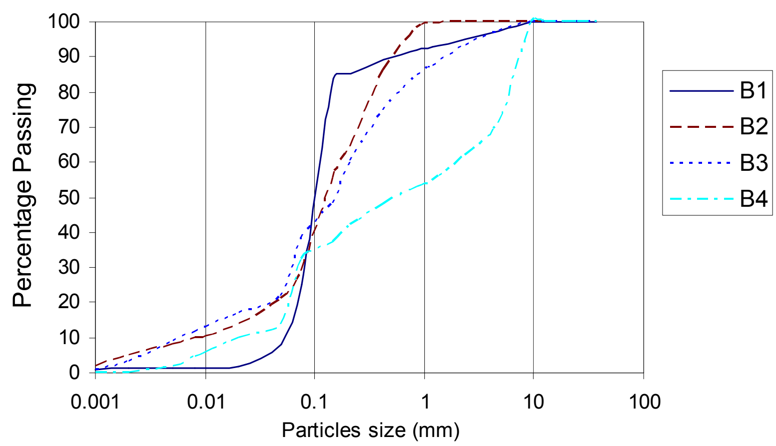

Fig.3b. Grain size distribution characteristics of soils compacted at West Africa (Unstable locations)

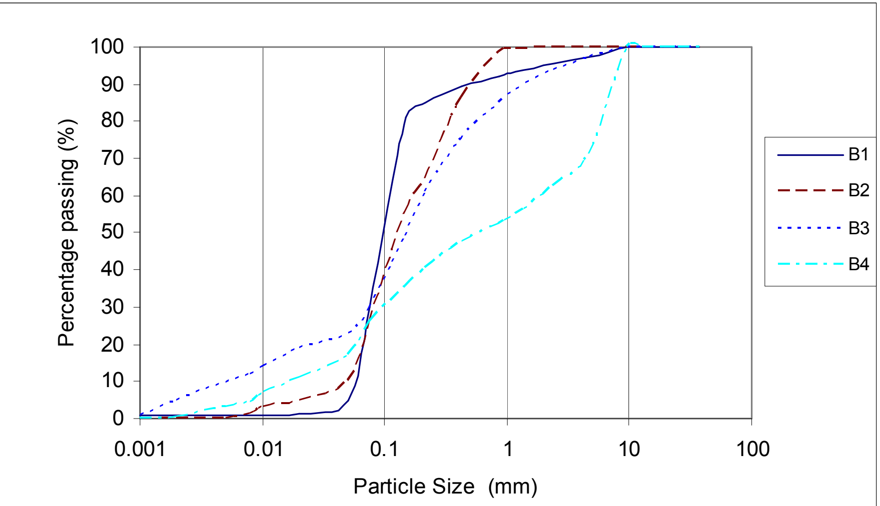

Fig.3d Grain size distribution characteristics of soils Modified AASHTO (Unstable Locations)

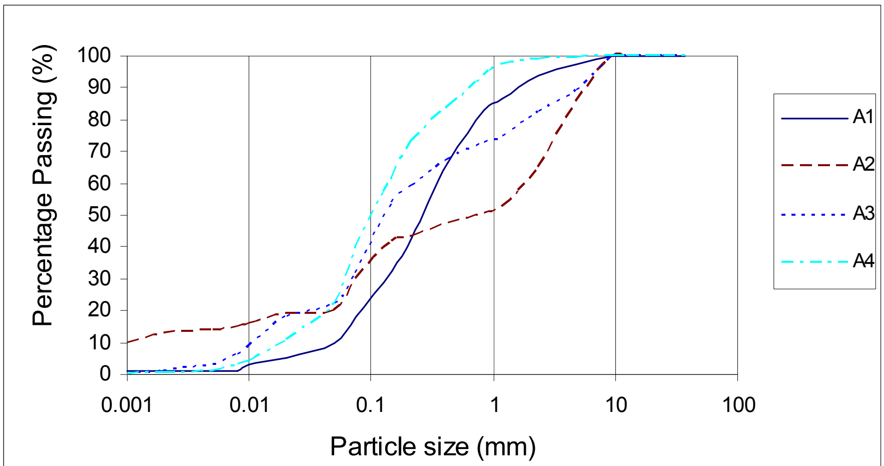

Fig 3c. Grain size distribution characteristics of soils compacted at Modified AASHTO

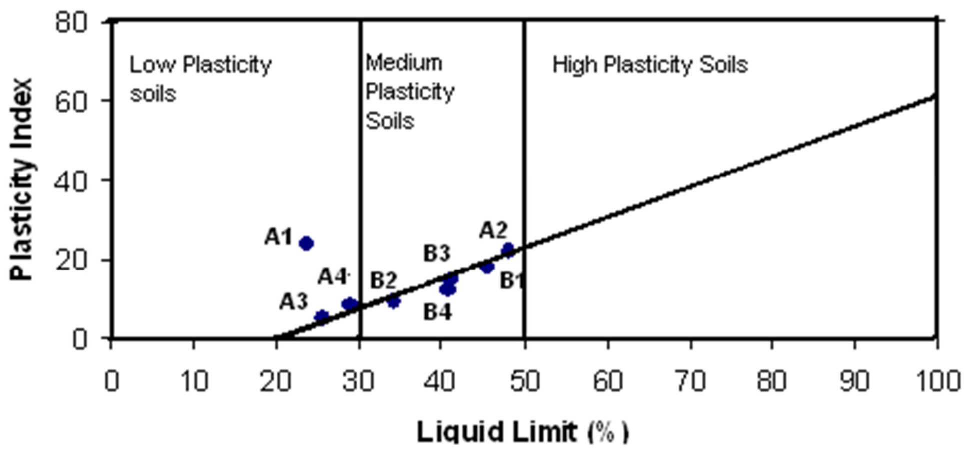

Fig. 4. Casagrande chart classification of the studied soils