PRINT ISSN 1119-8362
Electronic ISSN 2659-1502

Full-text Available Online at
https://www.ajol.info/index.php/jasem
http://www.bioline.org.br/ja

J. Appl. Sci. Environ. Manage.
Vol. 26 (12) 1969-1976 December 2022

Mineralogical and Geotechnical Properties of Subgrade Soils along Failed Portions of Abavo-Urhonigbe Road, Western Niger Delta, Nigeria

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ABSTRACT: This paper present the investigation of the possible causes of the incessant road failure associated with subgrade soils of the Niger delta using mineralogical and geotechnical properties assessment. Geotechnical investigation revealed clayey sand form the bulk of the subgrade soils composed generally of fine to medium grains with low to intermediate plasticity. Based on AASHTO the soil classifies as A-2 and A-6 with the granular soils being dominant. The average values obtained for California Bearing Ratio (CBR), maximum dry density (MDD), optimum-moisture-content (OMC) and shear strength are 19%, 1726 kg/m^3, 15.70% and 286 KPA respectively. The angle of friction ranged from 18^0 to 25^0 indicates high presence of sand. X-ray diffraction analyses reveal the absence of expandable clay mineral. Collapse potential ranged from 1.3 to 10.8% indicating slight to severe collapse. High amount of settlement and field observation of intense failure revealed the soils as collapsible, particularly when inundated under poor drainage conditions. Thus, the road failures observed in the study are as a result of the soil inherent properties and the failure of design relative to the Region peculiar geomorphological and climatic conditions.

DOI: https://dx.doi.org/10.4314/jasem.v26i12.11

Cite this paper as: UGBE, F. C; ODIBO, O. F; EMIOGE, E. A; ETOBRO, A. A. I; NWAKAJI, K. N. (2022). Mineralogical and Geotechnical Properties of Subgrade Soils Along Failed Portions of Abavo-Urhonigbe Road, Western Niger Delta, Nigeria. *J. Appl. Sci. Environ. Manage.* 26 (12) 1969-1976

Dates: Received: 18 November 2022; Revised: 17 December 2022; Accepted: 22 December 2022; Published Online: 31st December 2022

Keywords: Mineralogical; Geotechnical; subgrade; Abavo-Urhonigbe; Niger delta

Road is of great significance to the sustainable economic growth of a Nation, as it plays a vital role in socio-economic development of many nations Ademila (2017). Roads help in the transportation of people, goods and services, thereby providing employment opportunities, commercial activities which help in improving the quality of life of the general populace. The term subgrade as used in engineering refers to the in-situ content upon which is the lowest supporting layer in the pavement structure underlying the base layer. The Niger Delta where the area of study is situated is characterized by intense weathering due to tropical and subtropical climatic rainfall conditions. Diverse modes of road failures such as potholes block and longitudinal cracks, depression, rutting and ravelling have all been observed on failed roads. Investigation on road failures in Nigeria have been concentrated on the geotechnical properties of soils only without considering the mineralogy of the soils as part of the solution for pavement failures (Alo and Oni, 2018 and Odunfa et al., 2018). This paper is therefore aimed at investigating the subgrade soils along Abavo-Urhonigbe road western Niger Delta with an attempt to determine critical factors in the pavement performance by correlating the mineralogical and the geotechnical factors that are likely to have an impact on the stability of the highway and to establish the extent to which they can account for the recurrent failures of sections of the road.

MATERIALS AND METHODS

Description Of Study Area: The area of study (Fig. 1), Abavo–Urhonigbe road is located in the Western part

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of Niger Delta and falls within latitude N 05º 58' 49" to N 06 06' 22.8''and longitude E 006º 09' 57.7" to E 006º 11'33.5" of the Greenwich Meridian. The study area covers a distance of approximately 16km and characterized by alternation of stable and unstable section (Fig 2a. and Fig 2b)

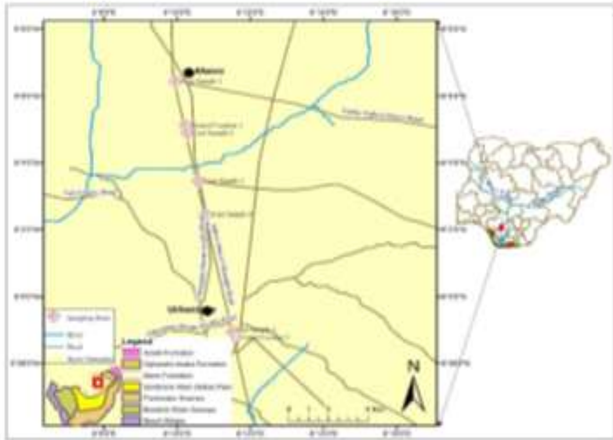

Fig 1. The map of the study Area

Fig 2a. Alligator Cracking and Edge subsidence

Fig 2b. Longitudinal Cracks

The subgrade samples used in this research work were collected using thin walled cylindrical method under percussion ASTM D2937 (2010) for undistributed samples and spade for disturbed samples from the road at depths ranging from 0.5m to 1m for each sampling point, using global positioning system GPS to accurately locate the sampling points. Seven samples of subgrade soils were collected from failed and stable sections of the road after the reconnaissance survey of the road was carried out to identify the failed sections. Undisturbed samples were place in a U-50mm tube and carefully protected with wax to avoid evaporation of natural moisture, disturbed samples taken were placed in plastic and sack bags respectively, properly labelled and sealed in a manner that each material can be distinctly identified and adulteration avoided. The samples were then transported to the laboratory for geotechnical and mineralogical analyses.

The geotechnical investigations include classification test; moisture content, Atterberg limits, particle size distribution including sedimentation by hydrometer and strength test; CBR (California Bearing Ratio), compaction, consolidation, unconfined compressive strength, unconsolidated undrained triaxial & collapse potential. These analyses were done in accordance with BS 1377 (1990) and ASTM D2166 (2016).

The mineralogical analyses using X-ray Diffraction (XRD) was done in the Nigeria geological research laboratory Kaduna to determine the mineralogy of the clay deposit. The X-ray identifies the structural layers which is dependent on the d-spacing of the clay mineral.

This spacing indicates the arrangement of the atoms in a mineral. The x-ray on passing through the clay samples give peaks that is typical to each type of minerals which make up the sample.

RESULTS AND DISCUSSIONS

Mineral Composition: The mineral composition shows that the predominant minerals found are quartz and kaolinite. Other minor minerals found are hematite and odinite. The quartz content as shown in X-ray diffractogram ranges from 65.07 to 82.88% as represented by peaks from Locations 1 and 2 (Fig. 3) and this correlates well with the values of angle of friction and particle size distribution revealing the high presence of sand. Kaolinite being the dominant clay mineral has values ranging from 10.58-24.03 % (Table 1). It is clay mineral with low shrink and swell capacity having stable physical properties. The indication of little or no swelling or expandable clay minerals has therefore been revealed by XRD test, indicating kaolinite as the major clay mineral of the soil samples. Hematite which is found in most of the studied soils is a mineral form of iron (ii) oxide (Fe_20_3) and one of the several oxides widespread in highly weathered soils which is responsible for the red and red-brown colors on many ancient and tropical soils. The less pronounced percentages of hematite/odinite indicate low particle cementation depicting the soils as relatively immature lateritic soils (Table 1)

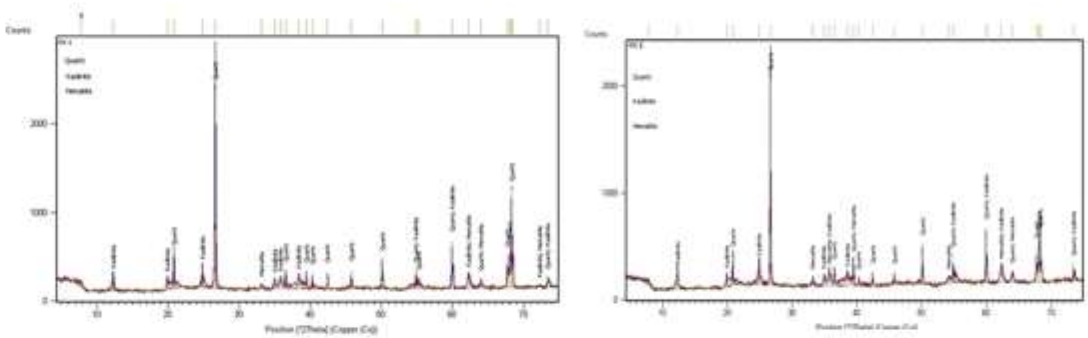

Fig 3. XRD test results depicting minerals present and their peak values for Locations 1 and 2 from the Abavo-Urhonigbe

The Particle Size Distribution (PSD); The soils from the sampled locations consist of variable proportions of gravels, sand and fine fractions as represented in (table 2), gravel fractions: (0.1% – 19 %), sand fractions ;(55 % - 79 %), silt particles; (1.6 % – 6 %) and clay fractions (16 % - 40%). The PSD curve from Fig. 4, reveal fine to medium grained soils consisting mainly of A-2-7, A-2-4, A-2-6 and A-6 type of the America Association of State Highway and Transportation Officials (AASHTO) classification system with the granular soil being dominant. The control points (CC1 and CC2), locations 2, 3, & 5 classifies as granular materials while location 1 & 4 classifies as clayey material which is consistent with the study of Ugbe (2011). The particle size distribution reveals the dominance of sand over fines which indicate a non-uniform distribution of grain sizes revealing the soil is poorly graded soil.

Table 1: Percentage composition of minerals in the soils

Minerals	percentage (%) composition of minerals in samples			
	Location 1	Location. 2	Location 3	Location 4
Quartz	76.25	65.09	82.88	76.03
Kaolinite	18.96	24.03	10.58	18.08
Hematite	4.79	9.85	-	5.89
Odinite	-	-	6.54	-
Total	100	100	100	100

Table 2: Summary of geotechnical properties of the soils

LOC.	COORDINATES		Gravel %	Sand %	Silt %	Clay %	LL %	PL %	PI %	USCS	AASHTO
	Longitude (N)	Latitude (E)									
1	06⁰06'25.4''	006⁰09'57''	7	55	3	40	40	16	24	SC	A-6
2	06⁰04'52.7''	006⁰10'29''	19	60	5	16	43	22	21	SC	A-2-7
3	06⁰03'27.2''	006⁰19'34''	1	68	5	26	29	12	17	SC	A-2-6
4	06⁰02'26.8''	006⁰10'47''	2	56	3	39	38	15	23	SC	A-6
5	05⁰58'53''	006⁰ 11'16''	5	79	6	10	24	14	10	SC-SM	A-2-4
CC1	06⁰05'06.2''	006⁰10'13''	2	65	3.5	29.5	32	15	17	SC	A-2-6
CC2	05⁰58'49.1''	006⁰11'33''	0.1	77.3	1.6	21	23	10	13	SC	A-2-6

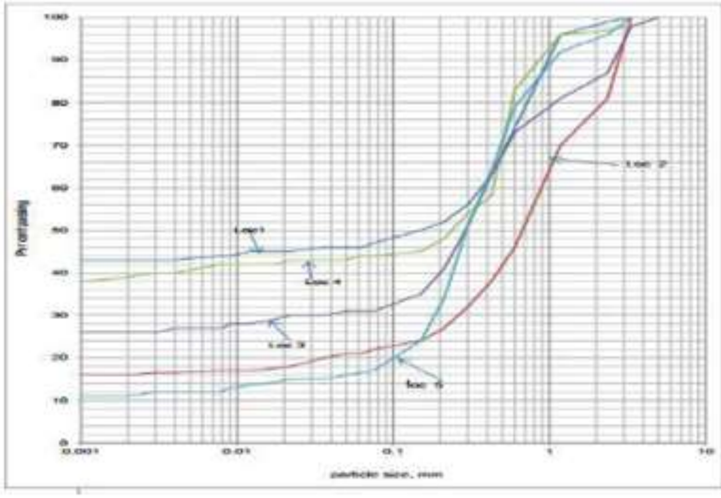

Fig 4. Typical Particle Size Distribution Curve

The Consistency Limits (Atterberg limits): The various groupings of this classification system correlate in a general way with engineering behavior of soils according to ASTM D3282 (2015) and (AASHTO) which serves as a useful guide to determine the quality of soils and classifying same for highway construction. The studied samples falls on the A-2 to A-6 class which is similar to results obtained by Ugbe (2011) that investigated the Western Niger Delta respectively. The control points (CC1, CC2), locations 2, 3 & 5 classifies as granular materials A-2 which are excellent to good subgrade material while locations 1 & 4 classifies as A-6 which are fair to poor subgrade materials (Table 4). According to the unified soil classification scheme (USCS) the soils are classified as SC-SM. Based on Casagrandes chart, a plot of plasticity index against liquid limit indicates that the samples are mainly CL and OL soils as shown in (Fig. 5).

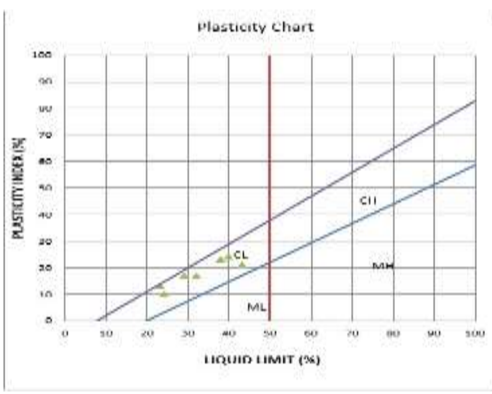

Fig 5. Position of the samples in the plasticity chart

The liquid limits of samples from locations 1, 2 and 4 are above 35% while the plasticity index is below 30%. Generally, plasticity for the soils in the failed and stable sections are expected to exhibit low to medium swelling potentials Ola (1982). The plasticity index of soils beneath the control stable segments ranges from 13-17 % while that of the failed segment ranged from 10-24 %. The soils are of low to intermediate plasticity, no value of plasticity index is greater than 50 %.*Compaction:* The maximum dry density (MDD) and optimum moisture content (OMC) has revealed values ranging from 1590kg/m^3 to 1782kg/m^3 and 14.2 to 19.1% respectively (Table 3). Location 2 had values short of the specification of FMWH (1997), having the lowest MDD at OMC and as a result of the weak engineering properties such location should be (cut to spoil) removed and replaced with a suitable subgrade material having desirable engineering properties during construction, The other locations 1, 3, 4 and 5 met the MDD of 1700 kg/m^3 at OMC 5-15 % as graphically shown in (Fig. 6), but have relatively moderate values and would require ground improvement to increase the density of the subgrade soil.

Table 3: Compaction and CBR characteristics of the sample

Location	O.M.C (%)	MDD (kg/m^3)	CBR Value (Unsoaked) (%)	CBR (Soaked) %	% Reduction in CBR
1	15.0	1,738	79	19	75.95
2	19.1	1,590	25	8	68.00
3	14.4	1,782	77	22	71.43
4	15.6	1,741	82	20	75.61
5	14.2	1779	99	26	73.74
			Average CBR		95.72%

California Bearing Ratio: The CBR values for unsoaked and soaked (24hours) ranging from 25-99 % and 8-26 % respectively with an average percentage reduction of 72.95 % (Table 3), which is similar to the result obtained in (Ugbe et al. 2021). These values can be used for simulating the site condition where there is ingress of water below the pavement. The FMWH

(1997) specification recommended limit for soaked CBR for subgrade soils compacted using standard proctor method is 10 %. Locations 1, 3, 4 & 5 met the standard specification having 19, 22, 20 & 26 % respectively but location 2 falls short of the recommended standard having 8%. This deficiency could be attributed to high dispersive properties because structurally the soil is unstable and can collapse when the soil is inundated.

Triaxial Unconsolidated Undrained Test (uu) and UCS: The unconsolidated undrained (CU) triaxial test gave cohesion ranging from 112 KPa to 442 KPa or KN/m^2. The shear stress and normal stress relationship is usually a straight line defined by an angle known as the angle of friction for granular soils. Which has revealed values ranging from 18^0 to 25^0 indicating high presence of sand? The studied samples reveal values of modulus elasticity ranging from 4.7 to 6.2 (Mpa) on an average most of the soils indicate high shear resistance. 50 % highest point of strain at which failure occur is shown on (Table 4) and (Fig. 7).

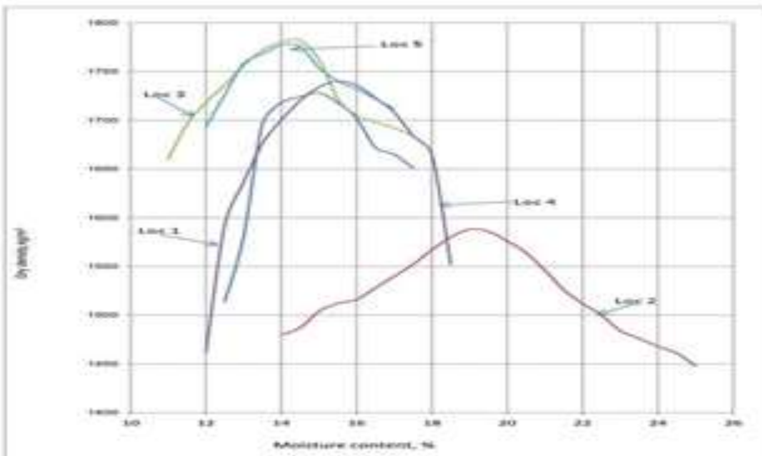

Fig 6. Typical compaction graph of the samples from Abavo-Urhonigbe road

Table 4: The shear strength and UCS parameters of the samples from Abavo-Urhonigbe road

Location	Angle of friction Internal (degree)	Cu (kpa)	E50 (%)	E50 (mpa)	qu (kpa)	ST	Strain At Failure (%)
1	20^0	287	5.1	5.6	838	1.6	6.3
2	18^0	112	2.4	4.7	230	8.3	9.5
3	21^0	303	4.9	6.2	917	5.4	10.5
4	21^0	285	5.4	5.3	966	6.2	11.1
5	25^0	442	2.4	4.7	885	1.5	12.6

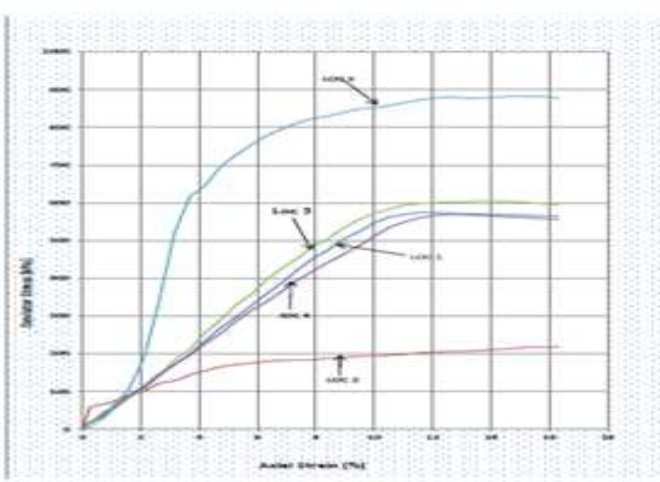

Fig 7. Typical stress-strain relationship of the samples from Abavo-Urhonigbe road

Table 5: Soil classification on sensitivity After (Mitchell and Soga, 2005)

Sensitivity S_t	Classification
0-1	Insensitive of clays
1-2	Slightly sensitive clays
2-4	Medium- sensitive clays
4-8	Very Sensitive clays
8-16	Slightly quick clays
16-32	Medium quick clays
32-64	Very quick clays
>64	Extra quick clays

Unconfined compressive strength test: It provides an approximate value of the strength of the cohesive soils in terms of total stress and also to check the sensitivity between the undisturbed and remoulded soils. The UCS (qu) values (Table 4) relating with the consistency of clays for locations 1, 3, 4 & 5 indicates high work hardening due to influence of sand which correlates with high quartz content from the results of mineralogy. Location 2 had the lowest value with qu (230) kpa. The soil classifies on the basis of sensitivity (ST) after (Mitchell and Soga, 2005) as shown in (Table 5) classifying the soils as slightly sensitive clays to slight quick clays ASTM D2166 (2016).

Consolidation: Estimation of settlement relating with the Mv= Coefficient of volume compressibility, Cv= coefficient of consolidation, Cc, compression index (Figure 8) gave an indication of large volume changes that is associated with the soils having settlements ranging from 10 cm to 103cm along the sampled points of the study area as represented in (Figure 8) which could be attributable to displacing of water from the soil due to exerting pressure from continuous applied vertical stress and duration of sustained load creating voids as opined by Head (1982). Settlements as much as 103cm from road level is bad because of the negative effect on transporters especially on smaller vehicles as experienced in location 2. Location 1, 3, 4 and 5 has estimated settlements of 58, 21, 27 & 10cm respectively (Table 6**)**.

Table 6: Settlement and collapse potential characteristics of the studied soils

LOC	e_o	Po (kpa)	Cv	Mv	Kv	Estimated Settlement (cm)	Collapse Potential Ie, (%)
1	0.75	54	$2.20E^{-11}$ m^2/s	0.589 m^2/mn	1.29^{-10} ms/s	58	6.2
2	0.85	86	$2.20E^{-11}$ m2s	0.589 m^2/mn	1.29^{-12} m/s	103	10.8
3	0.57	68	$3.59E^{-8}$ m2/s	6.77^{-6} m^2/mn	2.43^{-13} m/s	21	3.3
4	0.57	70	$6.62E^{-3}$ m2/s	1.59^{-4} m^2/mn	1.05^{-10} m/s	27	4.6
5	0.47	86	$6.41E^{-9}$ m2/s	4.20^{-6} m^2/mn	2.69^{-16} m/s	10	1.3

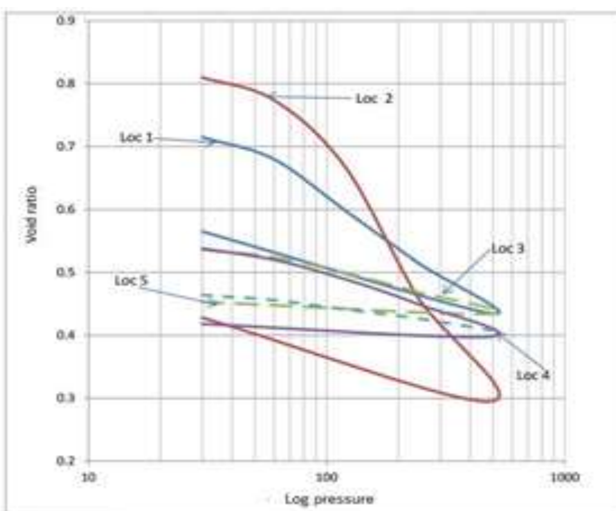

Fig 8. e log p Curve of the Samples from Abavo-Urhonigbe Road

Collapse Potential: Soil susceptible to collapse are collapsible soils, they experience largely induced settlement when saturated. Virtually no settlement as high as 10 cm should be allowable but so far there is no specification from FMWH (1997) available for settlement on roads and highways. The collapse potential values ranges from 1.3 to 10.8 % as shown in Table 6. The collapsible soil structure undergoes large deformation upon wetting (Silveira and R.A. Rodrigues, 2020). Classification based on collapse index = Ie ASTM D5333 (2010) Table 7 and Fig. 9, indicates slight to severe collapse which relate that the depressions are due to high precipitation prevalent in the Niger Delta, therefore adequate drainage at the shoulders of the road will be salient to avoid loss of subgrade strength due to ingress of water into the pavement.

Table 7: Collapse index classification LE, ASTM D5333 (2010)

Degree of collapse	Collapse index le, %
None	0
Slight	0.1 to 2.0
Moderate	2.1to 6.0
Moderately Severe	6.1 to 10.0
Severe	>10

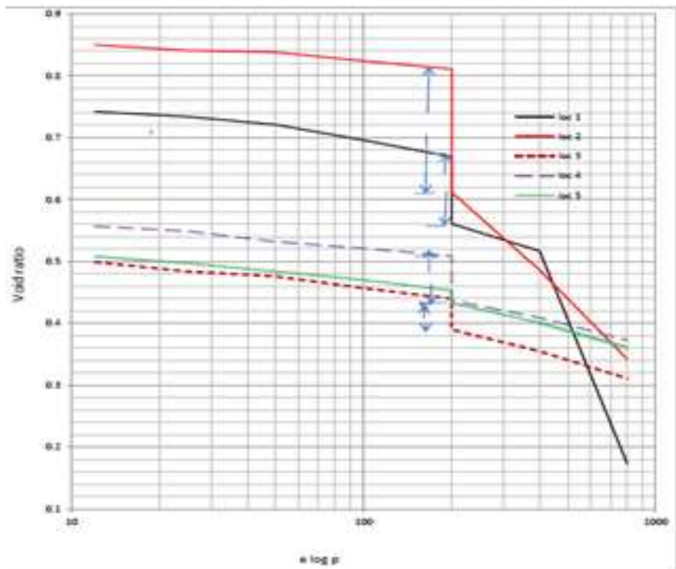

Fig 9. Collapse potential curve for Abavo-Urhonigbe

*Conclusion***:** The soils are A-2 and A-6 with the A-2 being dominant which indicates on an average that the soils are good subgrade materials. The mineralogy has identified the soil samples are predominantly quartz-kaolinite lateritic soils with absence of expandable clay minerals. The low percentage of hematite/odinite indicates minimum particle cementation depicting the soils as relatively immature. Geotechnical facts have revealed the soil is susceptible to collapse when inundated indicating they are collapsible subgrade soils due to largely induced settlements. Stabilization by chemical injection is recommended to control liquid ingress into layers to forestall liquid influence on the subgrade.

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