International Journal of Earth Sciences Knowledge and Applications (2022) 4 (2) 226-241

International Journal of Earth Sciences Knowledge and Applications
journal homepage: http://www.ijeska.com/index.php/ijeska

Research Article

e-ISSN: 2687-5993

Geoengineering Deterministic Properties of Tropical Red Soil in Sobe, Edo State, Nigeria in Relation to Civil Engineering Structure

Falowo Olumuyiwa Olusola[1]*

[1]*Department of Civil Engineering Technology, Faculty of Engineering Technology, Rufus Giwa Polytechnic, Owo, Ondo State, Nigeria*

INFORMATION

Article history
Received 29 December 2021
Revised 08 February 2022
Accepted 09 February 2022

Keywords
Geochemical
Compression modulus
Embankment
Flexible pavement
Footing

Contact
*Falowo Olumuyiwa Olusola
oluwanifemi.adeboye@yahoo.com

ABSTRACT

The long serviceability expectancy of civil engineering structures depends on the index properties and bearing capacity of the foundation soil. Therefore, the study of engineering behaviour of soil is extremely important during the design, construction and post construction phases of civil engineering projects. This study investigates the index properties of Sobe tropical red soil in relation to building foundation, embankment, and flexible pavement constructions. The study adopted geochemical analysis, in-situ cone penetration test, borehole logging, and geotechnical laboratory soil analysis using American Standard of Testing and Material. Findings revealed the USCS and AASHTO class of the soil to be CH – CL (medium graded soil) and A-7-6 respectively, signifying poor foundation material, of high compressibility and expansion, with intermediate - high plasticity. The soil is characterized by unit weight (av. 18.3 KN/m^2), soaked CBR (av. 7%), cohesion (av. 26.4 KN/m^2), angle of friction (18°), compression index (0.4489), compression modulus (2.78 N/mm^2), coefficient of volume compressibility (0.7721 m^2/KN), group index number (11.6), plasticity index (34.74 %). The material showed average bearing pressure of 111 KN/m^2 at 1 m depth for building foundation. The average (av.) values of the major mineral oxides present in the samples are Na_2O (av. 1.98 %), K_2O (av. 2.54 %), Al_2O_3 (av. 18.79 %), Fe_2O_3 (av. 19.83 %), and SiO_2 (av. 61.18 %). However, Al_2O_3, Fe_2O_3, and SiO_2 constitute 95 % of the mineral oxides. The silica - sesquioxide ratio (Se) of the soil showed that all the soil sampled soil are lateritic with a range of 1.27 – 1.96 (av. 1.59). The soil has fair to poor stability for embankment slope, thin cores, blankets and dike sections. Thus using the group index and CBR design chart for flexible pavement, the combined thickness of base and surfacing should be 30 cm and 38.1 cm for sub-base/subgrade course. In conclusion, the soil required improvement or stabilization either with chemical (lime, cement, fly ash or asphalt) or by mechanical method, which would invariably reduce its plasticity/compressibility, and increase the shear strength and bearing capacity.

1. Introduction

The stability of civil engineering structures is a function of the index properties of the subsoil which supports the structure. Index properties such as permeability, porosity, density, specific gravity, moisture content, etc., need to be studied and determined critically through thorough geotechnical investigation. These properties determine the design bearing pressure and settlement of structures, which is the basis for design and construction of civil engineering structures (Arora, 2008). These parameters will make it easier for foundation and or geotechnical engineers to recommend appropriate foundation parameters and considerations; and construction material for a particular site.

Through soil exploration, collapsible soil, highly compressible soil (peat, clay, organic soil plastic silts) can be noted and appropriate soil improvement or stabilization can be recommended (Bowles, 1984; Sherwood, 1993). The objectives of soil investigation are to determine or establish the subsurface sequence, determine the thickness of the individual stratum, determine the depth to competent bedrock, determine the groundwater condition and water table, delineate geological hazards or structures that may affect the structure, to study the causes of failure of existing works (Das, 2015). Sometimes many engineers/designers usually assume soil parameters of small size projects based on experience of the site area and at the end of such projects, they pay "painfully" for their costly assumption. Consequently, experience or familiarity with an engineering site cannot be substituted with thorough soil studies prior design and construction. Even though the extent of the investigation depends largely on the purpose of the structure, economy (budget), and variability or complexity of the soil condition at the site.

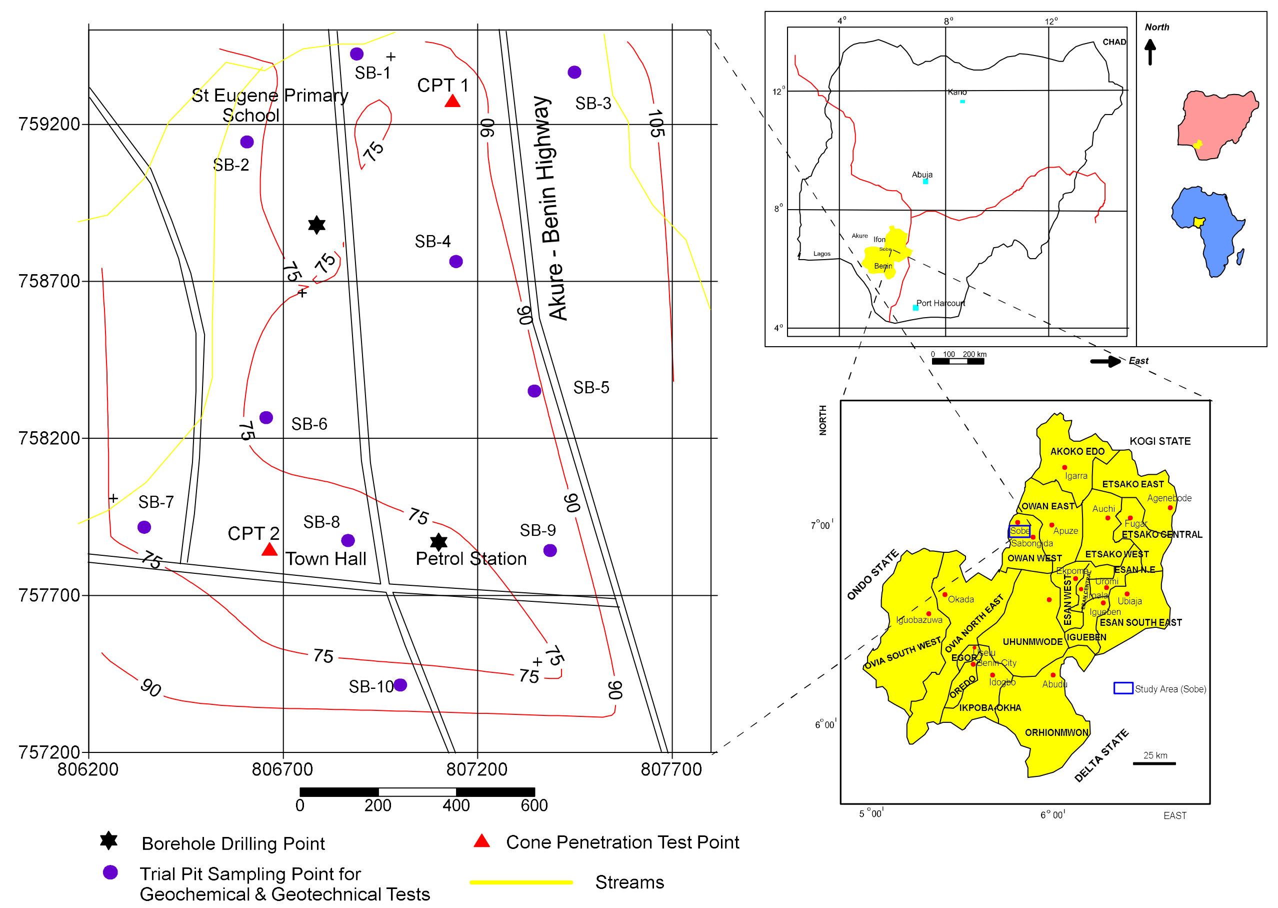

Fig. 1. Location map of the study area

A detailed soil exploration usually involves boring (deep and shallow), sampling and field test, and laboratory tests for determination of various soil properties in relation to the design of structures; aerial photographs and geophysical methods (Bello and Adegoke, 2010; Ezenwaka et al., 2014). Trial pit is applicable to all types of soils. It is a relatively cheaper way of obtaining site information through sampling, and it does not require sophisticated equipment. It involves excavation of shallow pit less than 5 m and the soil is inspected in its natural condition (Peck et al., 1974). In addition, disturbed and undisturbed samples (Venkatranmaiah, 2006) can be taken conveniently. Auger boring is also used in advancing borehole into the ground for the purpose of soil investigation (Sanglerat, 1972). They are usually suitable in cohesive in and soft soil above the water table. Hand operated augers are power driven and can penetrate up to maximum depth of 10 m.

Other methods of boring include wash boring, rotary drilling, and percussion drilling. Wash boring is a fast and simple method of boring, by the use of chopping bit fixed at the end of point of a hollow drill rod. Through the jetting and chopping action of drill rod and bits, the soil is loosened. Rotary drilling is suitable when deep-hole sampling is needed in difficult terrains or formations with boulders and fractured rock or water logged sand (Sanglerat, 1972). The method involves cutter bit or a barrel with a coring bit attached to the lower end of drill rods, is rotated by power rig. The soil or rock material loosened is then carried out by pumping water or drilling mud through the hollow drilling rod. In case of percussion drilling, the soil is loosened by the continuous impact of blows of a heavy drilling bit. The bit is also attached to the drilling rod and is raised and dropped alternatively in the borehole (Garg, 2007).

Apart from boring methods, there are other field or situ tests which have proven to be useful in determination of soil engineering properties (Falowo et al., 2017; Nwankwoala and Warmate, 2014). The commonly used methods are standard penetration test (SPT), cone penetration test (CPT), dynamic cone penetration test (DCPT), vane shear test (VST), etc (Cetin and Ozan, 2009; Douglas and Olsen, 1981; Shiva and Darga, 2016). In SPT, a split spoon sampler is made to penetrate 15 cm within the subsoil by light blows of a 65 kg hammer which drops on the top of the drill rod. The driving energy is supplied by the fall of the drop weight.

Consequently, the number of blows called N-Number required to achieve a penetration of 30 cm of a sampler is recorded (Rogers, 2006). On other hand, CPT is a direct sounding test which provides a continuous record of variation of penetration resistance (qc) with depth (Schmertmann, 1978; Nwankwoala, 2015).

The cone, usually has an apex angle of 60° and base diameter of 35.7 mm with cross sectional area of 10 cm2. The test procedure is done by pushing the cone into the soil either manually or hydraulically at rate of 2 cm/s (Nwankwoala et al., 2014). The pressure required to achieve this is referred to as cone resistance.

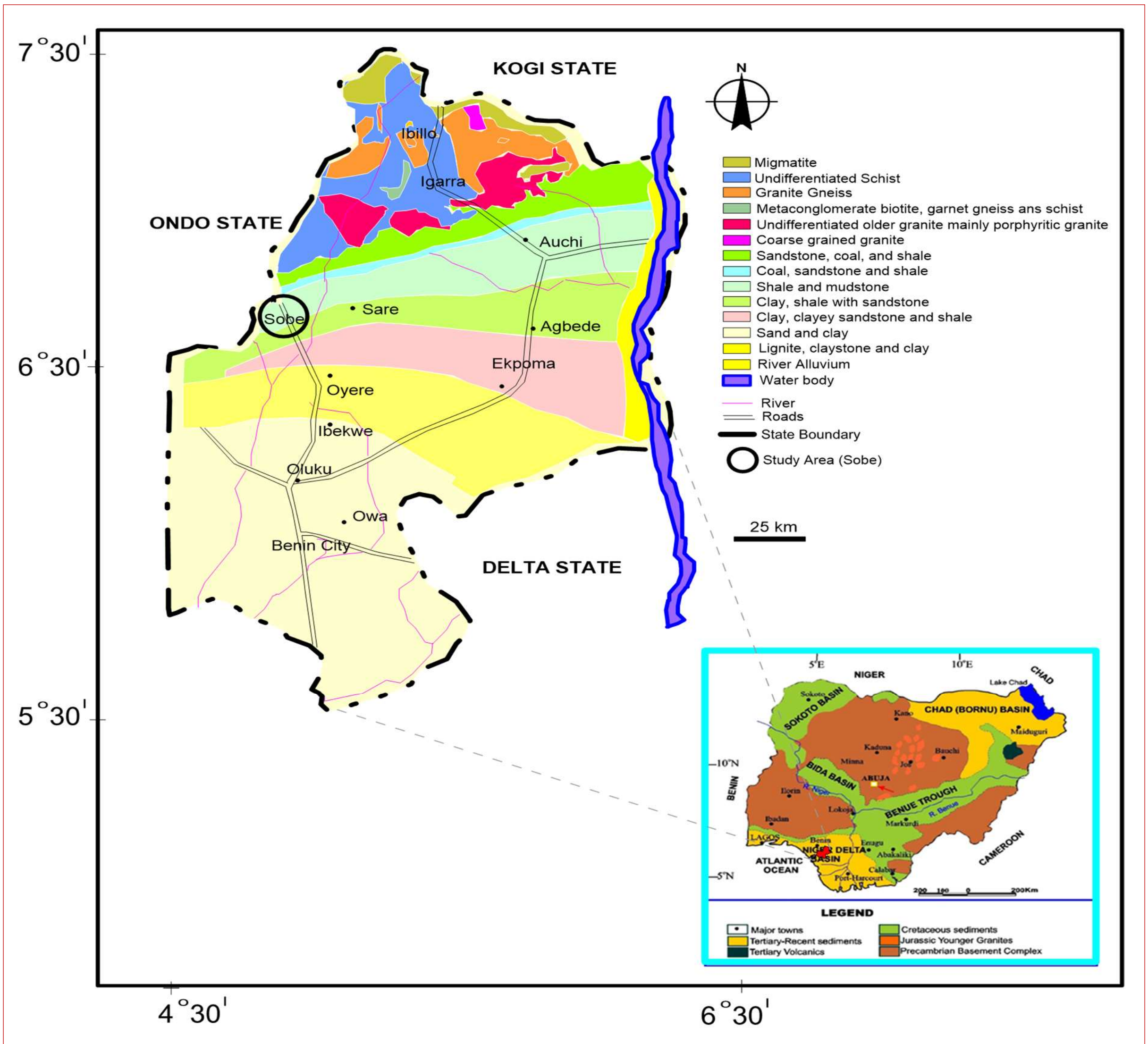

Fig. 2. Geological map of Edo State modified after Nigeria Geological Survey Agency (2006)

However, in recent time the static penetrometer had been improved to include piezocone (Baligh et al., 1981), which gives instantaneous recording of cone resistance, side friction, and the pore water pressure. The advantage of CPT over SPT is that it gives more reliable result when defining subsoil stratification and soil type. The pressuremeter is an in-situ stress-strain test conducted on the wall of a borehole using a cylindrical probe that can be inflated radially (Rogers, 2006). The pressuremeter consists of three parts: probe, the control unit and the tubing. Pressuremeter has proven to give valuable information for the design of foundations. Field vane shear test is used to obtain the undrained shear strength of soft sensitive clays. They are usually conducted in deep beds of the stratum to be tested (Das, 2015).

Geophysical methods measure changes or variation on physical characteristics of soil materials, e.g. density, electrical resistivity, magnetism, elasticity, or combination of these properties (Venkatranmaiah, 2006). Geophysical methods have been applied in engineering works such as seepage studies, foundations, abutments, large embankments, dams, etc (Brosten et al., 2005; Olayanju et al., 2017). However, the versatility of the use of geophysical methods in foundation engineering studies is limited, since quantification of subsoil characteristics condition are usually impossible. In addition to this, vital information on groundwater is generally missing. Although geophysical methods can provide missing information between widely spaced borings, but they cannot replace boring in subsurface investigation.

Commonly adaptable geophysical methods relevant in subsurface investigation are seismic refraction and electrical resistivity (Telford et al., 1990; Soupious et al., 2006). Others include self-potential, electromagnetic profiling, ground penetrating radar, and borehole logging. The seismic refraction method is based on the fact that seismic waves travel at different velocities in different geological formations (Singh, 2008; Lowrie, 2007). The method involves inducing or introduction of shock wave/energy into the ground at known distance and time, then, the refracted wave is picked up by the geophones. The results from seismic refraction often aid in determining the depth to competent rock. The validity of seismic method is based on the following assumptions: each stratum is homogenous and isotropic; the boundaries between strata are either horizontal or inclined planes; each stratum is of sufficient thickness to reflect a change in velocity on a time-distance plot; and the velocity of wave propagation for each succeeding stratum increases with depth (Milson, 2003; Sharma, 1997).

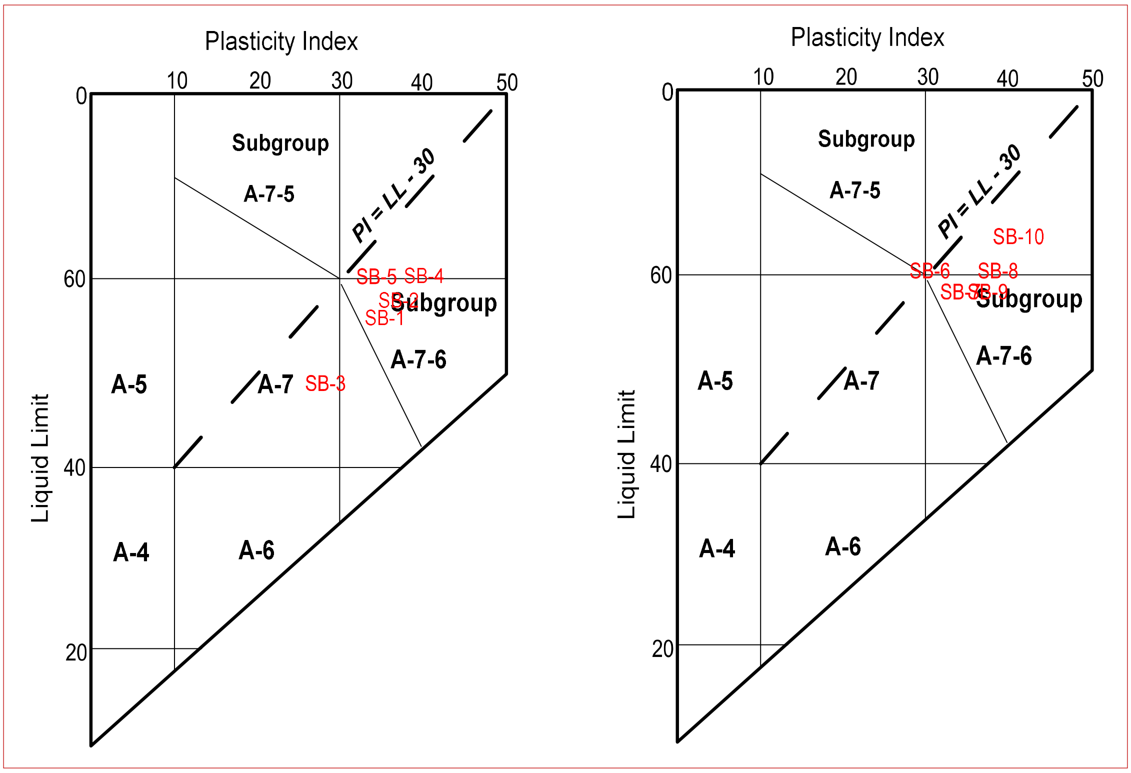

Fig. 3. Plots of the samples on A-7 classification chart

The electrical resistivity method records differences in the electrical resistance of soils and rock types (Coker, 2015). The induced current used in this method flow through the soil due to electrolytic action, which is a function of the concentration of dissolved salts, mineralogy, fluid content, porosity, degree of water saturation. Hence the resistivity of soil decreases as both water content and concentration of salts increases (Gadallah and Fisher, 2009; Singh, 2008). In electrical resistivity, the commonest technique used are the sounding (when variation of resistivity is required with depth) and profiling (when lateral variation of resistivity is required). The profiling provides 2-dimensional model interpretation in the vertical and horizontal direction along a survey line; while the soundings provide 1-dimensional model of true layer resistivity and thickness beneath the center of the electrode array (Keller and Frischknecht, 1966; Soupious et al., 2005).

Tropical red soil soils are usually the product of an in-situ (lateritic) weathering process of a basement rock, under tropical climate condition (Agbede, 1992; Gididasu and Kuma, 1987; Gidigasu, 1976). Their usefulness has been recorded in different literatures (Kamtchueng et al., 2015; Charman, 1988; Elarabi et al., 2013).

Therefore, this study determines the in-situ properties, geochemical, and geotechnical laboratory studies of the subsoil in Sobe area of Edo State, Nigeria by combining geotechnical analysis, CPT, borehole drilling with geochemical analysis with a view to recording the subsurface geo-engineering information that would be useful in safe design of civil engineering structures such as pavement, buildings, and embankment. The objectives of the study are to establish the subsurface sequence, and engineering competence of the strata. This information would help or guide those involved in design and construction of engineering structures, in the choice of foundation type and construction material to be used.

2. Materials and Methods

2.1. Description of the study area

T Sobe town is one of the towns in Owan west local government area of Edo State and located within latitude 757200-759500 mN and longitude 806200-807700 mE. It is situated approximately 5 km south of Ifon and 20 km north of Owan. The town falls within the rainforest zone of Nigeria with mean annual rainfall of about 2200 mm and mean monthly temperature in the range of 25 °C to 28 °C. The climate of the area is of the rainforest characterized by the wet season (between April-October) and the dry season (between October-March). The topography is gently undulating southward generally less than 100 m above the sea level (Fig. 1).

2.2. Geology

T The study area is underlain by the sedimentary rocks of the Dahomey basin (Fig. 2) which has been studied by many authors such as Omatsola and Adegoke (1981), Kogbe (1976), Jones and Hockey (1964), Ogbe (1972), Reyment (1965), etc., and from their investigation, the basin consists inland/coastal/offshore basin that stretches from southeastern Ghana through Togo and the Republic of Benin to southwestern Nigeria. The basin is separated from the Niger Delta by a subsurface basement high called the Okitipupa Ridge. Its offshore extent is poorly defined. Sediment deposition follows an east-west trend. According to the works of aforementioned workers, through available deep borehole cores, describes the formations based on stratigraphic units observed as Abeokuta Formation and Araromi Formation.

The Abeokuta Formation comprises mainly sand with sandstone, siltstone, silt, clay, mudstone and shale interbeds (Obaje, 2009). It usually has a basal conglomerate which may measure about 1 m in thickness and generally consists of poorly rounded quartz pebbles with a silicified and ferruginous sandstone matrix or a soft gritty white clay matrix. The Araromi Formation was defined by Omatsola and Adegoke (1981) comprises fine to medium-grained basal sand overlain by shale and siltstones with thin intercalations of marl and limestone. The shale is grey to black and has a high organic content; thin beds of lignite are frequent (Obaje, 2009). The available borehole drilling conducted in the area showed sequence similar to that of Araromi Formation. The area is drained by tributaries of Ose and Ogbese Rivers.

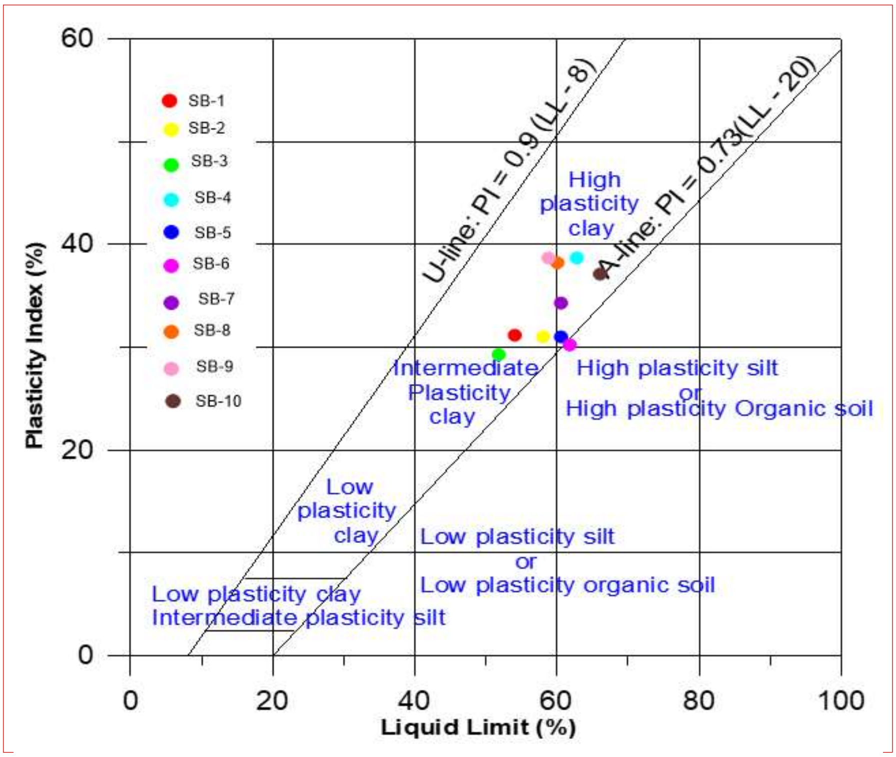

Fig. 4. Casagrande plasticity chart of the sampled soil

2. Tectonic Setup of the Region

2.3. Data Acquisition and Methodology

The nature and extent of site investigation depends on purpose of the intending structure, in terms of usage. Therefore, the study covers typical structures such as road, building. Earthwork and embankment with varying load specification. This is done so that accurate baseline information or data can be provided to aid the selection and preparation of proper foundation/excavation.

Consequently, the method adopted for this method involved reconnaissance study (RES), and detailed soil exploration. This was achieved after thorough desk study and review of literatures relating to geotechnical and foundation studies.

The reconnaissance study involved preliminary feasibility study of the area, before detailed planning was prepared. This helps in obtaining rough/sketchy information or ides of the soil type. During this study rough soil profile was established and representative soil sampling of the major soil strata was collected and also the groundwater condition of the area was assessed. It was believed that this initiative would help in deciding on future programme of exploration.

In addition, the topography, drainage, surficial soil characteristics were assessed, while the geological map and soil map were studied critically to justify/revalidate the existing maps. The detailed soil investigation engaged boring, sampling, and testing. The boring involved ten (Fig. 1) trial pits of subsurface depth of 3 m. The boring was manually excavated by the use of digger, and samples were taken at 1.5 m depth and the side wall of the hole inspected. The samples were labeled accordingly.

The samples were thereafter transported to the laboratory for the following tests, analyzed in according to ASTM (2006): visual inspection (D-2488), water content determination (D-2216), specific gravity (D-854; D-5550), particle size distribution (D-422), classification test (D-2487; D-3282), hydrometer (D-1140; D-422), Atterberg test (D-4318), California ratio (D-1883), Triaxial test (D-2850), Triaxial compression (D-2850), consolidation test (D-2435), and unit weight (D-2216).

Therefore, using the triaxial test parameters obtained, the bearing pressure of the soil at depth of 1.0 m for foundation width of 0.5 m was done using (Terzaghi) equation 1:

$$q_{ult} = cN_cS_c + qN_q + 0.5\gamma BN_yS_y \quad (1)$$

where B is footing size, c is cohesion, q is effective stress, γ is soil density, N_c, S_c, N_y, S_y are shape factors (Tables 1, 2). The in-situ field test involved cone penetrometer test on two points (Fig. 1). This method involved the determination of the resistance of soil to penetration at a slow uniform rate of a series of push of rod having a cone at its base and measuring at interval the penetration resistance of the cone and also the local friction resistance on a friction sleeve and pore pressure in the vicinity of the cone. The CPT equipment utilized the Dutch cone penetrometer with an anvil, driving rod, and other accessories.

Table 1. Angle of friction and corresponding shape factors

Degree	N_c	N_q	N_g
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	56.2	36.5	36.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
48	258.3	287.9	780.1
50	347.5	415.1	1153.2

Table 2. Foundation types and corresponding shape factors

For	S_c	S_g
Strip	1.0	1.0
Round	1.3	0.6
Square	1.3	0.8

The machine nominal capacity is 10-tonnes and is operated by using hydraulically operated driving mechanism. The cone tip angle of the penetrometer used in this study is 60° and rods are 100 cm long. In order to obtain the cone resistance value, the cone was pushed vertically at a rate of 2cm/s a depth of 0.25 m each time. Penetration resistance (*qc*), sleeve friction (*fs*) and the depth of penetration were recorded at each station and processed into plots. All the test reached refusal before the anchors pulled out of the subsurface (Ilori, 2015). The layer sequences were interpreted using the friction ratio, while cone resistance contrast between the various layers, inflection points of the penetrometer curves were interpreted as the interface between the different lithologies (Mayne, 2007; Robertson, 1990). Both qualitative and quantitative interpretation of the CPT readings in this study followed the guidelines of ASTM D 5778. The CPT data was normalized to standard overburden pressure of 100 KN/m2 (Moss et al., 2006) using equation 1 and Table 3. Hence from the result of the CPT unconfined compressive strength, ultimate bearing capacity was derived, using Table 1 and the following equations:

$$q_{cn} = C_n q_c \quad (2)$$

where q_{cn} is normalized value of q_c due to overburden and C_n is correction factor.

$$c_u = \frac{q_{cn}}{N_k} \quad (3)$$

where C_u is unconfined compressive strength, q_c is cone resistance, N_k is equal to 17 to 18 for normally consolidated clays or 20 for over consolidated clay. The bearing capacity using normalized cone resistance values was determined for D/B ≤ 1.5 for clay/silt dominated soil:

$$Strip: Q_{ult} = 2 + 0.28q_c \quad (kg/cm^2) \quad (4)$$

$$Square: Q_{ult} = 5 + 0.34q_c \ (kg/cm^2) \quad (5)$$

The chemical analysis of the ten collected samples was done to determine the mineral oxides that were present in each sample. The preparation of the sample for this analysis started by sieving the soil with 2 mm sieve and 2 g of the sieved sample was being put into digesting tube and digested using HCl, then with $HClO_4$ and H_2O_2. The samples were heated to dryness and make up with distilled water in a 100 ml volumetric flask. The resultant solution was analyzed using X-ray fluorescence and atomic absorption spectrophotometer (AAS). The silicon oxide and aluminium oxide were analyzed with nitrous oxide while Iron-oxide with oxyacetylene; P_2O_5 and TiO_2 were determined by a colorimetric method. The steps were repeated for the remaining samples, and the samples were subsequently allowed to stand for at least I hour in the solutions while they were frequently stirred (Logmo et al., 2013).

Table 3. Effective overburden pressure with corresponding correction factor

q (kN/m^2)	C_n	q (kN/m^2)	C_n
20	2.8	220	0.59
40	1.76	240	0.56
50	1.58	250	0.54
60	1.40	260	0.53
80	1.18	280	0.52
100	1.00	300	0.51
120	0.90	320	0.48
140	0.81	340	0.47
150	0.77	350	0.46
160	0.73	360	0.46
180	0.68	380	0.45
200	0.63	400	0.45

The silica: sesquioxide ratio (S_e) has served as a basis for classification of residual soils (Maigien, 1966), and this was determined for all the samples. Ratios less than 1.33 have sometimes been considered indicative of true laterites, those between 1.33 and 200 of lateritic soils, and those greater than 2.00 of non-lateritic tropically weathered soils (Martin and Doyne, 1927).

Two boreholes were drilled by rotary method in the study area (Fig. 1). Its accessories include drilling bit, drilling rods and pipes, water/mud tank, air compressor, etc. A polythene bag called sample bag was used for collection of sample for visual investigation. The samples were collected when there was contrast change in soil lithology down hole. The drilling was advanced with dual air rotary (DR) drilling method. This method features a unique lower rotary drive that is used to advance steel casing through unconsolidated overburden. The borehole was done by a rotating bit to which a downward force is applied. The bit is fastened to, and rotated by, a drill string, composed of high quality drill pipe and drill collars, with new sections or joints being added as drilling progresses with depth. Soil/rock cuttings were lifted up to the surface from the hole by the drilling fluid which was continuously circulated down the inside of the drill string through water courses or nozzles in the bit and upward in annular space between the drill pipe and borehole. The cuttings were evacuated with pumps and reverse circulation.

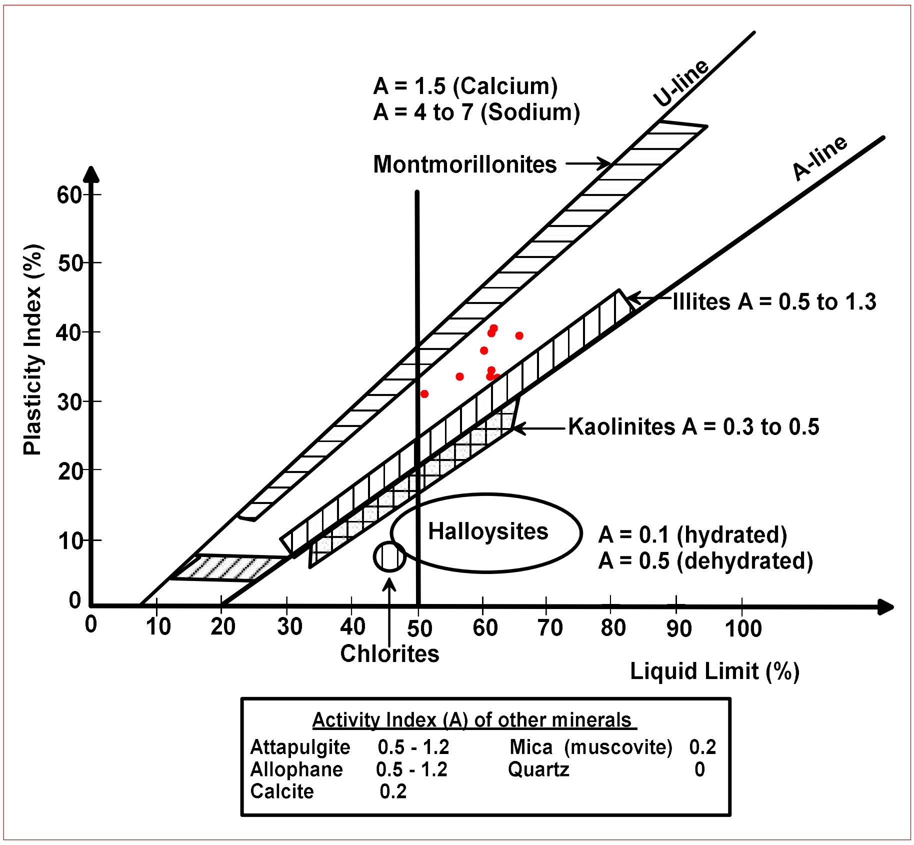

Fig. 5. Location of clay minerals on the Casangrande plasticity chart and Activity index values

3. Results and Discussion

3.1. Geotechnical studies

The results of the geotechnical tests are presented in Table 4.

3.1. 1. Natural Moisture Content

Moisture content is the ratio of the weight of the water in a soil specimen to the dry weight of the specimen (Roy and Bhalla, 2017). The NMC of the samples varied from 17.2 to 31.2% with an average of 21.9 %. The result showed that the soils have moderately high moisture in their natural state. The values obtained is within the threshold (10-40 %) of inorganic and insensitive silts and clays, which is a function of the mineralogy, formation environment, and structure of the clay. The moisture content of a soil, which can be used to determine the volumetric strain of a soil and the greater amount of water a soil contains, the less the interaction between adjacent particles and the more the soil will behave like a liquid (Oghenero et al., 2014). In other words, shear strength decreases with increasing amount of water contained in a soil (Mitchell and Soga, 2005).

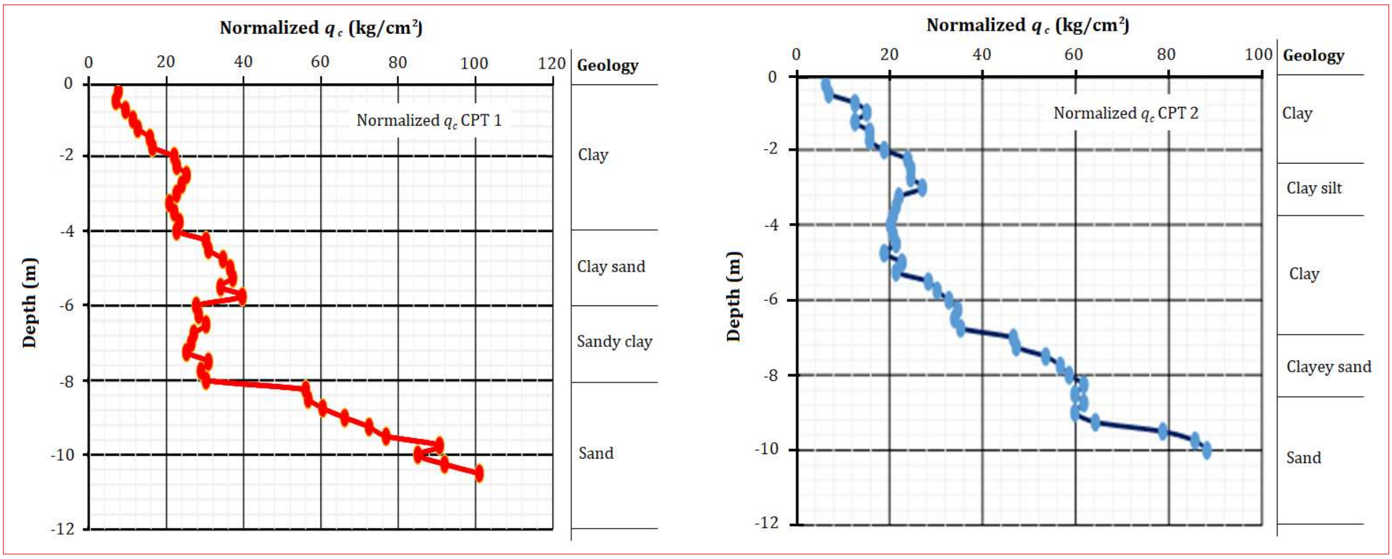

Fig. 6. The plot of cone resistance against depth for the two CPTs

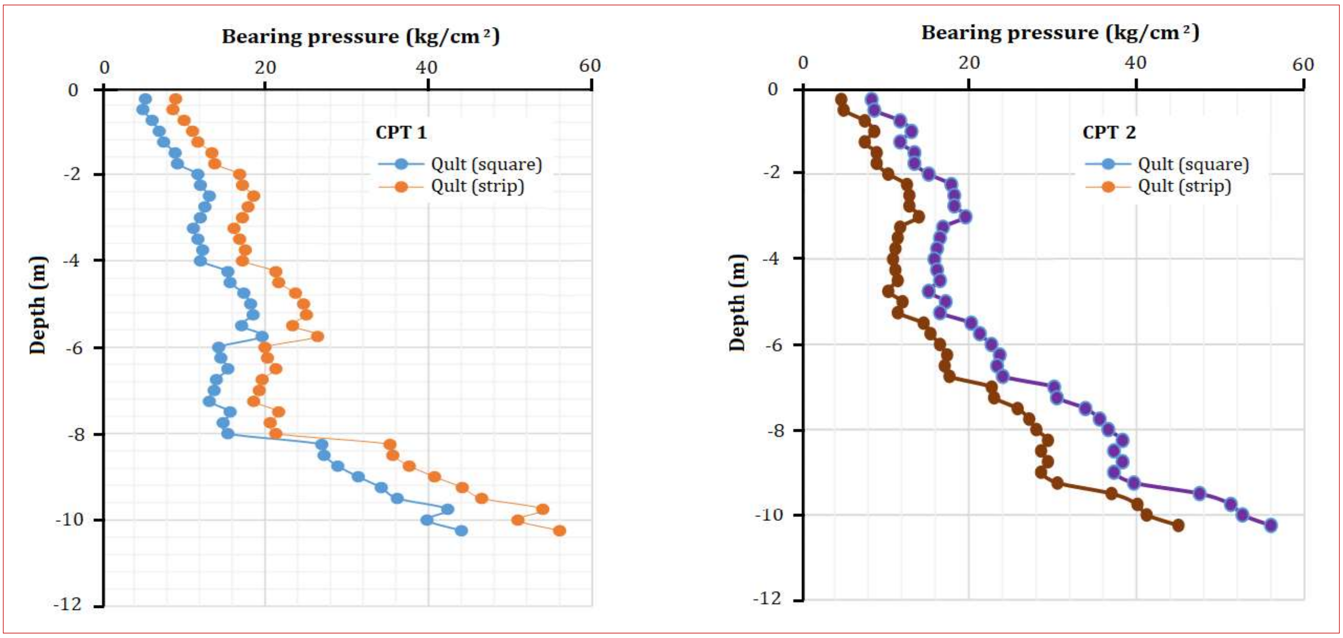

Fig. 7. Line graph of ultimate bearing pressure for square and strip footing with B/D less than 1.5

3.1.2. Grain size distribution

Particle size distributions by mechanical sieve and hydrometer analyses are useful for soil classification purposes, which gives the proportion of individual grains in the sampled soil. The results (Table 4a) showed the range and average (av.) percentage of sand (17.2-31.2 %, av. 21.90 %), silt (23.0-36.1 %, av. 30.85 %), and clay (37.6-57.1 %, av. 47.25 %). The sum of the fines in the sample ranged from 68.8 - 82.8 %, and average of 78.1 %. The % fines is greater than 35 % specified by Federal Ministry of Works and Housing (FMWH, 1997), Nigeria, hence a poor foundation for building and road (Oladeji and Raheem, 2002).

Consequently, the soil may require some degree of improvement or stabilization (Hausmann, 1990; Radhakrishnan et al., 2014). The unified soil classification system categorized the soil as CH and CL, while American Association of State Highway and Transportation Officials (AASHTO, 2006) classified the as A-7-5/A-7-6 (Fig. 3) i.e. poor foundation characteristics.

Table 4a. Summary of geotechnical analysis showing the particle size distribution, Consistency limit and soil classification

Sample no	Location		NMC (%)	Grain size distribution				SG	Consistency limits			Group Index GI/Rating	AASHTO Class	USCS Class
	Easting (m)	Northing (m)		Sand (%)	Silt (%)	Clay (%)	Fines (%)		PL (%)	LL (%)	PI (%)			
SB-1	806889	759424	19.5	19.9	36.1	44.0	80.1	2.67	25.4	57.9	32.5	9/Fair	A-7-6	CL
SB-2	807448	759366	19.3	24.1	35.8	40.1	75.9	2.66	26.2	59.4	33.2	7/Fair	A-7-6	CL
SB-3	806606	759143	18.9	31.2	31.2	37.6	68.8	2.65	20.8	50.3	29.5	5/Fair	A-7-5	CL
SB-4	807144	758763	23.4	17.2	33.2	49.6	82.8	2.65	22.7	62.1	39.4	15/Poor	A-7-6	CH
SB-5	807345	758350	20.5	19.9	23.0	57.1	80.1	2.69	29.1	60.4	31.3	15/Poor	A-7-6	CH
SB-6	806656	758266	19.5	25.7	35.4	38.9	74.3	2.65	30.5	61.1	30.6	6/Fair	A-7-6	CH
SB-7	806342	757917	29.2	19.2	25.1	55.7	80.8	2.66	25.4	60.9	35.5	16/Poor	A-7-6	CH
SB-8	806867	757875	28.6	19.8	28.1	52.1	80.2	2.67	22.2	60.8	38.6	16/Poor	A-7-6	CH
SB-9	807386	757843	25.2	20.1	32.7	47.2	79.9	2.69	20.5	59.5	39.0	13/Poor	A-7-6	CL
SB-10	807001	757415	23.4	21.9	27.9	50.2	78.1	2.68	28.6	66.4	37.8	14/Poor	A-7-6	CH

Table 4b. Summary of geotechnical analysis showing the grading curve properties, CBR, cohesion, and consolidation parameters

Sample no	D-10	D-60	Cu	Unit Weight (KN/m³)	Triaxial Test		Soaked CBR (%)	*Mv* (m²/KN)	C_c	Compression modulus (N/mm²)	*K* (cm/s)
					Cohesion (KN/m²)	Angle of friction (°)					
SB-1	0.0065	0.06	9.23	17.75	32	15	8	0.3512	0.4311	3.23	0.000275
SB-2	0.0034	0.025	7.35	18.50	30	15	7	0.4823	0.4446	1.95	0.000293
SB-3	0.0028	0.025	8.93	18.90	30	18	8	0.2885	0.3627	2.65	0.000255
SB-4	0.0083	0.06	7.23	17.99	25	20	6	0.3923	0.4689	4.45	0.00032
SB-5	0.01	0.099	9.90	18.35	27	19	6	1.3652	0.4536	2.39	0.000382
SB-6	0.0068	0.0385	5.66	18.72	25	18	5	0.3659	0.4599	3.99	0.000319
SB-7	0.0075	0.055	7.33	18.24	22	18	6	0.4587	0.4581	1.42	0.000361
SB-8	0.0021	0.015	7.14	18.81	20	17	8	0.3652	0.4572	2.79	0.00031
SB-9	0.0035	0.0236	6.74	17.88	32	22	9	1.9568	0.4455	1.56	0.00025
SB-10	0.0095	0.09	9.47	17.65	21	18	7	1.6952	0.5076	3.33	0.000318

Both systems indicate poor behavior of soil as construction material. The Group index of the soil varied from 5 to 16 (av. 12). The effective diameter (D-10) of the samples is between 0.0021-0.01 (av. 0.00604), D-60 ranged from 0.015-0.099 (av. 0.04911), coefficient of uniformity (Cu) varied from 5.66-9.9 (av. 7.898). These values suggest that D-10 and D-60 is silty, while Cu depicts medium graded soil.

3.1.3. Specific Gravity

The specific gravity of solids (Gs) is a measure of solid particle density in reference to an equivalent volume of water. The value of specific gravity ranges between 2.65-2.69 with an average value of 2.67. These values within the standard range of 2.60 and 2.80 considered for normal soil (Wright, 1986; Jegede and Olaleye, 2013).

3.1.4. Consistency Limit

The consistency limits of a fine grained soil signify the moisture content at which the physical state of the soil changes (Roy and Bhalla, 2017). The Atterberg limit tests are referred to as index tests because they serve as an indication of several physical properties of the soil, including strength, permeability, compressibility, and shrink/swell potential (Singh, 2008). These limits also provide a relative indication of the plasticity of the soil (Pedarla et al., 2011). The liquid limit (LL) values ranged between 50.3 to 66.4 % with an average of 59.88 %; plastic limits (PL) is between 20.5 to 30.5 % (av. 25.14 %) and plasticity index (PI) varied from 29.5 to 39.4 %. (av. 34.74 %).

The soils in the study area are designated CH and Cl signifying clay of low and high plasticity respectively (Fig. 6). The FMWF (FMWH, 1997) recommends LL of 35 %, PI of 12 % as maximum for subgrade, and LL of 30 % and PI of 12 % as maximum for sub-base and LL of 30 %, PI of 10 % as maximum for base course, hence the soil is weak for pavement construction. The Casagrande plot of plasticity index against liquid limit showed that all samples generally fall within the CH class, above A-line signifying inorganic clay soil (Fig. 4).

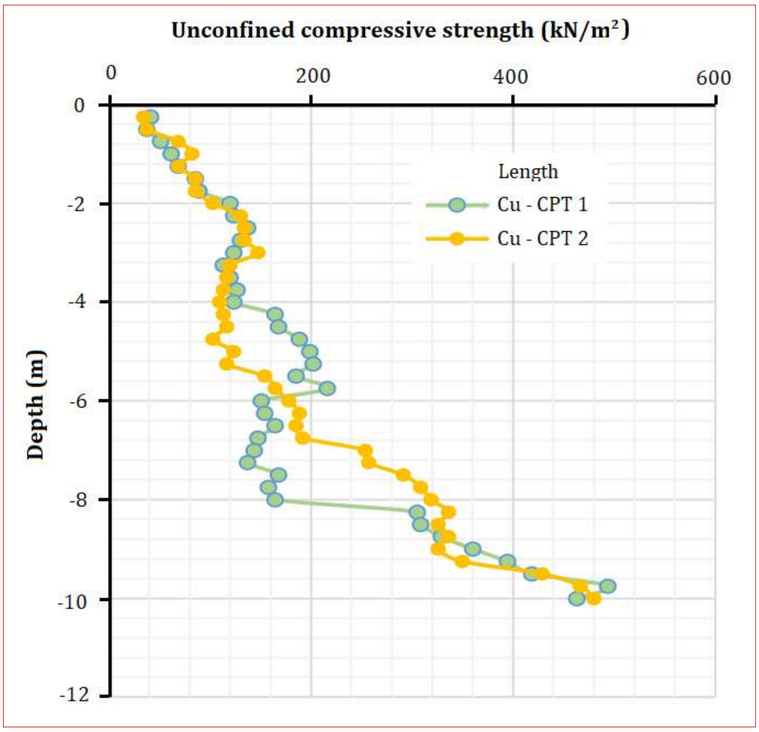

Fig. 8. Plot of unconfined compressive strength with depth

Another useful index which is based on the proportion of clay and plasticity index is known as the activity index (Pedarla et al., 2011). The activity index of a clay soil is denoted by "A" and is generally defined as the ratio of plasticity index to the clay fraction, usually taken as the percentage by weight of the soil with a particle size less than 0.002 mm. Clays with $0.75 < A < 1.25$ are classified as "normal" clays while those with $A < 0.75$ are "inactive" and $A > 1.25$ are "active." The values of activity index can be correlated to the type of clay mineral that, in turn, provides important information relative to the expected behavior of a clay soil. A clay soil that consists predominantly of the clay mineral montmorillonite behaves very differently from a clay soil composed predominantly of kaolinite. Fig. 5 shows the activities of various clay minerals and their location on the Casagrande's plasticity chart. All the soil samples plotted between montmorillonite and illites with "A" ranging from 1.3-1.5. This suggests soil of overlapping properties, from the same source, with the same degree of maturity.

3.1.5. Unit weight and triaxial tests

The dry unit weight of the samples ranged from 17.65 to 18.9 KN/m^3, and average of 18.28 KN/m^3. The soils recorded higher dry densities that accord them the potentials toward swelling, as dry densities greater than 17.60 KN/m^3 generally possess a high degree of swelling potential. The implication is that the soils will be prone to failure by settlement. The angle of friction (ϕ) range between 15-22° (av. 18°), while the cohesion (*c*) range between 20-32 kN/m^2 (av. 26 kN/m^2). According to the Unified Soil Classification System (USCS) soil having angle internal friction less than 20° are classified as soft, between 20 - 35° are classified as hard and above or greater than 35° are classified as stiff. Therefore, the soil can be classified as soft soil with moderately low cohesion. The compression modulus ranged between 1.42-4.45 N/mm^2 (av. 2.78 N/mm^2).

3.1.6. California Bearing Ratio

The soaked CBR values obtained for samples ranged from 5 to 9 % with an average of 7 %. The Federal Ministry of Works and Housing recommends 10 % minimum for subgrade, 30 % for sub-base and 80 % for base course soil respectively. Using Table 5 the soil is fairly good as subgrade material with expected very high volume change.

However, they did not satisfy requirement for base, sub base material for road construction. The high volume change can be minimizing by making the thickness of non-reactive overlying layer greater than 2 m in fills or greater than 1 m in cuts.

Alternatively, the soil be stabilized with additives such as cement, fly ash or lime for subgrade (Bell, 1996; Bose, 2012; Sherwood, 1993).

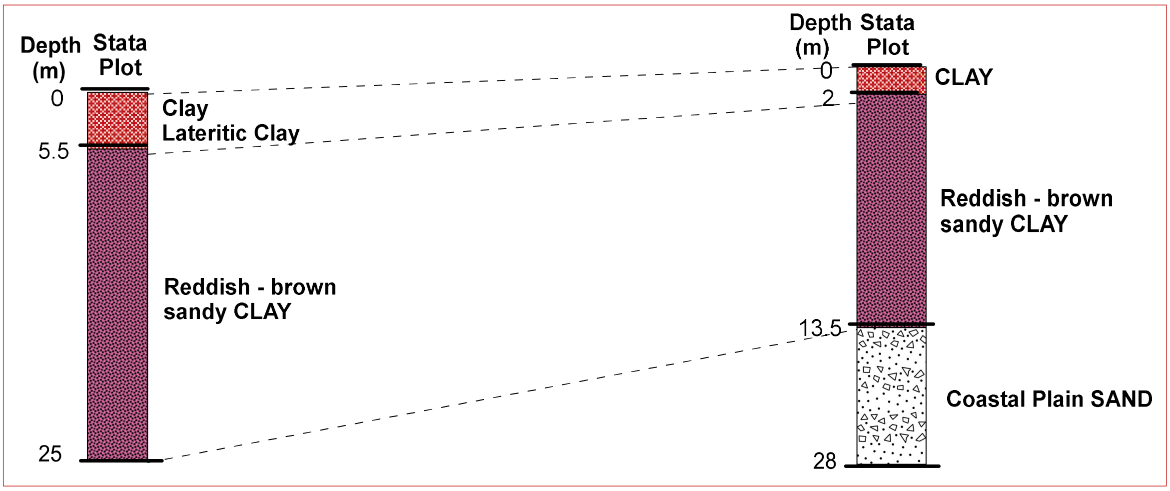

Fig. 9. Borelog sections

3.1.7. Permeability and Consolidation

The compression index varied from 0.3627 to 0.5076 (av. 0.4489) which denotes a plastic clay (0.15-1.0). The coefficient of volume change (compressibility) is the change in volume of a soil per unit of initial volume per unit increase in the pressure. The coefficient of volume compressibility (M_v) is between 0.2885 to 1.9568 m^2/kN (av. 0.7721 m^2/kN). The interpretation of *mv* in Table 6 confirmed the high plasticity of soil material in the study area. The compression modulus ranged from 1.42-4.45 N/mm^2 with an average of 2.78 N/mm^2 which is very high. Compression modulus has linear relationship with compressibility/ plasticity, the higher the modulus the higher the compressibility. The coefficient of permeability of the soils ranged from 0.00025 to 0.000382 cm/s.

Table 5. Subgrade strength classification

Soaked CBR	Strength Classification	Soaked CBR	Subgrade volume change classification
<1%	Extremely weak	<1%	Very low
1% - 2%	Very weak	1% - 2%	Low
2% - 3%	Weak	2% - 3%	Moderate
3% - 10%	Medium	3% - 5%	High
10% - 30%	Strong	>5%	Very High
>30%	Extremely strong		

Table 6. Typical values of M_v for Clayey material

Soil	M_v (m^2/KN)
Hard clay	0.125 – 0.625
Stiff clay	0.25 – 0.125
Plastic clay	2.0 – 0.25
Peat	10.0 – 2.0

Table 7. Ultimate Bearing Pressure Calculated for square footing at emplacement depth of 1 m

Sample no	Allowable Bearing Pressure (KN/m^2) For: B = 0.5m; D = 1m; L= 0.5m
SB-1	104
SB-2	99
SB-3	122
SB-4	121
SB-5	120
SB-6	105
SB-7	95
SB-8	82
SB-9	172
SB-10	91

3.1.8. Bearing Capacity

The ultimate bearing pressure for the sampled soil is presented in Table 7, with an average of 111 KN/m^2. All the soils are generally characterized by low bearing pressure (less than 200 KN/m^2). The bearing pressure obtained is for square foundation of width of 0.5 m, breadth of 0.5 m, and foundation depth of 1 m.

Table 8. General relationship of consistency and C_u of clayey soil

Consistency	C_u (m^2/KN)
Very soft	0 – 25
Soft	25 – 50
Medium	50 – 100
Very stiff	100 – 200
Hard	> 400

3.1.9. Cone Penetration Test

The plotted cone penetration sounding curves for the two locations is shown in Fig. 6. The curves generally showed similar responses. They are essentially characterized by clayey material in the upper 10 m and 6 m at location 1 and 2 respectively with sand serving as base material for CPT 2. From the CPT 1 signatures, the soil layering consists of clay (10-25 kg/cm^2) at depth 0-4 m, clay sand (25-40 kg/cm^2) at depth 4-6 m, sandy clay (25-30 kg/cm^2) at depth 6-10 m, and sand (40-105 kg/cm^2) at 10-12 m. CPT 2 is made of the following sequence, clay (5 - 20 kg/cm^2), clay silt (20-30 kg/cm^2), clay (30-40 kg/cm^2), clay sand (40 - 60 kg/cm^2), and sand (60-90 kg/cm^2). They are delineated within depth range of 0-2 m, 2-3 m, 3-5.5 m, 5.5-7 m, and 7-10 m respectively.

The ultimate bearing pressure for building foundation with depth-width ratio of 1.5 for strip and square footing gives the range of 5-44 kg/cm^2 (av. 18 kg/cm^2) and 9-52 kg/cm^2 (av. 24 kg/cm^2) respectively for CPT 1; and 5-45 kg/cm^2 (av. 19 kg/cm^2) and 8-56 kg/cm^2 (av. 26 kg/cm^2) respectively for CPT 2 (Fig. 7). The bearing pressure of 40 kg/cm^2 for lightweight structure was obtained at depth of 4.8 m and 6.8 m for CPT 1 and 2, respectively. Subsequently caution must be taken during the design of footing especially for heavy structures, adequate factor of safety must be adopted to ensure that settlement is reduced in relation to the bearing pressure. The unconfined compressive strength for the CPTs (Fig. 8) was interpreted using Table 8; for CPT 1, the soil varied from soft (1-2 m), medium (2-4 m), stiff (4-6 m), medium (6-8 m) and very stiff (8-10 m); and soft (1-2 m), medium (2-4.7 m), stiff (4.7-6 m) and very stiff (6-10 m) for CPT 2. Thus depth 2-6 m would be suitable for emplacement of building foundation, however with appropriate stabilization.

Table 9. Results of geochemical analysis

Element (%)	SB-1	SB-2	SB-3	SB-4	SB-5	SB-6	SB-7	SB-8	SB-9	SB-10
MgO	0.30	0.32	0.38	0.65	0.19	0.42	0.30	0.31	0.51	0.44
Al_2O_3	16.82	17.40	24.50	18.38	18.60	17.95	19.20	16.11	20.40	18.53
SiO_2	61.50	59.20	58.90	64.20	63.80	54.10	59.10	68.55	60.26	62.230
P_2O_5	0.00	0.01	0.00	0.01	0.10	0.01	0.01	0.01	0.00	0.00
Na_2O	0.16	4.02	3.08	0.85	0.90	2.44	3.85	1.11	2.40	0.99
K_2O	3.44	5.42	1.30	1.45	2.65	2.75	1.50	1.22	3.60	2.11
CaO	0.11	0.12	0.22	0.63	0.20	0.07	0.26	0.09	0.08	1.20
TiO_2	1.40	1.32	1.08	1.55	1.68	1.02	1.22	1.90	0.86	0.55
V_2O_5	0.05	0.12	0.08	0.04	0.06	0.01	0.01	0.25	0.35	0.09
Cr_2O_3	0.01	0.01	0.03	0.02	0.03	0.01	0.02	0.03	0.00	0.01
MnO	0.03	0.03	0.05	0.01	0.15	0.15	0.15	0.15	0.13	0.03
Fe_2O_3	18.55	18.20	16.08	17.68	22.78	24.55	19.19	18.82	22.11	20.32
CuO	0.03	0.03	0.03	0.01	0.30	0.01	0.00	0.10	0.11	0.00
Sesquioxide Ratio	1.74	1.66	1.45	1.78	1.54	1.27	1.54	1.96	1.42	1.60
Class	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic	Lateritic

3.2. Geochemical results

The results of chemical analysis showing the different element oxides in the soil samples, and silica: sesquioxide ratio (Se) is presented Table 9. The range and average (av.) of the mineral oxides present in the samples in ascending order are Na_2O (0.16-4.02 %; av. 1.98 %), K_2O (1.22-5.42 %; av. 2.54 %), Al_2O_3 (16.11-24.50 %; av. 18.79 %), Fe_2O_3 (16.08 - 24.55 %; av. 19.83 %), and SiO_2 (54.10-68.55 %; av. 61.18 %).

However, Al_2O_3, Fe_2O_3, and SiO_2 constitute about 95 % of the mineral oxides, reflecting the higher degree of oxidation; while P_2O_5 (0-0.10 %; av. 0.015 %), Cr_2O_3 (0 - 0.03 %; av. 0.017 %), MnO (0.01-0.15 %; av. 0.088 %), V_2O_5 (0.01-0.35 %; av. 0.106 %), CaO (0.07-1.2 %; av. 0.298 %), TiO_2 (0.55-1.90 %; av. 1.26 %), and MgO (0.19-0.65 %; av. 0.38 %) constitute 5% of the mineral elements. Silica - Sesquioxide ratio (Se) of the soil shows that all the soil sampled soil are lateritic with a range of 1.27-1.96 (av. 1.59).

3.3. Borelog log

Four major geological units are delineated on the borehole log section, which comprised clay, sandy clay, and sand. BH-1 reveals clay topsoil within upper 1 m depth, this grades into lateritic clay to depth of 5.5 m, underlain this stratum is reddish brown sandy clay which extended from 5.5 m to 25 m. BH-2 consists of clay (upper 2 m), reddish brown sandy clay (2-13.5 m) and basal sand (13.5-28 m). The geological sections agree with the clayey dominated soil interpreted from geotechnical results (Fig. 9). Consequently, adequate caution/ factor of safety must be taken during the design and construction of building, road, and embankment on the soil in the study area (Graham and Shields, 1984; Hadjigeorgiou et al., 2006).

3.4. Implication for building foundation design, embankment and pavement

3.4.1. Value for Building Foundation

The average ultimate bearing capacity of the soil for building foundation for depth-breadth ratio of 1.5 for strip and square footing are 18 kg/cm^2 and 26 kg/cm^2 respectively.

These bearing pressures are low and would only be suitable for lightweight structures, with adequate factor of safety. However proper soil improvement methods must be adopted (since clay tends to undergo volume change when desiccated) to ensure that the settlement is reduced in relation to the bearing pressure, since the soil are characterized by high compression index (av. 0.4489) which denotes a plastic clay, and high coefficient of volume compressibility (av. 0.7721 m^2/kN).

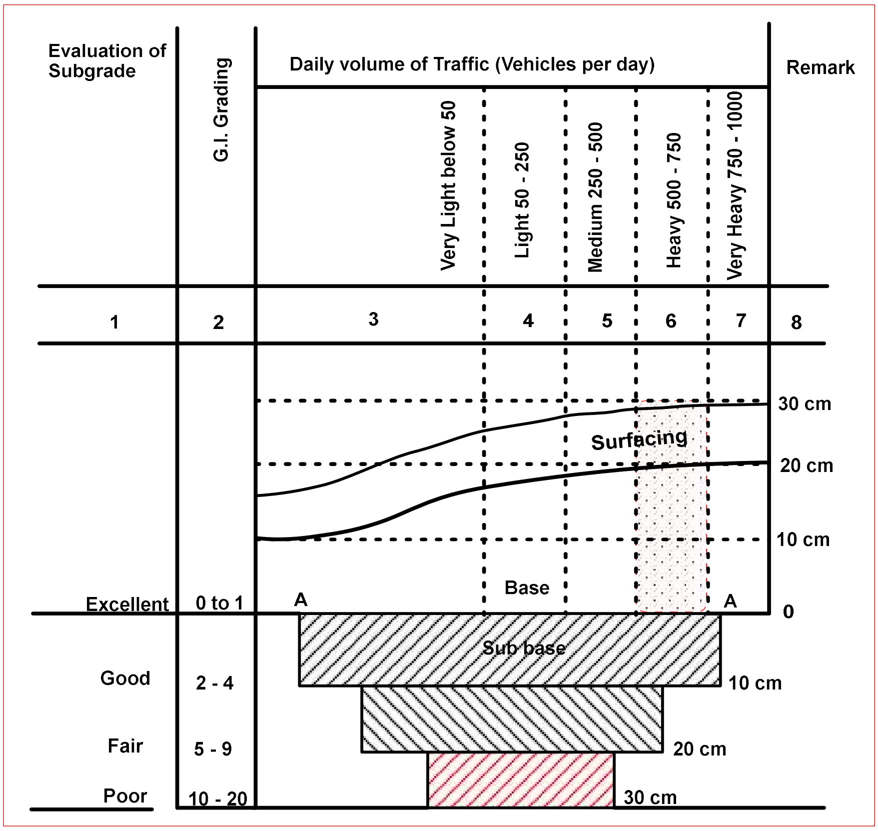

Fig. 10. Group Index Chart for the sampled soil showing the thickness of the sub-base and combined base and surface courses

3.4.2. Embankment

The USCS classification of the soil is CL-CH which depicts soil of fair to poor (compaction characteristics) stability for embankment slope, thin cores, blankets and dike sections (Black and Lister, 1978; Hadjigeorgiou et al., 2006).

3.4.5. Pavement and airfield

The value of the soil is unsuitable for base and sub-base courses, while the subgrade value of the soil ranges from fair – poor (with medium to high compressibility and expansion) especially when not subjected to frost action (FHWA, 2006). The drainage characteristics of the soil is practically impervious with soaked CBR less than 10. The design of flexible pavement is normally based on Group Index method or California Bearing Ratio method (George and Uddin, 2000; Wright, 1986). Fig. 10 showed the G.I. chart which is based on G.I. number and traffic intensity, which can be used to determine the combined thickness of surfacing, base and sub-base courses.

This is given by the ordinate above A-A as shown in Fig. 10, therefore, based on traffic count of vehicles plying the study area and, the traffic count was 288 vehicles per day. This was projected to be 550 vehicles per day in the next 3-5 years based on forecasted population census. Pavement is designed to carry future traffic, which usually increases over the years. Predicting future traffic growth is not always accurate. This inaccuracy in predicting future traffic affects the accuracy of predicting pavement performance and consequently pavement designed life (Brown, 1996).

Hence, daily volume of 550 vehicles per day was used for the design of the flexible pavement for the study area. Subsequently the combined thickness of base and surfacing should be 300 mm, of which 200 mm may be base and 100 mm for surfacing. Using the G.I. values range of 5-16 (av. 12), the sub-base should be about 30 cm (300 mm).

Nonetheless, the subgrade showed poor grade. The CBR method is used in determining the strength of subgrade and the construction material from an ad-hoc penetration test (Wright, 1986) and the thickness of the pavement is obtained from design curves (Fig. 11).

From the CBR chart, the thickness of 15 inches (38.1 cm) was obtained for sub-base/subgrade course. This was corroborated by succession of clayey material recorded from borehole logging and in-situ CPT. Definitely, the soil required improvement or stabilization either with chemical (lime, cement, fly ash) or by mechanical method, which would invariably reduce its plasticity/compressibility and invariably increase the shear strength and bearing capacity.

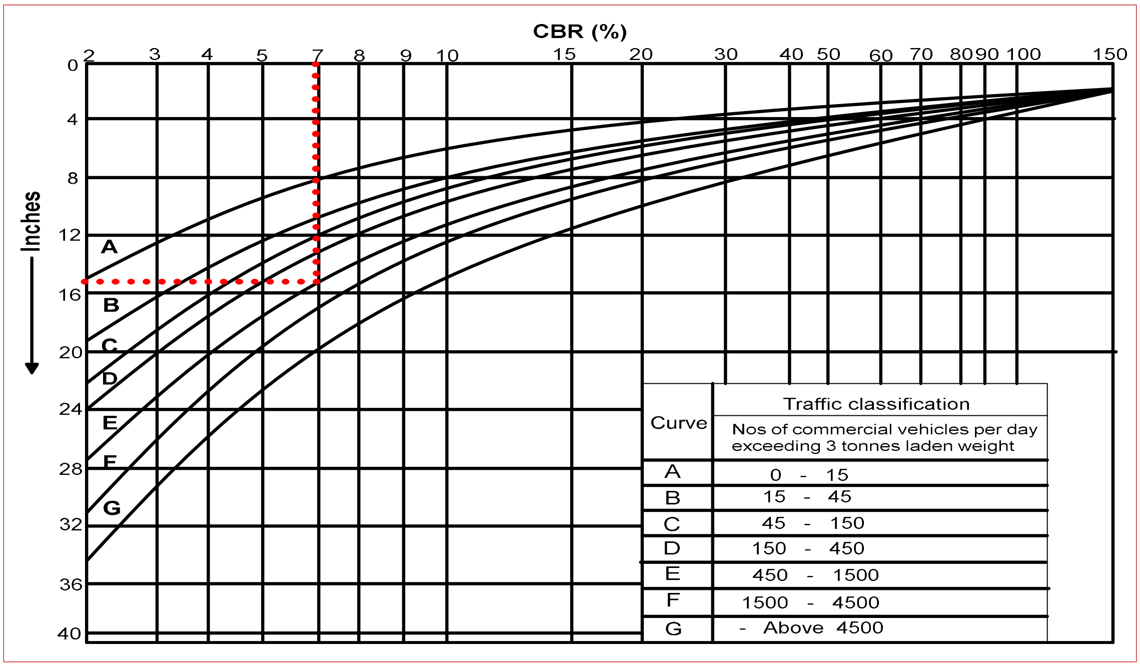

Fig. 11. CBR Design Curves as per IRC-37 (1970)

Conclusion

The study assesses the engineering properties of tropical red soil in Sobe Area of Edo State, Nigeria for embankment, flexible pavement sub-base/subgrade and building foundation footings, using geochemical, geotechnical, and borehole logging. Findings revealed that the soil is lateritic clay with an overlapping montmorilonite and illites characteristics, with high compressibility/expansion. The AASHTO class of the soil is A-7-6 and USCS of CH-CL with high activity, which is regarded as poor foundation material. All parameters measured fell short of Federal Ministry of Works and Housing specification for foundation material. However, the material showed average ultimate bearing pressure of 111 kN/m2 at 1 m depth for building foundation with enough compensation (factor of safety) for the expansible clayey material. The soil has fair to poor stability for embankment construction. The design for combined thickness of base and surfacing should be 30 cm and 38.1 cm for sub-base/subgrade course. Considering the poor nature of the soil and effect of flooding that usually characterized the area especially during the wet season as a result of high annual rainfall and low altitude (less than 100 m above the sea level), there is tendency of high moisture content that could induce undue settlement of the soil and cause serious threat to civil engineering structures.

Conflict of Interest

The authors declare no conflict of interest exists in this publication.

References

AASHTO, 2006. Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Parts I and II, American Association of State Highway and Transportation Officials, Washington, D.C.

Agbede, O.A., 1992. Characteristic of Tropical Red Soils as Foundation Materials. Nigerian Journal of Sciences 26, 237-242.

Arora, K.R., 2008. Soil mechanics and foundation engineering (Geotechnical Engineering). Standard Publishers Distributors, Delhi.

ASTM, 2006. Annual Book of ASTM Standards – Sections 4.02, 4.08, 4.09 and 4.13. ASTM International, West Conshohocken, PA.

Baligh, M.M., Azzouz, A.D., Wissa, A.Z.E., Martin, R.T., Marrison, M.H., 1981. The piezocone penetrometer. Symposium on Cone Penetration Testing and Experience, ASCE, geotechnical engineering division, St Louis, pp 247-263.

Bell, F.G., 1996. Lime stabilization of clay minerals and soils. Engineering Geology 42 (4), 223-237.

Bello, A.A., Adegoke, C.W., 2010. Evaluation of Geotechnical Properties of Ilesha East Southwestern Nigeria's Lateritic Soil. The Pacific Journal of Science and Technology 11 (2), 617-624.

Bose, B., 2012. Geoengineering properties of expansive soil stabilized with fly ash. Electronic Journal of Geotechnical Engineering 17, 1339-1353.

Black, W., Lister, N.W., 1978. The strength of fill sub-grades, its prediction and relation to road performance. ICE Conference on Clay Fills, 37-48. https://doi.org/10.1680/cf1979.00698.0004.

Bowles, J.E., 1984. Physical and Geotechnical Properties of Soils. McGraw-Hill Intl. Edi., Civil Engineering Series: London, UK. 1004.

Kamtchueng, B.T., Onana, V.L., Fantong, W.Y., Ueda, A., Ntouala, F.D., Wongolo, H.D., Ndongo, G.B., Ngo'o Ze, A., Kamgang, K.B., Ondoa, J.M., 2015. Geotechnical, chemical and mineralogical evaluation of lateritic soils in humid tropical area (Mfou, Central-Cameroon): Implications for Road Construction. International Journal of Geo-Engineering 6 (1). https://doi.org/10.1186/s40703-014-0001-0.

Brosten, T.R., Llopis, J.L., Kelley, J.R., 2005. Using geophysics to assess the condition of small embankment dams. US Army Corps of Engineers of Engineer Research and Development Center, 28pp.

Brown, S.F., 1996. Soil mechanics in pavement engineering. Geotechnique 46, 383-426.

Cetin, K.O., Ozan, C., 2009. CPT-based probabilistic soil characterization and classification. Journal of Geotechnical and Geoenvironmental Engineering 135 (1), 84-107.

Charman, J.H., 1988. Laterite in Road Pavements. Special Publication 47, Construction Industry Research and Information Association, London.

Coker, J.O., 2015. Integration of Geophysical and Geotechnical Methods to Site Characterization for Construction Work at the School of Management Area, Lagos State Polytechnic, Ikorodu, Lagos, Nigeria. International Journal of Energy Science and Engineering 1 (2), 40-48.

Das, B.M., 2015. Principles of foundation engineering. Cengage learning, Boston.

Douglas, B.J., Olsen, R.S., 1981. Soil classification using electric cone penetrometer. In: Norris GM, Holtz RD (eds) Proceedings of the symposium on cone penetration testing and experience, St. Louis, Mo., 26–30 October 1981. Geotechnical engineering division, American Society of Civil Engineers, New York, pp 209-227.

Elarabi, H., Taha, M., Elkhawad, T., 2013. Some Geological and Geotechnical Properties of Lateritic Soils from Muglad Basin Located in the South-Western Part of Sudan. Research Journal of Environmental and Earth Sciences 5 (6), 291-294.

Ezenwaka, K.C., Ugboaja, A., Ahaneku, C.V., Ede, T.A., 2014. Geotechnical investigation for design and construction of civil infrastructures in parts of port Harcourt City of Rivers State, Southern Nigeria. The International Journal of Engineering Science 3, 74-82.

Falowo, O.O., Olabisi, W.K., Olaifa, G.E., 2017. Geochemical and Geotechnical Appraisal of the Regolith along Ado-Ilawe Road, Ekiti State, Nigeria for Engineering Construction. International Journal of Advance Research and Engineering Trends 2 (11), 277-284.

FMWH, 1997. Federal Ministry of Works and Housing, General Specifications for Roads and Bridges II, 145-284. Federal Highway Department: Lagos, Nigeria.

FHWA NHI-05-037, 2006. Geotechnical aspects of pavement. U.S. Department of Transportation Federal Highway Administration, 4-17.

Gadallah, M., Fisher, R., 2009. Exploration Geophysics, Verlag Berlin Heidelberg Publication, Springer, 274 pp.

Garg S. K., 2007. Physical and Engineering Geology, Khanna Publisher, Delhi, India, pp. 338-348.

George, K.P., Uddin, W., 2000. Subgrade characterization for highway pavement design. Final Report, Jackson, MS: Mississippi Department of Transportation.

Gidigasu, M.D., 1976. Laterite Soil Engineering. Elsevier Scientific Publishing Co., New York.

Gidigasu, M.D., Kuma, D.O.K., 1987. Engineering Significance of Laterisation and Profile Development Processes. Proceedings of 9th Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Lagos, Nigeria.

Graham, J., Shields, D.H., 1984. Influence of geology and geological processes on the geotechnical properties of a plastic clay. Journal of Engineering Geology 22 (2), 109-126.

Hadjigeorgiou, J., Kyriakou, E., Papanastasiou, P., 2006. A road embankment failure near Pentalia in Southwest Cyprus. The South African Institute of Mining and Metallurgy International Symposium on stability of rock slopes in open pit mining and Civil Engineering situation, Symposium Series S44 3–6, Victoria and Alfred Waterfront, Cape Town, 11.

Hausmann, M.R., 1990. Engineering principles of ground modification. McGraw-Hill, Maidenheach.

Ilori, A.O., 2015. Geotechnical characterization of a highway route alignment with light weight penetrometer (LRS 10), in southeastern Nigeria, International journal of Geo-Engineering 6 (7), 28.

IRC: 37, 1970. Guidelines for the design of flexible pavements, The Indian Roads Congress, New Delhi.

Jegede, O.G., Olaleye, B.M., 2013. Evaluation of engineering geological and geotechnical properties of Subgrade soils along the Re-Aligned Igbara-Odo Ikogosi Highway, Southwestern, Nigeria. The International Journal of Engineering and Science (IJES), 2 (5), 18-21.

Jones, H.A., Hockey, R.D., 1964. The Geology of Parts of Southwestern Nigeria. Lagos. Geological Survey of Nigeria Bulletin 31, 101.

Keller, G.V., Frischknecht, F.C., 1966. Electrical methods in Geophysical Prospecting. Oxford: Pergamon Press Inc.

Kogbe, C.A., 1976. Paleogeographic History of Nigeria from Albian Time. In: Geology of Nigeria. C.A. Kogbe (ed.). University of Ife: Nigeria, 237-252.

Logmo, E. O., Ngon Ngon, G.F., Samba, W., Mbog, M.B., Etame, J., 2013. Geotechnical, mineralogical and chemical characterization of the missole II clayey materials of Douala Sub-Basin (Cameroon) for construction materials. Open Journal of Civil Engineering 3 (2), 46-53.

Lowrie, W. 2007. Fundamental of Geophysics, second edition, Cambridge University Press, UK, 393 pp.

Maigien, R., 1966. Review of Research on Laterites. Natural Resources Research IV, United Nations Educational Scientific and Cultural Organization, Paris.

Martin, F.J., Doyne, H.C. 1927. Laterite and lateritic soils in Sierra Leone. Journal of Agricultural Science 17, 530-547.

Mayne, P.W., 2007. Cone penetration testing: a synthesis of highway practice. Project 20-`5. Transportation Research Board, Washington, D.C. NCHRP Synthesis 368.

Milson, J., 2003. Field Geophysics: The Geological Field Guide Series, 3rd Ed., Published John Wiley and Sons Ltd. 88 pp.

Mitchell, J.K., Soga, K., 2005. Fundamentals of soil behavior, 3rd edn. Wiley, New York. ISBN: 978-0-471-46302-3

Moss, R.E.S., Seed, R.B., Olsen, R.S., 2006. Normalizing the CPT for overburden stress. Journal of Geotechnical and Geoenvironmental Engineering 132 (3), 378-387.

Nigerian Geological Survey Agency, 2006. Geological and ineral Map of Akwa-Ibom State, Nigeria.

Nwankwoala, H.O., Amadi, A.N., Warmate, T., Jimoh, M.O., 2015. Geotechnical Properties of subsoils in Escravos Estuary, Western Niger Delta, Nigeria. American Journal of Civil Engineering and Architecture 3(1), 8-14.

Nwankwoala, H.O., Amadi, A.N., Ushie, F.A., Warmate, T., 2014. Determination of Subsurface Geotechnical Properties for Foundation Design and Construction in Akenfa Community, Bayelsa State, Nigeria. American Journal of Civil Engineering and Architecture 2 (4), 130-135.

Nwankwoala, H.O., Warmate, T., 2014. Geotechnical assessment of foundation conditions of a site in Ubima, Ikwerre Local Government Area, Rivers State, Nigeria. International Journal of Research Development 9, 50-63.

Obaje, N.G., 2009. Geology and mineral resources of Nigeria. Lecture Notes in Earth Sciences, Springer, Berlin, Heidelberg 120.

Ogbe, F.G.A., 1972. Stratigraphy of Strata Exposed in Ewekoro Quarry, Western Nigeria. In: Dessauvagie, T.F.J. and Whiteman, A.J., Eds., African Geology, University of Ibadan Press, Ibadan, 305-322.

Oghenero, A.E., Akpokodje, E.G., Tse, A.C., 2014. Geotechnical properties of subsurface soils in Warri, Western Niger Delta, Nigeria. Journal of Earth Sciences and Geotechnical Engineering 4, 89-102.

Oladeji, O.S. and Raheem, A.A. 2002. Soil Tests for Road Construction. Journal of Science, Engineering and Technology 9 (2), 3971-3981

Olayanju, G.M., Mogaji, K.A., Lim, H.S., Ojo, T.S., 2017. Foundation integrity assessment using integrated geophysical and geotechnical techniques: Case study in crystalline basement complex, southwestern Nigeria. Journal of Geophysics and Engineering 14, 675-690.

Omatsola, M.E., Adegoke, O.S., 1981. Tectonic Evolution and Cretaceous Stratigraphy of the Dahomey. Basin. Journal of Mining and Geology 18 (1), 130-137.

Peck, R.P., Hanson, E., Thornburn, T.H., 1974. Foundation engineering, 2nd edn. John Wiley and Sons, New York, pp 28-337.

Pedarla, A., Chittoori, S., Puppala, A., 2011. Influence of mineralogy and plasticity index on the stabilization effectiveness of expansive clays. Transportation Research Record Journal of the Transportation Research Board 2212, 91-99.

Radhakrishnan, G., Kumar, M.A., Raju, G., 2014. Swelling properties of expansive soils treated with chemicals and fly ash. American Journal of Engineering Research 3 (4), 245-250.

Reyment, R.A., 1965. Quantitative Paleoecologic Analysis of Ewekoro and Oshosun Formations of Western Nigeria. Geologiska Föreningen i Stockholm Förhandlingar 86, 248-256.

Robertson, P.K., 1990. Soil classification using the cone penetration test. Canadian Geotechnical Journal 27, 151-158.

Rogers, J.D., 2006. Subsurface exploration using the standard penetration test and the cone penetrometer test. Environmental and Engineering Geoscience 12 (2), 161-179.

Roy, S., Bhalla, S.K., 2017. Role of geotechnical properties of soil on civil engineering structures. Resources and Environment 7 (4), 103-109.

Sanglerat, G., 1972. The penetration and soil exploration, Development in geotechnical engineering, Elsevier Scientific Publishing, New York.

Schmertmann, J.H., 1978. Guidelines for cone test, performance, and design. Federal highway administration, Washington, D.C. Report FHWA-TS-78209.

Sharma, P.V., 1997. Environmental and Engineering Geophysics. Cambridge University Press, Cambridge.

Sherwood, P., 1993. Soil stabilization with cement and lime. Transport Research Laboratory, College Park.

Shiva, P., Kumar, K., Darga, K.N., 2016. Use of CPT and DCP based correlations in characterization of subgrade of a highway in Southern Ethiopia Region. International Journal of Geo-Engineering 7 (11), https://doi.org/10.1186/s40703-016-0025-8.

Singh, P., 2008. Engineering and General Geology, 8th Revised Edition, S.K. Kataria & Sons, New Delhi, 591 pp.

Soupios, P.M., Papazachos, C.B., Vargemezis, G., Fikos, I., 2005. Application of modern seismic methods for geotechnical site characterization Proc. Int. Workshop in Geoenvironment and Geotechnics (Milos Island, Greece, 12–14 September) pp 163-70.

Soupios, P., Papazachos, C.B., Vargemezis, G., Savvaidis, A., 2006. In situ geophysical investigation to evaluate dynamic soil properties at the Ilarionas Dam, Northern Greece Proc. 2nd Int. Conf. Advances in Mineral Resources Management and Environmental Geotechnology (Hania, Crete, Greece, 25–27 September) (Heliotopos Conferences) pp 149-56.

Telford, W.M., Geldart L.P., Sheriff, R.E., 1990. Applied Geophysics. Cambridge University Press, London. Wright, P.H. 1986. Highway Engineering, Sixth Edition. John Willey and Sons: New York, NY.

Venkatranmaiah, C., 2006. Geotechnical Engineering. New Age International Limited Publisher, pp. 541-603.