MJSAT 3 (1) 48-71

MJSAT

Malaysian Journal of Science and Advanced Technology

journal homepage: https://mjsat.com.my/

Subsoil Condition and Development of Geo-Data Set for Effective Design, Construction and Management of Engineering Structures in Ondo Metropolis, Southwestern Nigeria

Olumuyiwa Olusola Falowo*[1]

[1] *Dept. of Civil Engineering Technology, Rufus Giwa Polytecchnic Owo, Ondo State, Nigeria*

KEYWORDS

Ondo metropolis
Suitability index
Undrained shear strength
Foundation

ARTICLE HISTORY

Received 19 March 2023
Received in revised form 25 March 2023
Accepted 30 March 2023
Available online 31 March 2023

ABSTRACT

This study deals with development of engineering geo-data and modelling, as an essential tool for design, construction, and management of civil engineering structures in Ondo metropolis, Southwestern Nigeria. Findings revealed the soils are predominantly of sandy silt to clayey silt, silty sand to sandy silt, and sandy clay, with %fines of 48.25, and 51.75 % sand. The depth to basement rock was 22.2 m (avg.), and evidenced of fractured rock at depth range of 9.9 – 26 m. The average static water levels 2.5 m (in well) – 26 m (in borehole). The average values of plasticity index, soaked CBR, group index, unconfined compressive strength, and permeability are 23.9%, 7%, 6, 186.8 KN/m^2, and 1.68E-05 cm/s respectively. The recommended minimum thickness of 79 – 140 mm (avg. 109 mm) obtained from design curves will be sufficient for flexible pavement. The average allowable bearing capacity of the soil for square and round foundations varied from 234 – 297 KN/m^2 (avg. 268 KN/m^2) and 232 – 298 KN/m^2 (avg. 268 KN/m^2) with estimated total settlement of 17.69 – 18.88 mm (avg. 18.28 mm) for structural pressure of 100 KN/m^2. For embankment, the suitability index of the soil suggests a fair/expanding not collapsible construction material. Rocks of igneous and metamorphic rock of high compressive/shear strength, modulus of elasticity, high crushing strength, low deformability, and presumable bearing capacity of 8, 000 – 10, 000 KPa when fresh, and between 5000 – 7000 KPa when slightly weathered. Therefore are valuable as foundation constructions, aggregate in pavement, building stone, and armourstones. The correlation coefficients give between MDD/PI vs. CBR (0.1113), LL vs. coefficient of consolidation (0.0018), PI vs. undrained shear strength/effective overburden (0.0332), PI vs. angle of shearing (0.013), dry density vs. angle of shearing (0.2131), suitability index vs. CBRs (0.3494), clay contents vs. PI (0.422).

1. INTRODUCTION

An adequate ground investigation is an essential preliminary to the execution of a civil engineering project [1], [2], [3], [4]. Sufficient information must be obtained to enable a safe and economic design to be made and to avoid any difficulty during construction. The principal objects of the investigation are: to determine the sequence, thicknesses and lateral extent of the soil strata and, where possible, the depth to bedrock; to obtain representative samples of the soils (and rock) for identification and classification, and if necessary, for use in laboratory tests to determine relevant soil parameters; and to identify groundwater conditions [5], [6], [7], [8]. The investigation may also include the performance of in-situ test to assess appropriate soil characteristics. Additional considerations arise if it is suspected that the ground may be contaminated. The results of a ground investigation should provide adequate information, so that most suitable type of foundation for the proposed structure can be selected and to

*Corresponding author:
E-mail address: Olumuyiwa Olusola Falowo < mogfalowo@yahoo.co.uk >.

indicate if special problems are likely to arise during excavation [9], [10], [11].

One of the main cause of civil engineering structural failure is inadequate soil analysis and modelling resulting in poor foundation design and construction [12], [13]. Geophysics as well known, is the application of the principle of physics to the study of the earth [14], [15]. They are very good for engineering site investigation especially at reconnaissance stage [16], [17]. However, the methods are not suitable for all ground conditions. It can help to locate strata boundaries only if the physical properties of the adjacent materials are significantly different [18], [19]. The result of the geophysical survey can be cross checked against data obtained by direct methods such as boring. The method can provide rapid and economic results, especially where filling-in of details is required for widely spaced sample locations. It is also useful in estimating the overburden thickness or depth to bedrock. Geophysical methods are based on distinct physical properties of rocks [16]. The radiometric method measures variations in rock radioactivity. The magnetic method measures variations in magnetic susceptibility; the gravity method measures variation in variations in earth's density. The electrical methods measure change in the conductivity of rocks; seismic method is based on acoustic impedance contrast. The direct current resistivity method is particularly very useful as a rapid means of rock identification and site selection in foundation design and construction [20]. The electromagnetic method is useful as a reconnaissance tool in groundwater investigation for borehole drilling and installation; but it is however not as reliable as the resistivity method in terms of its sounding capacity and depth of penetration [15], [21].

The electrical resistivity of a formation is directly related to the nature, quality and quantity, and distribution of the formation water [21], [22], [23]. The resistivity surveying involves the passage of current into the ground by means of two electrodes (current electrodes) while the potential drop is measured between a second electrodes (potential electrodes). The two frequently used electrode configurations in engineering site investigation studies are the Wenner and Schlumberger [24] (figures 1 and 2 respectively). In wenner array, four electrodes are positioned in such a way that, potential electrode are in between the current electrodes, and the spacing between two adjacent electrodes is one third [15], [24]. But in Schlumberger configuration, the spacing between the potential electrodes must not exceed 40% of the half the distance of the spacing (AB) of the current electrodes [24]. For the Schlumberger VES array, the apparent resistivity is obtained from the equation 1:

$$\rho_s = {\pi R L^2}/{2L} \tag{1}$$

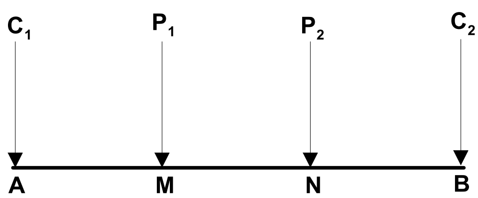

Fig. 1. The Wenner Electrode Configuration

C_1 and C_2 are current electrodes while P_1 and P_2 are potential electrodes, "A" is inter electrode spacing (AB/3).

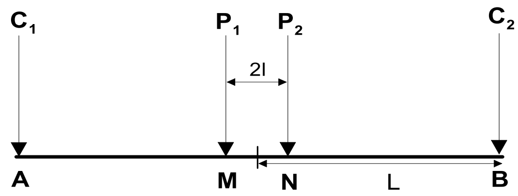

Fig. 2. The Schlumberger Electrode Configuration

Where ρ_s is the apparent resistivity (ohm-m), L = AB/2 i.e. half current-current electrode spacing, l is half potential – potential electrode spacing (m), while π is a constant

In field survey, the distance MN is kept constant while AB is expanded resulting in a rapidly decreasing potential difference across MN that ultimately exceeds the measuring capabilities of the instrument [24]. The VES could be used to investigate the subsoil in terms of water table, soil characteristics, the extent, depth and spread of each type of soil and delineation of the quality and depth of the competent soil type in civil engineering works. The resistivity method is also very useful in groundwater investigation, engineering site investigation, environmental impact assessment, mineral exploration, geothermal exploration, and geological mapping [24]. The magnetic method measures differences in magnetic susceptibility of rocks. All rocks, mineral and ore deposits are magnetized to a lesser or greater extent by the earth's magnetic field. As a consequence, in magnetic surveying, accurate measurements are made of the anomalies produced in the local geomagnetic field by this magnetization [21]. The intensity of magnetization and hence the amount of by which the earth's magnetic field is changed locally depend on the magnetic susceptibility of the material concerned. In addition to the magnetism induced by the earth's field, rocks possess a permanent magnetism that depends on their history. The strength of the magnetic field is measured in nanoTesla. The measured magnetic susceptibility are presented as isomagnetic contour maps of anomalies or as profiles across trend of linear anomalies with stations, at interval of as little as 1 m. A base station is set up beyond the anomaly where the geomagnetic field is uniform. The reading at the base station is taken as zero, and all subsequent readings are expressed as plus-or-minus differences. In CPT test the 10 cm^2 probe with a 60° tip attached to a series of rods is continually pushed into the ground. Typically the equipment consisting of a thrust mechanism, reaction frame and push rods are used to continually advance the cone into the ground at a rate of 20mm/s. A friction sleeve with a surface area of 1500mm^2 is located behind the conical tip. But in load cells are to continually measure the cone tip resistance Q_C and the sleeve friction resistance F_S [25], [26]

2. Description of the Study Area

Ondo metropolis is within the Ondo State, southwestern Nigeria, lies between the geographic coordinates of Eastings 754900 and 756500 mE and Nothings 859700 and 860900 mN of Zone 31N (Minna datum) in the Universal Traverse Mercator (UTM) coordinate system (figure 3). The climate is tropical, characterized by dry and wet seasons. The dry season lasts from November to March, while the wettest months are August and September, with a mean annual rainfall of 180 cm [27]. The relative humidity of the area drops from 85% to 95% between June and October to about 25% to 35% between November and December; the annual temperature ranges from 25°C to 31°C, and mean temperature of 24 °C [28]. The study area covers part of Ondo East and Ondo west as the area is accessible through asphaltic roads connecting Akure – Ondo - Ore highway, Ipetu – Ijesa – Ile oluji highway, and Ife – Okeigbo – Ondo highway. The study area is generally characterized by flat and gently undulating topography.

The area is situated in the deciduous rain forest area within south-western Nigeria. It has evergreen vegetation and urban settlement. The vegetation of the area is the rain forest type with dense evergreen forest of tall trees with thick vegetation (which may reach a height of 15 m and even more), and of different plants. They consist of light forests, shrubs, scattered cultivation, trees and plants like timber, oil palm, kolanut, rubber, cocoa and citrus are very prominent in the area. Topographic elevations vary from about 2 to 10 m above sea level. The study area is situated within the Precambrian Basement Complex with the outcrops which are Granite, gneiss and migmatite (figures 4 and 5). The rocks in the study environment is predominantly migmatitic, with the most predominant components being the granite-gneiss and grey gneiss. These rocks are covered by regoliths with thickness variation across the area. Topographically, the area is relatively rugged and undulating, with elevation of 300 -800 m above sea level.

3. MATERIAL AND METHODS

The general objectives of a site investigation is to assess the suitability of a site for the proposed purpose. As such, it involves exploring the ground conditions at or below the surface [21], [30], [31], [32]. It is a prerequisite for the successful and economic design of engineering structures and earthworks [10][33]. Accordingly a site investigation also should attempt to foresee and provide against difficulties that may arise during construction because of ground and/or other local conditions [34].

The method of study adopted for this research started with desk study, to preliminary investigation, main field survey, and laboratory studies [35]. The desk study was undertaken in order to make an initial assessment of the ground conditions and to identify any potential geotechnical problems. Also examination of literature maps, imagery and photographs relevant to the area. The study of geological map help to give an indication of the probable soil conditions of the area, since soils are product of weathering of rocks. Before the start of the field work an inspection of the site and the surrounding area was made on foot. River, existing excavation, road cutting, quarry site, Existing structures were also examined for signs of settlement damage. All these yielded pertinent information regarding the nature of the strata and groundwater conditions. A preliminary investigation on a modest scale may be carried out to obtain the general characteristics of the strata, followed by a more extensive and carefully planned investigation including sampling and in-situ testing. It was essential that the investigation was taken to adequate depth, so as to capture all strata liable to be significantly affected by any proposed structures [36], [37], since the soil is being investigated for different construction/structural purposes. The data acquisition map of the study is shown in figure 6.

The trial pits is a simple and reliable method of investigation, from which representative samples can be taken, and geologic strata can be examined [39], [40]. For this study, 20 trial pits were dug by means of hand-digger to a depth between 1 – 3 m. The excavated soil was placed about 1.0 m away from the edge of the pits [41]. No groundwater table was observed during the exercise. The samples were collected from the pits from its sides/bottom (for disturbed sample), while tube samples were collected below the bottom of the pit (for undisturbed). The disturbed samples were collected for shear strength parameters determination and consolidation test. Immediately, the pits were examined and samples collected, they were sand filled after use. The use of trial pits enables the in-situ soil to be examined visually, and thus the boundaries between strata and the nature of any macro-fabric can be accurately determined.

The GPS was used to take the coordinates of all sampling locations for all field surveys. The GPS is cost effective and time saving to traditional use of theodolites and levels. The geotechnical parameters were analyzed using America Standard for Testing and Material [42] and British Standard [43] procedures, with the following tests: natural moisture content (D2216), grain size distribution (D422; D1140), specific gravity (D854; D5550), consistency limit and linear shrinkage (D4318), density (BS 1377), triaxial (D4767; D2850), unconfined compressive strength (D2166), permeability (D2434), compaction (D1557; D698), California Bearing Ratio, one dimensional consolidation (D4186; D4546). The in-situ CPT test was done following ASTM-D3441-94 procedures.

The CPT equipment utilized the Dutch cone penetrometer with an anvil, driving rod, and other accessories. The machine nominal capacity was 10-tonnes and was operated by using hydraulically operated driving mechanism. The cone tip angle of the penetrometer used was 60° and rods of 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 (q_c), sleeve friction (f_s) 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 [44], [45], [46]. The layer sequences were interpreted using the friction ratio (figure 7), while cone resistance contrast between the various layers, inflection points of the penetrometer curves were interpreted as the interface between the different lithologies [44], [45]. 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 (qcn) of 100 KN/m^2 [46]. Hence from the result of the CPT, unconfined compressive strength (equation 2), ultimate bearing capacity [47] was derived (equations 3 and 4), ultimate capacity (Q_{ult}) and elastic modulus for strip and square using equations 5, 6 and 7 respectively, SPT - N_{cor} (equation 8) and Modulus number (equation 9).

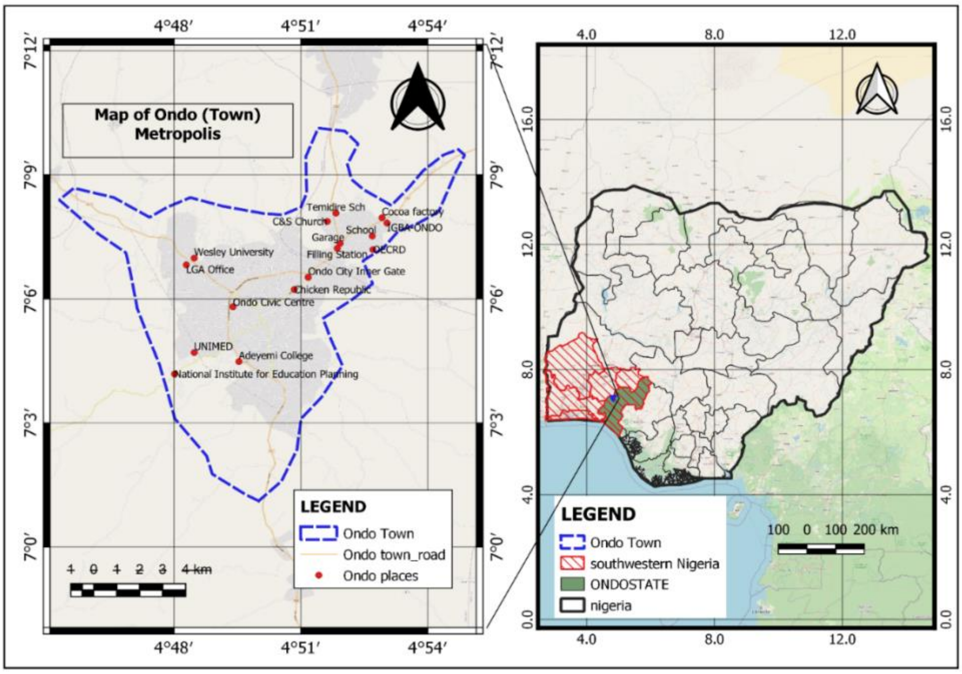

Fig. 3. Location map of the Study Area on map of Nigeria

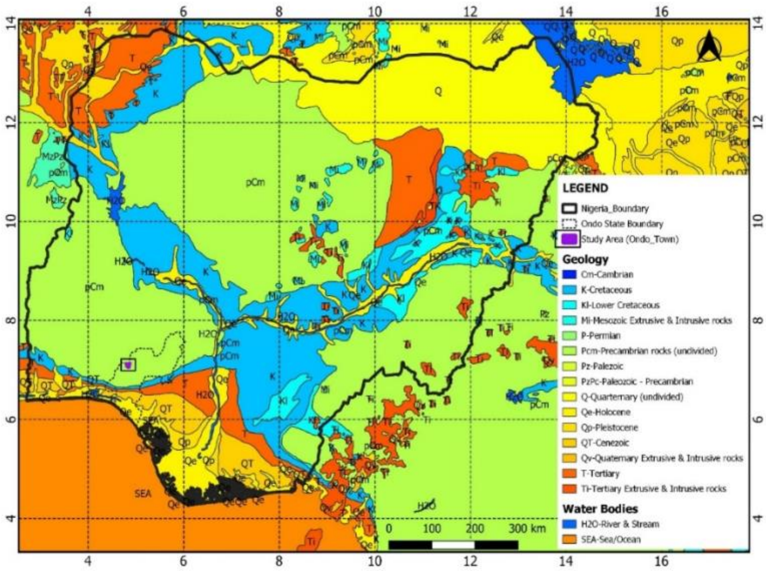

Fig. 4. Geological map of Nigeria showing the study area within the Southwestern Basement Complex (Modified after NGSA [29])

$$c_u = q_{cn}/N_k \tag{2}$$

where C_u is unconfined compressive strength, 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 $D/B \leq 1.5$ (in kg/cm^2):

$$\text{Strip: } Q_{ult} = 2 + 0.28q_c \tag{3}$$

$$\text{Square: } Q_{ult} = 5 + 0.34q \tag{4}$$

$$Q_{ult} = Q_{cn}/40 \ \ (\text{in kg/cm}^2) \tag{5}$$

$$E_{strip} = 3.5 \times Q_{ult} \tag{6}$$

$$E_{square} = 2.5 \times Q_{ult} \tag{7}$$

$$N_{cor} = \frac{Q_c}{4} \tag{8}$$

$$\text{Modulus Number} = 22.4\text{CBR}^{0.5} \tag{9}$$

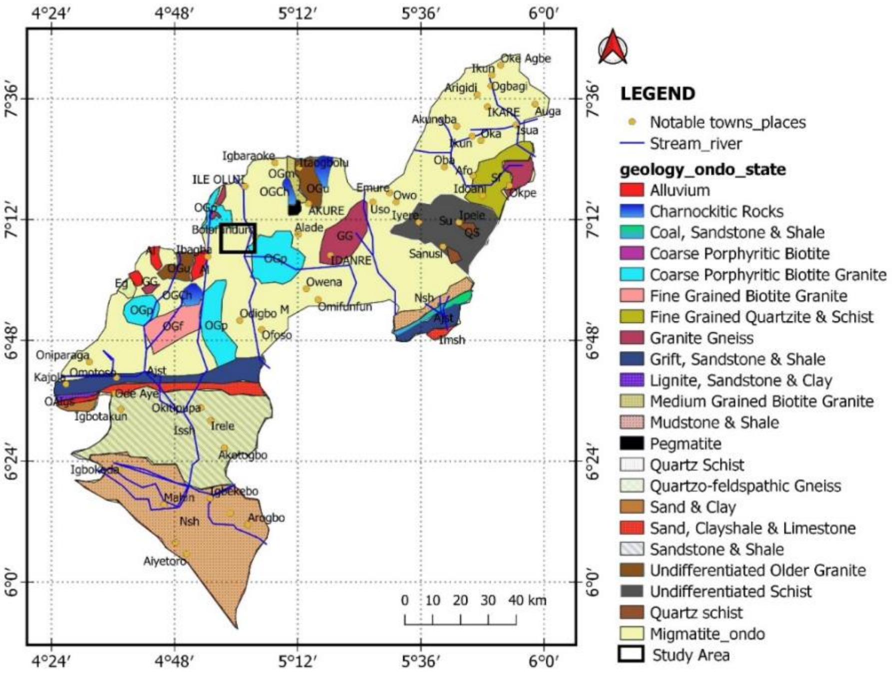

Fig. 5. Geological Map of Ondo State, showing the local rock units which included Migmatite and Coarse – Porphyritic Biotite Granite (modified after NGSA [38])

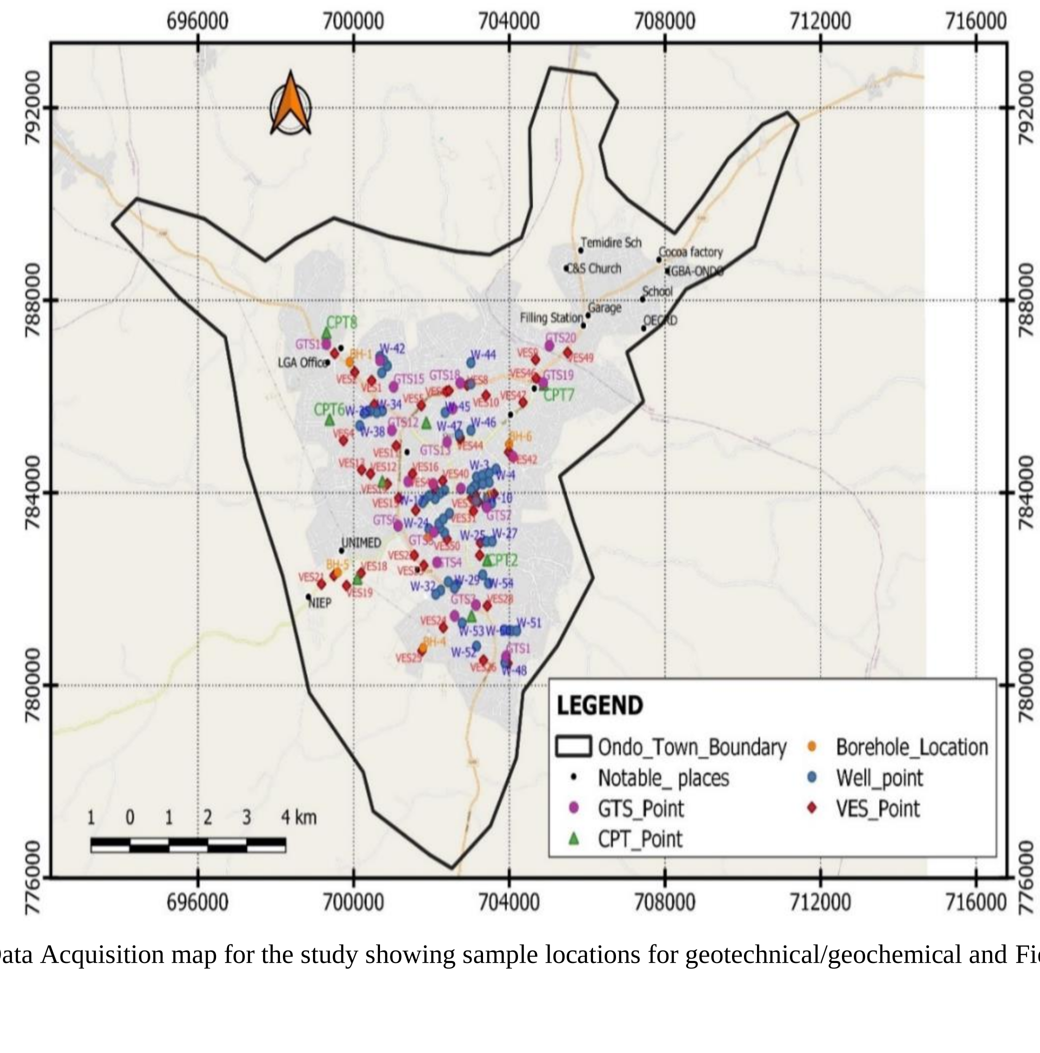

Fig. 6. Data Acquisition map for the study showing sample locations for geotechnical/geochemical and Field Survey

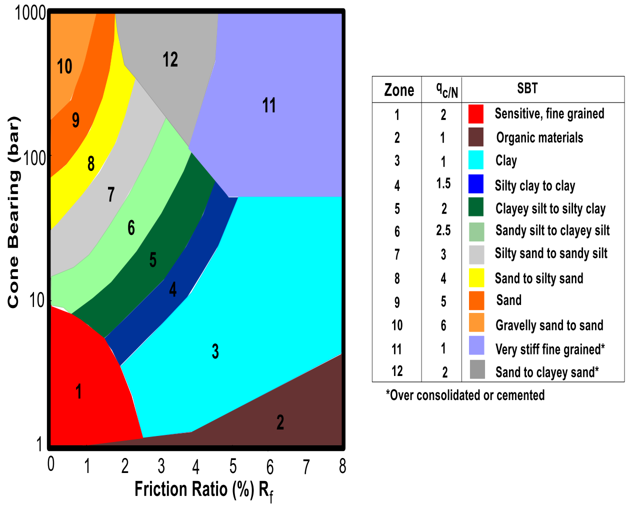

Fig. 7. Robertson Chart for Soil Classification using Cone Resistance Value and Friction Ratio [45]

From the analysis, the followings were derived: settlement (both elastic and consolidation), activity (equation 10), Group Index (GI), AASHTO and USCS classifications, suitability index (equation 11), bearing pressure models were developed from CPT results using Hatanaka and Uchida [48], Meyerhof [49], and Schmertmann [50] equations; with corresponding stresses (mean, +ve, and –ve stresses) using Burland and Burbidge [51] model. Correlations were made between parameters: MDD/PI vs. CBR, LL vs. coefficient of consolidation, PI vs. undrained shear strength/effective overburden, PI vs. angle of shearing, dry density vs. angle of shearing, suitability index vs. CBR, clay contents vs. PI. Mineralogy and micro fabric of the clay structure are studied using X-ray diffraction, differential thermal analysis and scanning electron microscope. In this study, the geochemical analysis was done using X-ray diffraction.

$$A = \frac{PI}{\%\, finer\ than\ 2.0\ mm} \tag{10}$$

$$Si = \frac{\%\, finer\ than\ 2.0\ mm}{LL\log(PI)} \tag{11}$$

The acquisition of VES data was done using the field procedures of Falowo and Dahunsi [52] and Falowo and Olabisi [53] using Schlumberger array with maximum current – current spread of 130 m, and potential – potential distance of 5m. A total of fifty VES was acquired. The quantitative interpretation of the VES curves involved partial curve matching and computer iteration technique. This technique assumes that the earth is made up of horizontal layers with differing resistivities. Any significant deviation (in dip angle greater than 10%) from this planar assumption in the stratigraphy will slightly distort the VES curve and introduce error in the VES interpretation results. Other sources of error are lateral inhomogeneity, suppression and equivalence. All these were taken care of during data analysis and interpretation. The depth sounding interpretation are presented as geoelectric section, which showed horizontal to near horizontal stratification of the subsurface geologic layers. Magnetic method was also used, with measurements taken at 1 m interval along a traverse with GSN 8 Proton Precision Magnetometer. The field procedures was in line with Falowo et al. [54]. The distance covered for the survey was 500 m, the same traverse established for the VES. Two sets of data were collected at each location and average determined, with sensor height at 1.5 m. The base station readings were taken before and after the data acquisition. The base station reading was used to correct the data for diurnal and offset corrections.

An important part of any ground investigation is the determination of water table level and any artesian pressure and its chemistry. The variation of level or pressure over a given period of time may also require determination. Groundwater observations are of particular importance if deep excavation are to be carried out. Water table level can be determined by measuring the depth to the water table in a borehole after stabilization, and it depends on the formation permeability [55]. Measurement therefore must be taken at regular intervals until the water level becomes constant. In addition to determination of the chemical elements in the soil samples, which is the product of rock dissolution or rock-water interaction. The chemical elements present in groundwater was not determined, even though is one of important aspect of engineering site investigation, particularly where salinity or the presence of corrosive effluents is suspected that can have deleterious effect on concrete such as sulphates and acidic waters, acid, bacteria, and oxidizing agents will affect steel foundation structures. These features were not reported in the water samples from desk study/literature review. In addition, personal interview with had with inhabitants of the town responded negatively to aforementioned elements. Consequently, no water quality test was carried. However, the static water level, hydraulic head determination, and hydraulic conductivity was determined from fifty five open wells and six boreholes.

4. Results and Discussions

4.1 Vertical Electrical Sounding

The summary of the VES is presented in Table 1, while a typical geologic section prepared for VESs 4, 12, 13, 14, 15, and 17, in NW – SE trend, is shown in figure 8. The curve types obtained from the study area varied from three layer curves (H), four layer curves (KH, HK, and QH), and five layer curves (HKH and KHK). The H curve type is the most preponderant (42 %) followed by KH (30 %), KHK (12 %), HKH (8 %), QH and HK (4 %). This implies that the area is generally made of high resistive topsoil, underlain by high conductive weathered layer, and basement rock. From the Table 1, topsoil has resistivity ranging from 25 – 490 ohm-m (avg. 196 ohm-m) and thickness varying from 0.5 – 2.4 m (avg. 1.20 m) and composed of clay, sandy clay and clayey sand. The subsoil is characterized with resistivity ranging from 44 – 649 ohm-m (avg. 326 ohm-m) and have same composition as the topsoil, with thickness ranging from 2.4 to 14.9 m (avg. 5.80 m). The weathered layer has resistivity ranging between 18 ohm-m and 732 ohm-m (avg. 148 ohm-m), indicating clayey weathered layer; the thickness ranged from 6.6 m and 24.1 m (avg. 15.1 m). The fractured basement has resistivity of 89 – 689 ohm-m (avg. 388 ohm-m), with thickness of 25.0 to 38.5 m (avg. 29.8 m (VES 45). The depth to this layer is between 9.9 m and 26.0 m (avg. 14.0). The depths to basement rock (113 – 3650 ohm-m (avg. 1085 ohm-

m) varied from 10.8 – 35.1 m (avg. 22.2 m), indicating moderate/thick weathering profile. Consequently, the topsoil, subsoil, and weathered layer are generally composed of sandy clay material, which can be regarded fairly competent soil material to support the civil engineering structures. Typical section shown in figure 7 are characterized by topsoil (65 – 305 ohm-m), subsoil (44 – 602 ohm-m), weathered layer (18 – 455 ohm-m), fractured basement (89 – 528 ohm-m) and basement rock (389 – 2330 ohm-m). The relief of the basement is uneven.

4.2 Magnetic method

The relative magnetic field intensity along the profile (figure 9) established for the VESs 4, 12, 13, 14, 15, and 17, in NW – SE trend showed amplitude variation of -479 nT to 151.70 nT (avg. -14.30 nT). This range of value is not unusual in basement complex, as similar values of -284 to 228 nT, -391 to 114 nT, -199 to 856 nT had been reported by Falowo et al. [54]. The profile showed relatively flat anomaly, which can be considered as magnetically homogeneous environment. However low magnetic anomalies observed as distances 420 – 444 m are indication of structural features such as fracture, lineation, fault or joint system. This feature reflected on the geoelectric profile as fractured zone. Consequently there is high degree of agreement between the magnetic and VES profiles.

4.3 Borehole Sections

The geologic section observed from six borehole cuttings is shown in figure 10. The cuttings were visually inspected in their natural state or condition. The geologic units observed from the sites investigated comprised sandy clay, clay, clayey sand, sand-clay mixture, fractured rock, and basement rock. The depth to the basement ranged from 19.5 m for borehole 04 (gneiss) – 49.0 m for borehole 06 (migmatite). The static water level ranged between 15.5 – 26.6 m. Consequently the SWL is deep in the study area. However, the upper 15 m composed of sandy clay, clay sand and clay, although clayey sand is the most preponderant. The structural features possibly fractures observed on the magnetic profile, and on geoelectric section agreed with the fractured zone delineated on the borehole sections. This zone is the main water bearing zone in the area, and the weathered layer.

4.4 Hydrogeological Study

The hydrogeological investigation enables the prediction about the influence of groundwater system in civil engineering works. This can be carried out to assess location and thickness of water zone, their confinement, and hydrogeological margins; the levels of water and their variations with seasons (time); their storage potential and transmissivity; and their quality. The data acquired from fifty across different rocks is presented in Table 2. The total depth of well investigated ranged from 5.6 – 13.5 m (avg. 8.9 m), even though the depth of the wells is at owner's discretion and availability of funds, but useful data were able to acquired. The water column which is storage/reservoir potential of the wells ranged from 1.5 – 7.5 m (avg. 4.50 m). The SWL varied from 2.5 m to 7.1 m (4.47 m), with corresponding hydraulic head of 240.5 – 277 m above the seal level (avg. 258.3 m). The information from the boreholes in Table 3, with total depth ranging from 35 (gneiss) – 51 m (granite) and an average of 44 m, showed SWL ranging from 18 – 26 m (avg. 22.2 m).

4.5 Geochemical Analysis

The stability and serviceability performance of soil for construction works is contingent upon the mineralogical make-up of the soil [30], [39]. The result of chemical analysis of selected mineral oxides contained in the soil samples, and silica-sesquioxide (S-S) ratio is presented in Table 4. They ranged from: MgO (0.19- 0.75 %, avg. 0.378), Al_2O_3 (15.16 – 24.5 %, avg. 18.45 %), SiO_2 (51.42 – 69.87 %, avg. 61.78), P_2O_5 (0 – 0.1 %, avg. 0.02 %), Na_2O (0.98 – 3.9 %, avg. 2.01 %), K_2O (0.23 – 4.52 %, avg. 2.45 %), CaO (0.82 – 0.27 %, avg. 0.07 %), TiO_2 (0.98 – 1.66 %, avg. 1.21 %), V_2O_5 (0.01 – 0.08 %, avg. 0.023 %), Cr_2O_3 (0 – 0.03 %, avg. 0.012 %), MnO (0.01 – 0.15 %, avg. 0.06 %), Fe_2O_3 (17.65 – 20.25 %, avg. 18.98 %), and CuO (0.01 – 0.03 %, avg. 0.02 %).

Consequently the soil is abundantly rich in SiO_2, Fe_2O_3, and Al_2O_3, with the concentration of SiO2 more than combined concentrations of other mineral oxides. This indicates that parent rock material in the study is silica-rich igneous rock, suggestive of granite, gneiss, rhyolite, dacite, granodiorite, diorite, andesite, quartz, and orthoclase porphyries. However, the geological observation showed granite, gneiss, migmatite gneiss dominated the environment. The S-S ratio varied between 1.39 – 1.97 (avg. 1.66). Accordingly, the soils' S-S ratio is within lateritic type range of 1.33 – 2.0 [56].

4.6 Geotechnical Analysis

Tables 4-6 present the summary of the geotechnical results. The natural moisture content varied from 9.3 to 16.2 % (avg. 12.93 %), this range is within the 5 – 15 % acceptable range favourable for civil engineering uses. Grain size analysis can be used to characterize the subsoil material for engineering works, which can serve as a guide to the engineering performance of the soil type and also provides a means by which soils can be identified quickly. The sand content ranged from 36.5 – 69.3 % (avg. 51.75 %), % silt and clay contents ranged from 6.3 to 25.0 % (avg. 14.36 %) and 14.4 to 53.9 % (avg. 33.85 %).

The %fines ranged from 30.6 to 63.5 (avg. 48.3). The composition of the soil is dominated (in order of magnitude) by sand, clay, and silt (SC-SM). The amount of %fines recorded is more than 35 % specification of Nigerian federal ministry of works and housing [57]. The plasticity chart (figure 11a) shows that the fines in the samples is dominated by clay of low plasticity/compressibility with LL less than 50 %. All the soil samples plotted above the A-line. In terms of clay mineralogy, the soil samples are plotted within the range of illite and illite/montmorillonite clay mineralogy group (figure 11b). Montmorillonite is made up of two silica sheets and one gibbsite sheet and bonded by Vander wall forces between the tops of silica sheets is weak and there's negative charge deficiency, water and exchangeable ions can enter and separate the layers. Hence montmorillonite has a very strong attraction for water and swells on absorption of water. Illite has a similar structure similar to montmorillonite, however in illite the interlayers are bonded together with a potassium ion linkage, making it to have relatively less attraction for water. The activity ranged from 0.35 to 0.79 (avg. 0.53) signifying inactive clay type.

Table 1. Interpreted VES Results

East	North	Elevation (m)	VES NO.	Resistivity (Ohmns-meter)					Thickness (m)				Depth (m)				Curve Type
				ρ_1	ρ_2	ρ_3	ρ_4	ρ_5	h_1	h_2	h_3	h_4	d_1	d_2	d_3	d_4	
700461	786330	249	1	222	36	559			1.2	16.5			1.2	17.7			H
700025	786507	249	2	438	59	989			1.1	13.3			0.9	14.4			H
700525	785830	248	3	25	323	98	1023		0.5	6.5	15.2		1.8	7	22.2		KH
699735	785087	249	4	65	256	96	991		0.9	3.4	10.4		1.1	4.3	14.7		KH
701736	785813	259	5	71	203	39	1362		1.2	9.1	12.8		1.1	10.3	23.1		KH
702381	786104	269	6	104	447	82	2025		1.1	3.6	9.7		1.9	4.7	14.4		KH
702446	786120	271	7	411	129	512	231	992	0.8	4.1	7.4	17.7	2.3	4.9	12.3	30	HKH
702914	786233	280	8	118	52	745			0.9	22.2			0.9	23.1			H
704673	786766	271	9	408	69	658			1.0	9.8			1.6	10.8			H
703398	786023	267	10	68	425	165	1471		1.2	8.2	15.5		2.5	9.4	24.9		KH
701090	784974	248	11	399	101	732	545		2.2	6.4	9.9		4.6	8.6	18.5		HK
700429	784393	251	12	305	89	447	113		1.1	4.8	12.4		4.9	5.9	18.3		HK
700203	784474	251	13	225	50	1656			0.8	18.2			1.2	19			H
700864	784184	254	14	142	18	2330			0.5	22.9			1.9	23.4			H
701155	783893	252	15	169	44	455	89	806	0.9	5.3	6.6	14.7	1.1	6.2	12.8	27.5	HKH
701510	784393	260	16	166	525	69	1121		0.8	7.8	16.6		1.2	8.6	25.2		KH
701591	783635	253	17	228	602	107	523	389	0.9	5.7	10.5	11.3	1.1	6.6	17.1	28.4	KHK
700187	782328	240	18	148	558	92	620	274	1.1	3.3	6.8	14.1	0.9	4.4	11.2	25.3	KHK
699815	782069	231	19	311	649	155	1222		1.2	5.6	17.9		1.6	6.8	24.7		KH
699493	782279	232	20	123	28	473			0.7	22.6			0.8	23.3			H
699170	782102	231	21	490	123	650			0.9	19.9			0.7	20.8			H
701558	782699	254	22	358	119	3650			1.5	20.2			0.8	21.7			H
701800	782489	247	23	145	99	52	696		2.1	4.4	12.3		1.2	6.5	18.8		QH
702301	781198	280	24	322	85	28	888		2.3	8.5	16.8		1.9	10.8	27.6		QH
701752	780714	256	25	176	88	2820			1.2	23.5			0.9	24.7			H
703334	780520	257	26	403	615	231	1999		1.2	6.6	18.2		1.9	7.8	26		KH
703963	780456	256	27	51	379	225	1500		0.9	3.3	21.7		1.5	4.2	25.9		KH
703430	781650	267	28	69	290	147	1203		1.3	7.2	24.1		1.1	8.5	32.6		KH
703237	782699	259	29	145	55	552			0.8	16.6			0.8	17.4			H
703253	782957	258	30	362	48	989			0.9	13.4			0.9	14.3			H
703075	783619	268	31	358	80	1208			0.9	22.8			0.9	23.7			H
703204	783812	273	32	154	42	909			1.4	14.4			1.4	15.8			H
703608	783974	278	33	89	522	252	1420		1.5	5.9	17.8		1.5	7.4	25.2		KH
703027	783893	273	34	149	86	441	101	778	0.9	2.4	6.6	15.1	0.9	3.3	9.9	25	HKH
703108	783925	273	35	154	525	237	1116		1.0	7.7	10.1		1.0	8.7	18.8		KH
703124	783958	274	36	236	72	573			1.2	18.6			1.2	19.8			H
703027	784087	273	37	458	98	1428			0.9	16.3			0.9	17.2			H
703108	784119	274	38	87	173	52	1330		1.6	3.4	18.8		1.6	5	23.8		KH
703156	784135	274	39	212	81	518	222	980	1.1	6.9	7.8	14.2	1.1	8	15.8	30	HKH
702285	784248	268	40	86	56	1901			2.4	16.7			2.4	19.1			H
702107	784071	264	41	109	362	38	655	221	0.8	2.6	7.6	23.2	0.8	3.4	11	34.2	KHK
704044	784797	280	42	147	625	332	689	554	1.2	4.5	9.3	20.1	1.2	5.7	15	35.1	KHK
703979	784845	278	43	120	55	989			1.4	14.7			1.4	16.1			H
702736	785120	273	44	111	90	1470			1.6	18.3			1.6	19.9			H
702736	785152	273	45	78	411	105	555	322	1.9	14.9	9.2	12.5	1.9	16.8	26	38.5	KHK
704689	786378	278	46	222	87	1121			2.2	20.8			2.2	23			H
704350	785878	269	47	92	369	108	497	358	0.9	4.6	7.1	16.4	0.9	5.5	12.6	29	KHK
699509	786895	255	48	58	258	100	902		0.9	5.3	16.4		0.9	6.2	22.6		KH
705496	786911	254	49	99	229	82	1018		1.2	6.3	18.5		1.2	7.5	26		KH
702397	783038	252	50	96	55	945			1.5	17.8			1.5	19.3			H

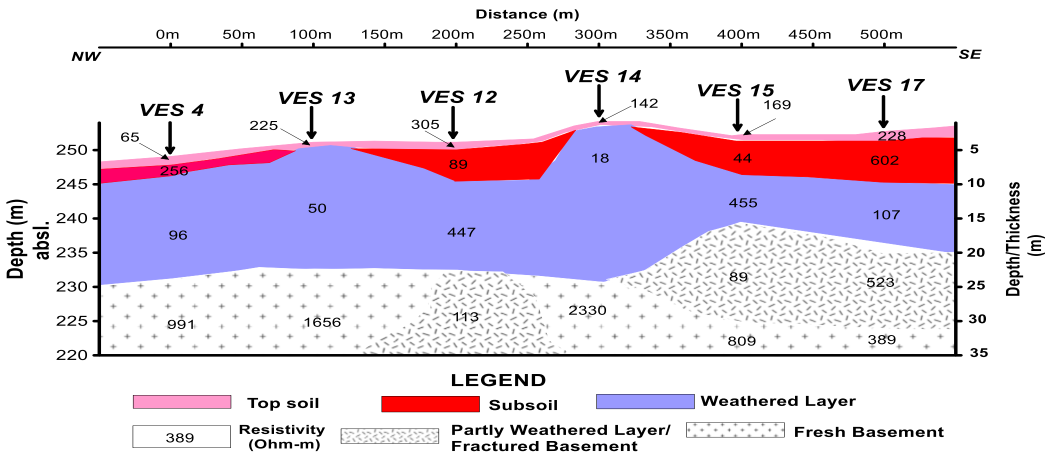

Fig. 8. Geologic Section/Profile along the selected VES point established in the study area

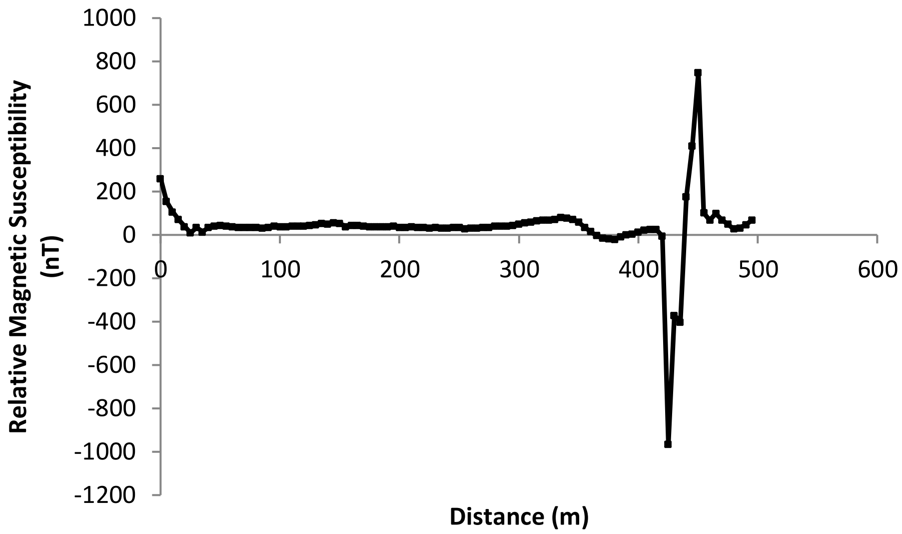

Fig. 9. Magnetic Profile along the selected VES points

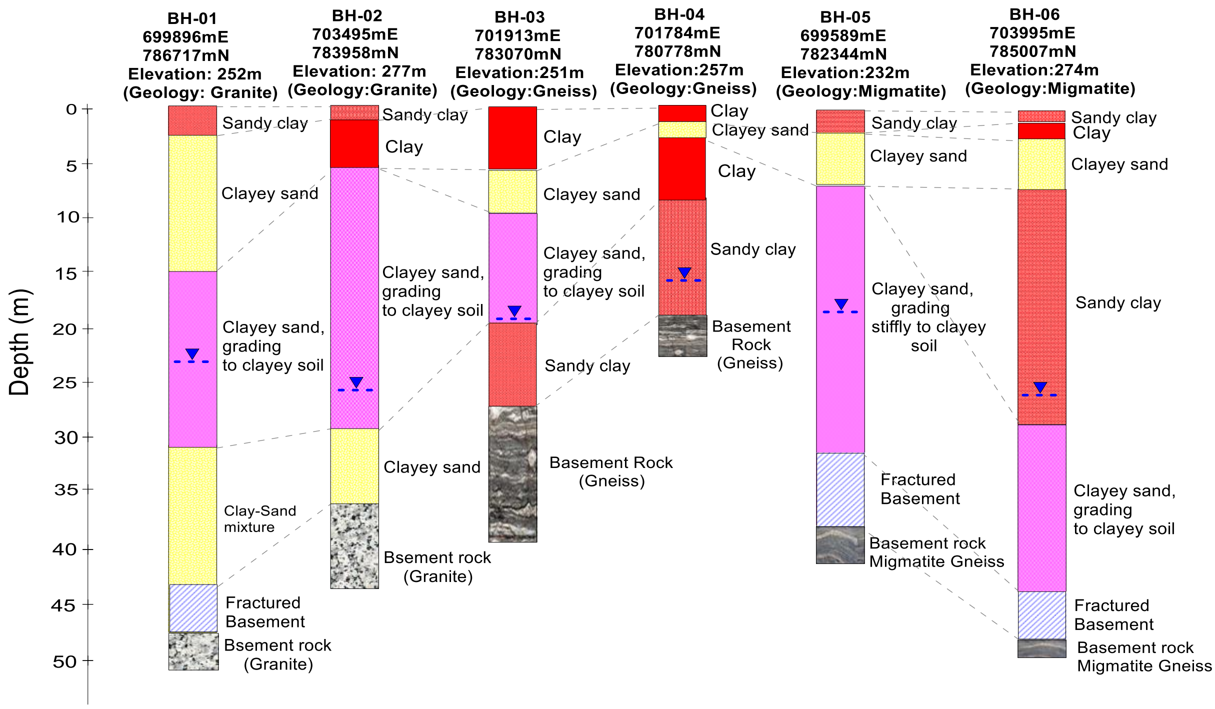

Fig. 10. Borehole sections showing the various geologic units observed from borehole cuttings

The specific gravity (SG) is closely related with soil's mineralogy and/or chemical contents; the higher SG, the higher the degree of laterization. In addition, the larger the clay fraction and alumina contents, the lower is the SG [58]. The values of specific gravity of the samples ranged between 2.65 – 2.69 (avg. 2.67). The standard range of value of specific gravity of soils lies between 2.60 and 2.80, these values are considered normal for construction works. The liquid limit (LL) values ranged between 31.1 to 49.1 % (avg. 40.6 %), plastic limits (PL) ranged between 16.8 to 39.0 % (avg. 23.9 %) and plasticity index (PI) is between 8.1 to 24.9 % (avg. 16.6 %). Soil with high LL, PL, and PI are usually characterized with low bearing pressure. The result indicated medium expansive soil with marginal degree of severity (IS: 1494). Hence the soils do not satisfied this requirements as subgrade material. The linear shrinkage ranged between 7.4 to 12.0 % (avg. 10 %), signifying a medium swelling potential. The group index (GI) values obtained ranged from 1 to 11 (avg. 6) corresponding to fair subgrade soil.

Table 2. Summary of the well information obtained from fifty wells across the wet and dry season

East	North	Well. No	Elevation (m)	Total Depth	SWL	Water Column (m)	Hydraulic Head (m)	Geology
703156	784329	W-1	273	7.6	3.5	4.1	269.5	Granite
703301	784361	W-2	274	8.5	3.3	5.2	270.7	Granite
703479	784409	W-3	276	8.0	4.2	3.8	271.8	Granite
703656	784490	W-4	280	7.1	5.6	1.5	274.4	Granite
703479	784216	W-5	278	5.6	2.6	3	275.4	Granite
703317	784184	W-6	275	7.2	2.9	4.3	272.1	Granite
703108	784135	W-7	274	6.4	4.0	2.4	270.0	Granite
703011	784054	W-8	273	7.8	3.5	4.3	269.5	Granite
703140	783828	W-9	273	7.0	4.1	2.9	268.9	Granite
703430	783893	W-10	275	6.5	3.3	3.2	271.7	Granite
703543	783780	W-11	275	7.8	3.6	4.2	271.4	Granite
703398	783716	W-12	272	8.9	5.2	3.7	266.8	Granite
702285	784248	W-13/VES40	266	9.7	4.5	5.2	261.5	Granite
702220	783990	W-14	264	10.5	5.1	5.4	258.9	Granite
702091	783877	W-15	261	8.7	3.3	5.4	257.7	Granite
701946	783941	W-16	260	11.3	6.5	4.8	253.5	Granite
702736	785152	W-17/VES45	258	9.5	3.6	5.9	254.4	Granite
701768	783796	W-18	257	10.5	4.2	6.3	252.8	Granite
702397	783038	W-19/VES50	253	9.5	3.9	5.6	249.1	Granite
702220	783264	W-20	254	6.4	3.6	2.8	250.4	Granite
704673	786766	W-21/VES9	255	7.5	4.1	3.4	250.9	Granite
702301	783457	W-22	256	12.3	5.5	6.8	250.5	Granite
702462	783570	W-23	259	12.0	6.2	5.8	252.8	Granite
701929	783248	W-24	252	9.5	4.5	5	247.5	Granite
701591	783635	W-25/VES17	260	9.9	3.9	6	256.1	Gneiss
703414	782989	W-26	261	8.7	6.1	2.6	254.9	Gneiss
703559	782989	W-27	263	9.8	5.1	4.7	257.9	Gneiss
702640	782086	W-28	261	6.8	3.8	3	257.2	Gneiss
703430	781650	W-29/VES28	261	10.7	4.5	6.2	256.5	Gneiss
700461	786330	W-30/VES1	257	8.8	3.6	5.2	253.4	Gneiss
702236	781973	W-31	258	9.6	4.7	4.9	253.3	Gneiss
702107	781892	W-32	258	5.6	2.5	3.1	255.5	Gneiss
700735	785701	W-33	248	13.5	6.0	7.5	242.0	Gneiss
700525	785830	W-34/VES3	248	12.6	5.8	6.8	242.2	Gneiss
700429	785701	W-35	247	10.8	4.9	5.9	242.1	Gneiss
700348	785684	W-36	247	13.1	6.5	6.6	240.5	Granite
700267	785652	W-37	247	11.5	5.8	5.7	241.2	Granite
700154	785394	W-38	248	10.6	6.2	4.1	241.8	Granite
700719	786491	W-39	252	12.3	6.9	5.4	245.1	Granite
700864	786637	W-40	254	9.7	7.1	2.6	246.9	Gneiss
700768	786750	W-41	254	10.2	6.8	3.4	247.2	Gneiss
700671	786830	W-42	254	11.8	5.7	6.1	248.3	Gneiss
702914	786233	W-43/VES8	281	5.8	4.0	1.8	277.0	Gneiss
703011	786701	W-44	274	6.2	3.2	3	270.8	Gneiss
702349	785668	W-45	269	8.8	4.1	4.7	264.9	Gneiss
703011	785297	W-46	265	8.0	5.0	3	260.0	Gneiss
702704	785216	W-47	273	6.2	3.2	3	269.8	Gneiss
703963	780456	W-48/VES27	256	9.5	4.3	5.2	251.7	Gneiss
703866	781150	W-49	266	10.2	5.4	4.8	260.6	Gneiss
704044	781133	W-50	265	8.7	3.5	5.2	261.5	Gneiss
704189	781133	W-51	264	9.5	3.9	5.6	260.1	Gneiss
703334	780520	W-52/VES26	266	7.8	4.2	3.6	261.8	Gneiss
702785	781295	W-53	274	8.5	2.5	6	271.5	Gneiss
701800	782489	W-54/VES23	264	5.6	2.9	2.7	261.1	Gneiss
703317	782295	W-55	262	6.9	2.7	4.2	259.3	Gneiss

Table 3. Borehole Information obtained from six boreholes

East	North	Borehole No.	Elevation (m)	Total Depth (m)	SWL (m)	Geology	Present State
699896	786717	BH-1	252	51	23	Granite	Functioning
703495	783958	BH-2	277	48	26	Granite	Functioning
701913	783070	BH-3	251	39	19	Gneiss	Functioning
701784	780778	BH-4	257	35	22	Gneiss	Functioning
699589	782344	BH-5	232	42	18	Granite	Functioning
703995	785007	BH-6	274	49	25	Granite	Functioning

Table 4. Result of the chemical analysis of selected mineral oxide

Sample No.	MgO	Al_2O_3	$Si0_2$	P_2O_5	Na_2O	k_2O	CaO	$Ti0_2$	V_2O_5	Cr_2O_3	MnO	Fe_2O_3	CuO	S-S Ratio	Class
OD-1	0.23	17.56	62.2	0.01	2.29	4.52	0.32	1.66	0.01	0.01	0.03	19.65	0.03	1.67	Lateritic
OD-2	0.33	19.98	63.5	0.01	3.25	3.4	0.22	1.45	0.01	0.01	0.03	18.23	0.01	1.66	Lateritic
OD-3	0.38	24.5	60.5	0.01	3.22	0.23	0.35	1.28	0.03	0.01	0.03	18.95	0.03	1.39	Lateritic
OD-4	0.65	18.38	59.88	0.01	1.02	0.56	0.82	1.25	0.02	0.01	0.03	18.66	0.01	1.62	Lateritic
OD-5	0.19	18.96	61.25	0.01	1.2	1.87	0.21	1.22	0.04	0.01	0.03	17.65	0.01	1.67	Lateritic
OD-6	0.42	17.25	58.95	0	0.98	3.05	0.25	1.32	0.03	0.02	0.05	18.27	0.02	1.66	Lateritic
OD-7	0.33	18.23	63.21	0	1.45	2.54	0.18	1.12	0.02	0.01	0.03	19.88	0.01	1.66	Lateritic
OD-8	0.23	17.22	69.8	0	3.25	2.32	0.24	1.04	0.01	0.01	0.01	18.24	0.01	1.97	Lateritic
OD-9	0.52	18.45	57.45	0	2.45	2.45	0.19	1.11	0.01	0.01	0.03	19.59	0.01	1.51	Lateritic
OD-10	0.47	15.66	60.2	0.01	3.1	3.36	0.22	1.15	0.02	0.01	0.12	19.22	0.03	1.73	Lateritic
OD-11	0.56	17.85	60.58	0	1.45	1.26	0.17	0.98	0.03	0.01	0.15	18.66	0.03	1.66	Lateritic
OD-12	0.31	17.65	65.87	0	3.9	3.65	0.21	1.02	0.01	0.01	0.03	17.73	0.01	1.86	Lateritic
OD-13	0.39	18.95	69.87	0	2.44	1.39	0.18	0.99	0.01	0.02	0.15	20.1	0.02	1.79	Lateritic
OD-14	0.75	17.7	63.23	0.01	1.02	1.45	0.63	1.24	0.08	0.03	0.03	19.46	0.01	1.70	Lateritic
OD-15	0.22	18.95	60.22	0.1	1.54	2.65	0.23	1.11	0.02	0.01	0.15	20.25	0.03	1.54	Lateritic
OD-16	0.42	19.2	64.1	0.1	1.2	2.59	0.21	1.09	0.02	0.01	0.03	18.63	0.01	1.69	Lateritic
OD-17	0.31	19.52	60.1	0.01	2.22	2.68	0.19	1.44	0.01	0.02	0.03	18.57	0.01	1.58	Lateritic
OD-18	0.31	17.74	63.32	0.01	1.65	2.53	0.07	1.23	0.03	0.01	0.13	19.25	0.01	1.71	Lateritic
OD-19	0.24	16.69	59.95	0.01	1.2	3.58	0.12	1.43	0.02	0	0.03	20.13	0.03	1.63	Lateritic
OD-20	0.29	18.5	51.42	0.01	1.26	2.9	0.47	1.08	0.03	0.01	0.15	18.45	0.02	1.39	Lateritic

The unit weight of the soils varied from 17.8 – 21.91 KN/m^3) (20.19 KN/m^3), cohesion of 29.6 – 64.5 KN/m^2 (avg. 56.6 KN/m^2), and angle of friction of 11.4 – 22.1° (avg. 13.1°). The unconfined compressive strength (UCS) ranged from 159.6 – 204.5 KN/m2 (avg. 186.8 KN/m^2). The hydraulic conductivity of the samples is between 2.68E-07 to 2.86E-04 cm/s (avg. 1.68E-05 cm/s) of poor drainage condition as per BIS. The maximum dry density (MDD) for the soil samples varied between 1809 and 2188 kg/m^3 (avg. 1997 kg/m^3) at standard proctor compaction energy while the optimum moisture content (OMC) ranged between 9.8 and 18.6 % (avg. 13.2 %). All the soil samples have moderate/moderately-high MDD at moderate OMC.

The California Bearing Ratio (CBR) is an empirical test employed in road engineering as an index of compacted material strength and rigidity, corresponding to a defined level of compaction. All compacted samples show soaked and un-soaked CBR values ranging between 3 and 11 % (avg. 7 %) and 51 – 82 % (avg. 68 %) respectively. The consolidation characteristics of the soils showed coefficient of consolidation-C_v (0.001102 – 0.01198 m^2/yr; avg. 0.010342 m^2/yr), coefficient of compressibility-a_v (0.16559 – 0.34187 MPa^{-1}; avg. 0.23990 MPa^{-1}), coefficient of volume compressibility-M_v (0.15231 – 0.2311 m^2/KN; avg. 0.194297 m^2/KN), compression index-C_C (0.0212 – 0.0465; avg. 0.0465), swelling index-C_S (-0.00569 to -0.00303; avg. -0.00461), recompression index-C_r (0.008 – 0.035; avg. 0.0216) and void ratio-e_o (0.20 – 0.34; avg. 0.2320).

The preconsolidation pressure applied was 0.040MPa. Consequently, using the averages of all the consolidation parameters, based on C_c the soils are sandy clay with low compressibility i.e. C_c less than 0.075; based on M_v the soils are likely to exhibit medium degree of compressibility typical of varved and laminated clays or firm to stiff clays (0.25 – 0.125 m^2/KN). The coefficient of consolidation is the indicative of the combined effect of compressibility and permeability of soil on the rate of volume change.

4.7 CPT Analysis

The results of the CPT is presented in Table 8, while the plotted sounding curves for the eight locations is shown in figure 12 showing the cone resistance (Q_c), sleeve resistance (S_r), friction ratio (F_R), allowable bearing capacity, and Modulus Number (M-number) with depth. The obtained values of Q_c ranged from 9 – 140 kg/cm^2 (avg. 69 kg/cm^2), S_r varied from 21 – 458 kg/cm^2 (avg. 188 kg/cm^2), Q_{cn} is between 4 - 364 kg/cm^2 (avg. 150 kg/cm^2), F_R ranged from 1.60 – 3.99 (avg. 2.65), Q_{all} varied from 20.58 – 586.53 KN/m^2 (avg. 164.45 KN/m^2), UCS is in between 3.59 – 87.1 KN/m^2 (avg. 28.33 KN/m^2), C_u ranged from 1.79 – 43.55 KN/m^2 (avg. 14.17 KN/m^2), M-number varied from 5 – 74 9 (avg. 35), E_{square} is between 154.35 – 2229.5 KN/m^2 (avg. 1041.8 KN/m^2), E_{strip} ranged from 203 - 3121 KN/m^2 (avg. 1398 KN/m^2), N_{cor} varied from 2-35 (avg. 18), and σ_o is between 4.38 – 44.15 KN/m^2 (avg. 17.69 KN/m^2). The allowable bearing pressure for strip (Q_{strip}) and square (Q_{square}) ranged from 296 – 3395 KN/m^2 (avg. 1640 KN/m^2), and 443 - 4206 KN/m^2 (avg. 2034 KN/m^2) respectively.

The geologic units showed for CPT 1 (0 - 0.5 m - sandy silt to clayey silt; 0.5 – 0.75 m – silty sand to sandy silt; 0.75 – 1.0 m - sandy silt to clayey silt; 1.0 – 1.25 m - sand to clayey sand (cemented); CPT 2 (0 – 0.25 m – sand to clayey sand cemented; 0.25 – 1.0 m – sandy silt to clayey silt); CPT 3 (0 – 0.25 m – silty clay to clay; 0.25 – 0.5 m – sandy silt to clayey silt; 0.5 – 0.75 m – silty sand to sandy silt; 0.75 – 1.0 m – sandy silt to clayey silt; 1.0 – 1.25 m – sand to clayey sand (cemented); CPT 4 (0 - 0.5 m - sandy silt to clayey silt; 0.5 – 0.75 m - clayey silt to silty clay; 0.75 – 1.0 m - sandy silt to clayey silt; 1.0 – 1.25 m - silty sand to sandy silt; 1.25 – 1.50 m - sandy silt to clayey silt; 1.5 – 1.75 m - silty sand to sandy silt); CPT 5 (0 – 0.25 m – clayey silt to silty clay; 0.25 – 0.5 m – sandy silt to clayey silt; 0.5 – 0.75 m – silty sand to sandy silt; 0.75 – 1.0 m – sandy silt to clayey silt; 1.0 – 1.25 m – silty sand to sandy silt); CPT 6 (0 – 0.25 m – sandy silt to clayey silt; 0.25 – 0.5 m – sandy silt to clayey silt; 0.5 – 1.0 m – clayey silt to silty clay; 1.0 – 1.3 m – sandy silt to clayey silt; 1.3 – 1.5 m – silty sand to sandy silt).

Table 5. Summary of Geotechnical Analysis showing the particle size distribution, Consistency limit and soil classification

Sample No.	Location		Elev. (m)	NMC (%)	Grain size Distribution				SG	Consistency Limits			SL (m)	Group Index	AASHTO Class	USCS Class
	Easting (m)	Northing (m)			% Sand	% silt	% Clay	% Fines		PL (%)	LL (%)	PI (%)				
OD-1	703914	780601	259	9.3	58.9	19.3	21.8	41.1	2.69	21.1	35.3	14.2	7.4	2	A-6	SC-SM
OD-2	702591	781440	274	14.5	52.1	16.4	31.5	47.9	2.65	22.3	47.2	24.9	9.1	8	A-7-6	SC-SM
OD-3	703140	781666	268	13.1	68.3	14.7	17.0	31.7	2.68	19.4	32.4	13.1	11.5	1	A-2-6	SC-SM
OD-4	702139	782554	251	12.3	68.0	17.6	14.4	32.0	2.67	24.2	32.3	8.10	12.0	1	A-6	SC-SM
OD-5	702059	783183	252	12.4	69.3	15.5	15.1	30.6	2.68	22.2	31.1	8.95	12.0	1	A-2-4	SC-SM
OD-6	701139	783312	242	13.3	58.2	15.5	26.3	41.8	2.67	24.2	36.5	12.3	9.6	2	A-6	SC-SM
OD-7	703414	783699	272	16.2	42.0	11.2	46.8	58.0	2.66	23.8	43.3	19.5	11.9	9	A-7-6	SC-SM
OD-8	704092	784748	282	15.5	48.3	25.0	25.7	51.7	2.67	28.0	39.2	11	8.5	4	A-6	SC-SM
OD-9	702753	784087	270	10.2	63.5	18.4	18.2	36.5	2.67	39.0	48.5	9.5	7.6	1	A-5	SC-SM
OD-10	702042	784184	264	14.3	49.1	22.4	28.5	50.9	2.67	29.4	49.1	19.7	10.2	7	A-7-5	SC-SM
OD-11	701397	784232	258	13.9	51.8	8.5	39.7	48.2	2.67	18.5	42.6	24.1	11.3	7	A-7-6	SC-SM
OD-12	700977	785297	244	14.0	44.2	6.9	48.9	55.8	2.66	30.1	47.4	17.3	9.5	8	A-7-6	SC-SM
OD-13	702397	785055	277	11.6	47.8	13.3	38.9	52.2	2.67	24.3	38.9	14.6	8.3	5	A-6	SC-SM
OD-14	702543	785749	271	9.7	56.9	16.1	27.0	43.1	2.67	19.4	37.8	18.4	10.4	4	A-6	SC-SM
OD-15	701026	786201	252	12.4	44.3	14.8	40.9	55.7	2.68	19.2	40.2	20.8	11.5	9	A-7-6	SC-SM
OD-16	700671	786750	254	11.0	48.7	12.6	38.7	51.3	2.67	25.7	48.6	22.9	10.9	9	A-7-6	SC-SM
OD-17	699299	787088	256	14.8	36.5	11.7	51.8	63.5	2.66	22.2	42.7	20.5	8.8	11	A-7-6	SC-SM
OD-18	702736	786281	277	12.1	40.6	12.2	47.2	59.4	2.67	16.8	33.2	16.4	9.7	7	A-6	SC-SM
OD-19	704867	786281	273	13.7	39.8	6.3	53.9	60.2	2.67	25.7	45.5	19.8	9.9	10	A-7-6	SC-SM
OD-20	705028	787056	261	14.2	46.6	8.7	44.7	53.4	2.67	23.3	39.1	15.8	10.2	6	A-6	SC-SM

Table 6. Summary of Geotechnical Analysis showing the grading curve properties, CBR, cohesion, and consolidation parameters

Sample No.	Unit Weight (KN/m^3)	Triaxial Test		UCS (KPa)	K (cm/s)	Clay Mineralogy	Activity	
		Cohesion (KN/m^2)	Angle of friction (°)				Values	Soil Type
OD-1	17.80	29.6	22.1	159.6	2.86E-04	I-M	0.65	Inactive
OD-2	19.80	61.8	11.9	179.8	8.54E-07	I-M	0.79	Normal
OD-3	21.91	52.0	13.1	161.9	5.66E-06	I	0.77	Normal
OD-4	21.27	62.2	13.0	187.2	5.36E-06	I	0.56	Inactive
OD-5	21.19	54.4	13.0	167.5	6.79E-06	I	0.59	Inactive
OD-6	20.93	63.0	14.7	200.2	1.59E-06	I	0.47	Inactive
OD-7	20.65	64.5	13.9	204.5	3.66E-07	H	0.42	Inactive
OD-8	21.25	59.2	11.5	198.9	2.68E-07	I	0.44	Inactive
OD-9	19.50	55.8	11.8	187.3	4.98E-06	M	0.52	Inactive
OD-10	21.75	61.4	12.6	202.6	6.11E-07	I	0.69	Inactive
OD-11	19.85	61.2	13.6	185.3	4.28E-06	I	0.61	Inactive
OD-12	19.65	54.7	11.9	189.6	2.98E-07	M-I	0.35	Inactive
OD-13	20.20	52.3	12.6	195.7	3.45E-07	M-I	0.38	Inactive
OD-14	21.24	58.1	11.4	198.8	6.76E-07	M-I	0.68	Inactive
OD-15	19.56	55.6	12.6	179.4	5.22E-06	M-I	0.51	Inactive
OD-16	18.85	50.0	12.4	181.4	3.67E-06	I-M	0.59	Inactive
OD-17	19.62	56.5	13.3	183.1	2.10E-06	I	0.40	Inactive
OD-18	18.88	60.7	13.5	180.6	2.38E-06	I	0.35	Inactive
OD-19	19.45	55.9	12.1	190.2	3.64E-06	I	0.37	Inactive
OD-20	20.45	62.3	11.7	201.5	4.44E-07	I	0.35	Inactive

Table 7. Compaction characteristics, CBR, and Consolidation tests conducted on the soil samples

Sample No.	MDD Kg/m^3	OMC	CBR_S	CBR_U	C_v (m^2/yr)	a_v MPa^{-1}	m_v MPa^{-1}	σ_p MPa	C_c Index	C_s	C_r	e_o
OD-1	2188	10.5	11	81	0.01192	0.18324	0.15655	0.0400	0.0244	-0.00501	0.018	0.34
OD-2	1961	18.6	6	69	0.01093	0.29383	0.22927	0.0400	0.0387	-0.00321	0.028	0.27
OD-3	2096	13.6	9	78	0.01143	0.23458	0.19164	0.0400	0.0310	-0.00419	0.023	0.21
OD-4	2136	13.1	5	80	0.01026	0.23231	0.19268	0.0400	0.0307	-0.00410	0.022	0.20
OD-5	2115	12.8	10	82	0.01019	0.23501	0.19580	0.0400	0.0311	-0.00399	0.023	0.21
OD-6	2075	14.4	8	68	0.00949	0.27799	0.22927	0.0400	0.0399	-0.00303	0.027	0.23
OD-7	2155	10.2	6	55	0.00848	0.29875	0.22990	0.0400	0.0347	-0.00333	0.030	0.21
OD-8	1856	18.2	6	69	0.01195	0.17255	0.15231	0.0400	0.0214	-0.00555	0.015	0.22
OD-9	1905	15.2	11	77	0.00777	0.31202	0.23110	0.0400	0.0411	-0.00465	0.029	0.22
OD-10	2005	9.8	5	62	0.01134	0.19820	0.16517	0.0400	0.0212	-0.00510	0.009	0.20
OD-11	1998	10.9	7	66	0.01122	0.17632	0.17374	0.0400	0.0244	-0.00569	0.012	0.21
OD-12	1968	15.6	4	60	0.01198	0.18887	0.15989	0.0400	0.0289	-0.00554	0.008	0.24
OD-13	2050	9.8	4	56	0.01156	0.21001	0.15622	0.0400	0.0256	-0.00500	0.012	0.23
OD-14	1965	14.8	9	77	0.01142	0.16559	0.16213	0.0400	0.0320	-0.00522	0.020	0.25
OD-15	1865	14.5	8	69	0.01081	0.24231	0.22996	0.0400	0.0465	-0.00487	0.027	0.23
OD-16	1809	13.3	6	54	0.01145	0.24566	0.21267	0.0400	0.0329	-0.00411	0.030	0.21
OD-17	1888	14.8	6	75	0.01099	0.34187	0.21553	0.0400	0.0374	-0.00436	0.035	0.26
OD-18	1920	11.4	8	60	0.01033	0.29996	0.21989	0.0400	0.0369	-0.00498	0.028	0.25
OD-19	1870	12.6	3	74	0.00142	0.31121	0.22888	0.0400	0.0315	-0.00488	0.028	0.22
OD-20	2115	10.4	5	51	0.01189	0.17786	0.15333	0.0400	0.0369	-0.00546	0.008	0.23

From the CPT 7 signatures, the soil layering consists of silty clay to clay (0 – 0.25 m), sandy silt to clayey silt (0.25 – 0.5 m), silty sand to sandy silt (0.5 – 0.9 m), sandy silt to clayey silt (0.9 – 1.3 m), and sand to clayey sand (cemented) (1.3 – 1.8 m). CPT 8 is made of the following sequence, sandy silt to clayey silt (0 – 0.6 m), clayey silt to silty clay (0.6 – 0.9 m), sandy silt to clayey silt (0.9 – 1.2 m), silty sand to sandy silt (1.2 – 1.5 m), sandy silt to clayey silt (1.5 – 1.8 m), and silty sand to sandy silt (1.8 – 2.25 m). Since the upper 1.0 m is usually regarded as weak soil zone for most civil engineering construction, the depths of 0.5 m and below composed of sandy silt to clayey silt, and silty sand to sandy silt. This agreed with the result of the VES, borehole sections, and grain size distribution, which identified the topsoil/subsoil as sandy clay/clay sand and silty sand. The average Q_c (69 kg/cm2), Q_{all} of 164.5 KN/m2 obtained can support light/medium weight foundation structure without excessive settlement.

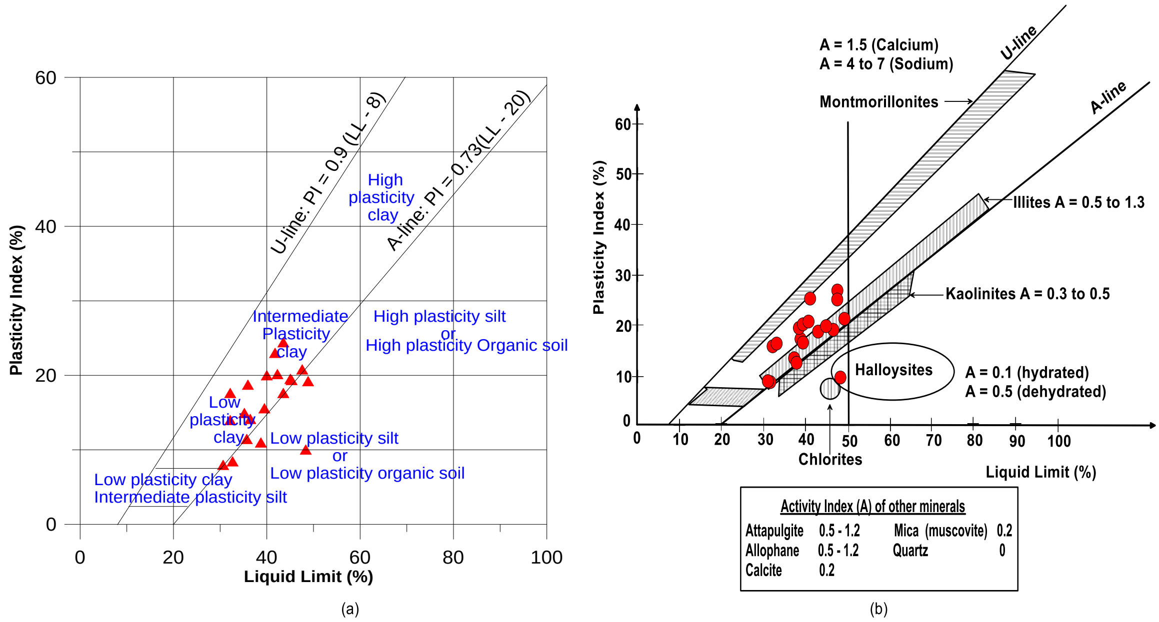

Fig. 11. (a) Plasticity Chart for Fine Contents of the soil samples (b) Clay mineralogy group of the soil samples with most within Illite – Illite/montmorillonite

The refusal depths for the survey varied between 1 - 1.25 m, and are usually terminated in silty sand to sandy silt and sandy to clayey sand (cemented). Using the values of CU the soil, the consistency of the soils is in between soft to firm. From the graph, the Q_C, M-Number, and Q_{all} increase with depth.

4.8 Geotechnical Parameters modelling and correlations

The obtained graphs and empirical models/equations for the parameters correlated are shown in figure 13. The obtained MDD/PI was correlated with soaked CBR determined from the laboratory and gives weak positive correlation (R^2) of 0.1113 and linear regression model (equation 12):

$$\text{CBR (soaked)} = 0.0148x + 4.8592 \tag{12}$$

In this relationship, x = MDD/PI

The LL was plotted against coefficient of consolidation. This gives a regression model of equation 13, with weakly positive correlations (R^2) of 0.0018.

$$\text{Coefficient of consolidation} = -8E\text{-}06x + 0.0112 \tag{13}$$

In these relationships, x = LL

The relationship between PI and undrained shear strength/effective overburden, is shown by the regression model in equation 14, with R^2 of 0.0332.

$$\frac{undrained\ shear\ strength}{effective\ overburden} = 0.0132x + 4.8789 \tag{14}$$

Where x is PI.

The correlation between dry density and angle of shearing gives equation 15, with correlation coefficient of 0.2131.

$$\text{Angle of shearing} = -0.9755x + 32.831 \tag{15}$$

Where x is dry density

The plot of PI and angle of shearing, gives correlation coefficient of 0.013, and the model is presented in equation 16.

$$\text{Angle of shearing} = -0.0515x + 13.99 \tag{16}$$

Where x is PI.

The relationship between suitability index and soaked CBR, gives a weak positive correlation of 0.3494, and the regression model shown in equation 17.

$$\text{CBR (soaked)} = 4.9704x – 8.2437 \tag{17}$$

Where x is suitability index.

In addition, the obtained clay content was correlated with PI and gives weak positive correlation (R^2) of 0.422 and linear regression model (equation 18).

$$PI = 0.2542x + 7.9865 \tag{18}$$

Where x is clay content.

5.0 Implication for Civil Engineering Construction

5.1 Pavement and Airfield

The engineering properties of soil desired for foundation under highway and airfield should have adequate strength, good compaction, adequate drainage, and acceptable compression and expansion properties [59]. The design of flexible pavement is normally based on Group Index method or California Bearing Ratio method [60], [61]. The drainage characteristics of the soil is poor with soaked CBR generally less than 10. . The AASHTO [62] classification of the soil for subgrade is good – poor; and USCS as fair - poor (Table 9).

Table 8. Results of the CPT and other estimated soil properties using the resistance values

Depth (m)	Q_c	S_r	Q_{cn}	F_R	Q_{all}	UCS	Cu	M-no.	E_{sq}	E_{strip}	N_{Cor}	σ_o	Qa Strip	Qa Square
CPT-1: 703027mE; 781440mN; 270m absl														
0.25	15	31	42	2.08	92.61	13.67	6.84	8	330	203	4	4.38	449	630
0.5	35	78	98	2.22	216.09	31.98	15.99	20	552	385	9	8.75	962	1252
0.75	70	147	196	2.10	432.18	64.17	32.09	40	941	706	18	13.13	1858	2340
1.0	95	363	266	3.82	586.53	87.10	43.55	54	1218	934	24	17.50	2498	3118
1.25	120	440	240	3.67	529.20	78.27	39.13	48	1496	1163	30	22.25	2261	2829
CPT-2: 70343mE; 782602mN; 261m absl														
0.25	10	21	28	2.14	22.87	4.01	2.01	6	171.50	240	3	4.38	321	474
0.5	50	162	140	3.23	114.33	20.89	10.45	28	857.50	1201	13	8.75	1346	1718
0.75	75	263	210	3.50	171.50	31.34	15.67	42	1286.25	1801	19	13.13	1986	2496
1.0	110	439	308	3.99	251.53	46.05	23.03	62	1886.50	2641	28	17.80	2883	3584
CPT-3: 700090mE; 782215mN; 237m absl														
0.25	12	42	34	3.15	27.44	4.87	2.44	7	205.80	288	3	4.38	373	537
0.5	40	90	112	2.25	91.47	16.60	8.30	23	686.00	960	10	8.75	1090	1407
0.75	78	168	218	2.16	178.36	32.62	16.31	44	1337.70	1873	20	13.13	2063	2589
1.0	98	382	274	3.90	224.09	40.91	20.45	55	1680.70	2353	25	17.80	2575	3211
1.25	122	458	215	3.75	175.35	31.49	15.74	43	1315.16	1841	31	22.25	2029	537
CPT-4: 700735mE; 784232mN; 253m absl														
0.25	18	36	50	2.01	41.16	7.44	3.72	10	308.70	432	5	4.38	526	723
0.5	45	99	126	2.20	102.90	18.75	9.37	25	771.75	1080	11	8.75	1218	1563
0.75	65	236	182	3.63	148.63	27.05	13.52	37	1114.75	1561	16	13.13	1730	2185
1.0	78	193	218	2.48	178.36	32.35	16.17	44	1337.70	1873	20	17.50	2063	2589
1.25	85	184	238	2.17	194.37	35.07	17.53	48	1457.75	2041	21	22.00	2242	2807
1.50	98	381	274	3.89	224.09	40.36	20.18	55	1680.70	2353	25	27.75	2575	3211
1.75	130	332	364	2.55	297.27	53.79	26.90	74	2229.50	3121	33	34.74	3395	4206
CPT-5: 701865mE; 785458mN; 261m absl														
0.25	10	24	28	2.36	22.87	4.01	2.01	6	171.50	240	3	4.38	321	474
0.5	38	85	106	2.23	86.89	15.75	7.87	21	651.70	912	10	8.75	1039	1345
0.75	70	150	196	2.14	160.07	29.17	14.58	40	1200.50	1681	18	13.50	1858	2340
1.0	85	252	238	2.97	194.37	35.23	17.61	48	1457.75	2041	21	19.50	2242	2807
1.25	115	270	322	2.35	262.97	47.71	23.85	65	1972.25	2761	29	25.56	3011	3740
CPT-6: 699380mE; 785523mN; 250m absl														
0.25	20	40	56	2.02	45.73	8.30	4.15	11	343.00	480.20	5	4.38	578	785
0.5	55	118	154	2.15	125.77	23.03	11.52	31	943.25	1320.55	14	8.75	1474	1874
0.75	68	233	190	3.42	155.49	28.31	14.16	38	1166.20	1632.68	17	13.50	1807	2278
1.0	80	281	224	3.51	182.93	33.08	16.54	45	1372.00	1920.80	20	19.50	2114	2651
1.25	95	264	238	2.78	193.96	34.82	17.41	48	1454.69	2036.56	24	24.75	2238	2801
1.50	128	242	320	1.89	261.33	47.14	23.57	65	1960.00	2744	32	29.78	2992	3717
CPT-7: 704883mE; 786249mN; 272m absl														
0.25	9	21	25	2.38	20.58	3.59	1.79	5	154.35	216.09	2	4.38	296	443
0.5	22	46	62	2.08	50.31	8.89	4.44	12	377.30	528.22	6	8.75	629	848
0.75	35	79	98	2.25	80.03	14.16	7.08	20	600.25	840.35	9	13.50	962	1252
1.0	58	186	162	3.21	132.63	23.65	11.82	33	994.70	1392.58	15	19.50	1551	1967
1.25	76	236	190	3.10	155.17	27.55	13.77	38	1163.75	1629.25	19	24.75	1803	2274
1.50	98	389	245	3.97	200.08	35.65	17.83	49	1500.63	2101	25	29.78	2306	2884
1.75	110	219	193	1.99	157.21	27.24	13.62	39	1179.06	1651	28	35.79	1826	2301
2.0	135	216	238	1.60	194.04	33.83	16.91	48	1455.30	2037.42	34	40.90	2239	2802
CPT-8: 399299mE; 787330mN; 257m absl														
0.25	10	24	4.38	2.41	22.87	4.01	2.01	6	171.50	240.10	3	4.38	321	474
0.5	18	42	8.75	2.36	41.16	7.17	3.59	10	308.70	432.18	5	8.75	526	723
0.75	32	74	13.50	2.30	73.17	12.88	6.44	18	548.80	768.32	8	13.50	885	1158
1.0	48	108	18.50	2.24	109.76	19.42	9.71	27	823.20	1152.48	12	18.50	1295	1656
1.25	68	158	23.13	2.33	138.83	24.59	12.29	34	1041.25	1457.75	17	23.13	1620	2051
1.50	89	280	27.75	3.15	181.71	32.34	16.17	45	1362.81	1908	22	27.75	2100	2635
1.75	99	208	33.25	2.10	141.49	24.45	12.23	35	1061.16	1486	25	33.25	1650	2088
2.0	115	206	39.24	1.79	165.29	28.54	14.27	41	1239.70	1735.58	29	39.24	1917	2411
2.25	140	232	44.15	1.66	201.23	34.97	17.49	50	1509.20	2112.88	35	44.15	2319	2900

From the result of the study, the GI ranged from 1-11 (avg. 6) corresponding to fair subgrade for highway construction, with expected recommended minimum -thickness of 79 – 140 mm (avg. 109 mm) obtained from design curves (Table 9). The average soaked CBR of the soils is 7% which fell below 10% recommended standard for subgrade, base or subbase. Thus, the soil is unsuitable for base and sub-base courses, while the subgrade value of the soil, generally ranges from excellent fair - poor (with low compressibility and expansion) especially when not subjected to frost action [63], [64].

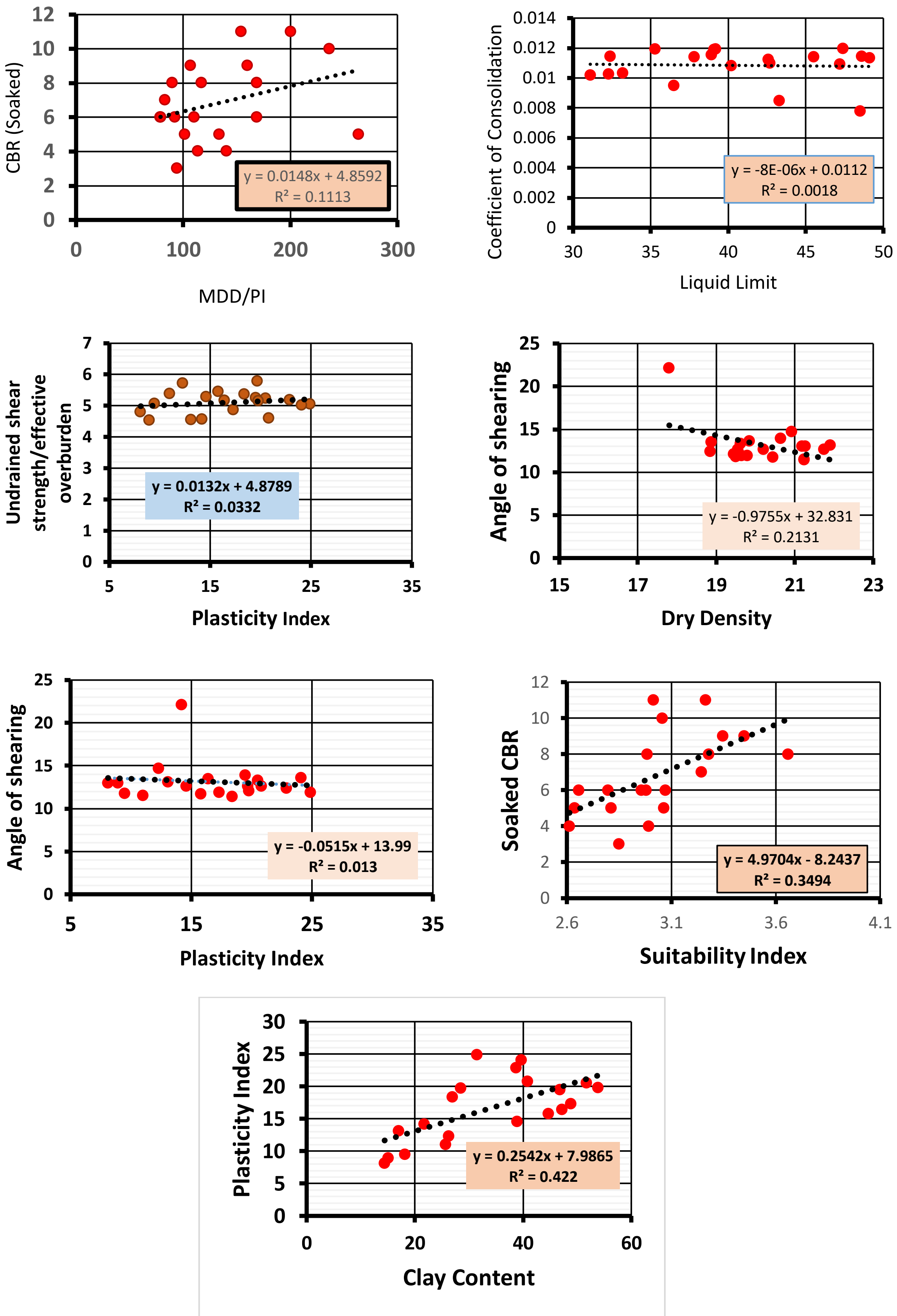

Fig. 13. Geotechnical parameters correlation for some of the engineering properties of the soils

Consequently, an inexpensive/economic mechanical stabilization or soil gradation and compaction will help in improving the bearing capacity and drainage characteristics.

5.2 Building Foundation

The average allowable bearing capacity of the soil for square and round foundations varied from 234 – 297 KN/m^2 (avg. 268 KN/m^2) and 232 – 298 KN/m^2 (avg. 268 KN/m^2). The estimated immediate/elastic settlement ranged from 6.76 – 7.66 mm (avg. 7.13 mm); and consolidation settlement varied between 10.9 – 11.62 mm (avg. 11.2 mm). The total settlement obtained is in between 17.69 – 18.88 mm (avg. 18.28 mm) for structural pressure of 100 KN/m^2. These results showed that the soils exhibit more of consolidation settlement than elastic, which implies that the soils behave more of fine soil material, even though the grain size analysis recorded average of 51.75 % for sand and 48.25 % for fines.Accordingly, the fines tend to control the geotechnical property of the soils in the study area, since it was established that soil with more than 35 % fines, tends to show some degree of plasticity and cohesion. From the CPT result, the average allowable pressure was estimated to be 371 KN/m^2 for average depth of 1.0 m. These bearing pressures are fair and would only be suitable for light/medium weight structures, with adequate factor of safety. The bearing pressures (using Hatanaka and Uchida [48], Mayne [65], Schmertmann [50] and Meyerhof [49] equations) gave bearing capacity models [66] with respect to foundation width [67] as shown in figure 14. The deformation criterion was calculated using Burland and Burbridge [51] equation. The applied factor of safety is 3.0, for maximum allowable settlement of 25.0 mm.

However proper soil improvement methods must be adopted (since clay/plastic silt tends to undergo volume change when desiccated), to ensure that the settlement is reduced in relation to the bearing pressure, although the soil are characterized by low compression index (avg. 0.03236) and coefficient of volume compressibility (avg. 0.194297 m^2/KN). Summarily the estimated settlement is within the standard 25 mm for building foundations of 100 KN/m^2.

5.3 Embankment

For satisfactory performance of an embankment material, the soils should have high stability and strength and well graded; coarse grained (such as sand, gravel) is usually preferable to fine soil. The suitability index of the soils ranged from 0.37 – 1.73 (avg. 1.02). The USCS classification of the soil is SC-SM which depicts soils of fair stability; that can be used for impervious core for flood control structures. The suitability index of the soil suggests a fair/expanding not collapsible construction material, as shown also in figure 15 having low-medium swelling potential. The compaction characteristics of the soil is fair. SC – SM soils have slight to medium compressibility and expansion, while the drainage characteristics is poor to practically impervious. Thus, since the soils have high MDD at moderate OMC (avg. 1997 kg/m^3; 13.2 %) greater than 1500 kg/m3, they are ordinarily considered suitable. The American Association of State Highway and Transportation Official [62] classification of the fines in the samples as A-6/A-7-6. A-6 soil are typical of plastic clay having a high percentage passing 0.075 mm and usually characterized with high volumetric change between wet and dry states. A-7-6 materials have high plasticity indices in relation to the liquid limits and are subject to extremely high volume change. A-7-5 materials have moderate plasticity indices in relation the liquid limits and may be highly elastic as well as subject to volume change. Therefore the soils with A-7-6/A-6 fines can be placed at the bottom of embankment and to remain in the top 0.5 m below subgrade in highway construction.Therefore comparing the important soils parameters such as plasticity, compressibility, strength (shear), workability, and compaction characteristics, the soils are rated according their utility for dams, canals, foundations, and highway. The relative score given to the soil is in the order of desirability from 1 to 14 i.e. high to low respectively. The findings from this study also confirmed some earlier suggestions to the effect that the coarser the material, the greater generally is its strength and the finer the material, the worse are its engineering properties. Thus, from the Table 11, the soil are generally fair.

5.4 Rock Properties

The rocks mapped in the study area are granite, granite gneiss, migmatite gneiss, and gneisses (figure 16). These rocks are usually characterized by high crushing strength and thus can be trusted in most construction works, especially as foundation and road stones. Igneous rocks are impervious, hard and strong and form very strong foundation for most civil engineering projects such as dams, reservoirs; because of their low porosity. The granitic rocks are rich in quartz, feldspar, and accessory mica (muscovite, biotite), amphiboles (hornblende, augite, hyperstene, magnetic, apatite, garnet, and tourmaline. Their texture ranged from medium to coarse grained, while some are porphyritic. The gneisses are megascopically crystalline foliated metamorphic rocks. They are characterized with mineral segregation into layers or bands of contrasting colour, texture and composition. The gneisses show bands of micaceous minerals alternating with bands of equidimensional minerals like feldspar, quartz. The migmatite are mixed rocks that consist of intimately associated members of igneous rock (granitic rock) and metamorphic (gneisses) groups. They are widespread in the study area.

The compressive strength of a rock depends on a number of factors such as mode of formation, composition, texture, structure, moisture content, and extent of weathering. According to Hunt [69] igneous rock have been crystalline in character, compact, and interlocking in texture and uniform in structure, and possess very high compressive/shear strength, modulus of elasticity [70], [71]. However, for metamorphic rocks the foliation, schistocity, and cleavage greatly affect their compressive strength in magnitude and direction [30]. Table 11 showed that the residual soils of most granites, gneisses and migmatite are low activity clays and granular soil, which is in agreement with earlier results, while Table 12 showed the expected properties of rocks observed in the study area, as they are expected to have very high strength, low deformability; and presumable bearing capacity of 8, 000 – 10, 000 KPa (Table 13) especially when fresh (FR), and can in between 5000 – 7000 KPa when partly or slightly weathered (SW). Falowo [72] conducted geotechnical analysis of some rocks (porphyritic granite, fine grained granite, migmatite, granite gneiss, quartz schist, granodiorite, charnockite, and quartzite) within the same geological province, for aggregate impact value, aggregate crushed value, point load strength test, specific gravity, water absorption and unconfined compression test, and direct shear strength using BS, ASTM D-2216 and ISRM procedures.

Table 9. The Highway and Foundation Characteristics of the soil with expected settlements

Sample No.	Subgrade Rating		GI Class	Rec Thickness (mm)	Suitability Index	Bearing Capacity (KN/m^2) Square Footing		Bearing Capacity (KN/m^2) Round Footing		Settlement (mm)		
	USCS	AASHTO Class				Q_T	Q_A	Q_T	Q_A	Elastic	Consol	Total
OD-1	Poor to Fair	Poor	Fair	79	0.71	830	276	893	298	7.66	10.93	18.59
OD-2	Poor to Fair	Poor	Poor	104	0.93	829	277	846	282	7.21	11.2	18.41
OD-3	Poor to Fair	Good	Good	97	0.59	713	238	732	244	6.76	11.06	17.82
OD-4	Poor to Fair	Poor	Good	99	0.41	839	280	857	286	6.89	11.07	17.96
OD-5	Poor to Fair	Good	Good	99	0.46	767	256	759	253	6.91	11.08	17.99
OD-6	Poor to Fair	Poor	Fair	104	0.79	873	291	865	289	6.96	11.2	18.16
OD-7	Poor to Fair	Poor	Poor	132	1.39	891	297	883	284	7.03	11.05	18.08
OD-8	Poor to Fair	Poor	Fair	102	0.68	827	276	819	273	6.9	11.11	18.01
OD-9	Poor to Fair	Fair	Good	99	0.37	777	259	770	257	7.28	11.56	18.84
OD-10	Poor to Fair	Poor	Poor	99	0.75	856	285	849	283	6.79	10.9	17.69
OD-11	Poor to Fair	Poor	Poor	114	1.29	846	282	839	280	7.2	10.91	18.11
OD-12	Poor to Fair	Poor	Poor	104	1.28	764	255	757	252	7.24	11	18.24
OD-13	Poor to Fair	Poor	Poor	122	1.16	736	245	729	243	7.12	10.95	18.07
OD-14	Poor to Fair	Poor	Fair	122	0.90	813	271	805	269	6.9	11.02	17.92
OD-15	Poor to Fair	Poor	Poor	99	1.34	775	258	768	256	7.26	11.62	18.88
OD-16	Poor to Fair	Poor	Poor	102	1.08	702	234	695	232	7.42	11.3	18.72
OD-17	Poor to Fair	Poor	Very Poor	132	1.59	786	262	779	260	7.25	11.35	18.6
OD-18	Poor to Fair	Poor	Poor	99	1.73	836	279	829	276	7.41	11.25	18.66
OD-19	Poor to Fair	Poor	Very Poor	124	1.54	778	259	771	257	7.29	11.2	18.49
OD-20	Poor to Fair	Poor	Poor	140	1.37	862	287	855	285	7.07	11.26	18.33

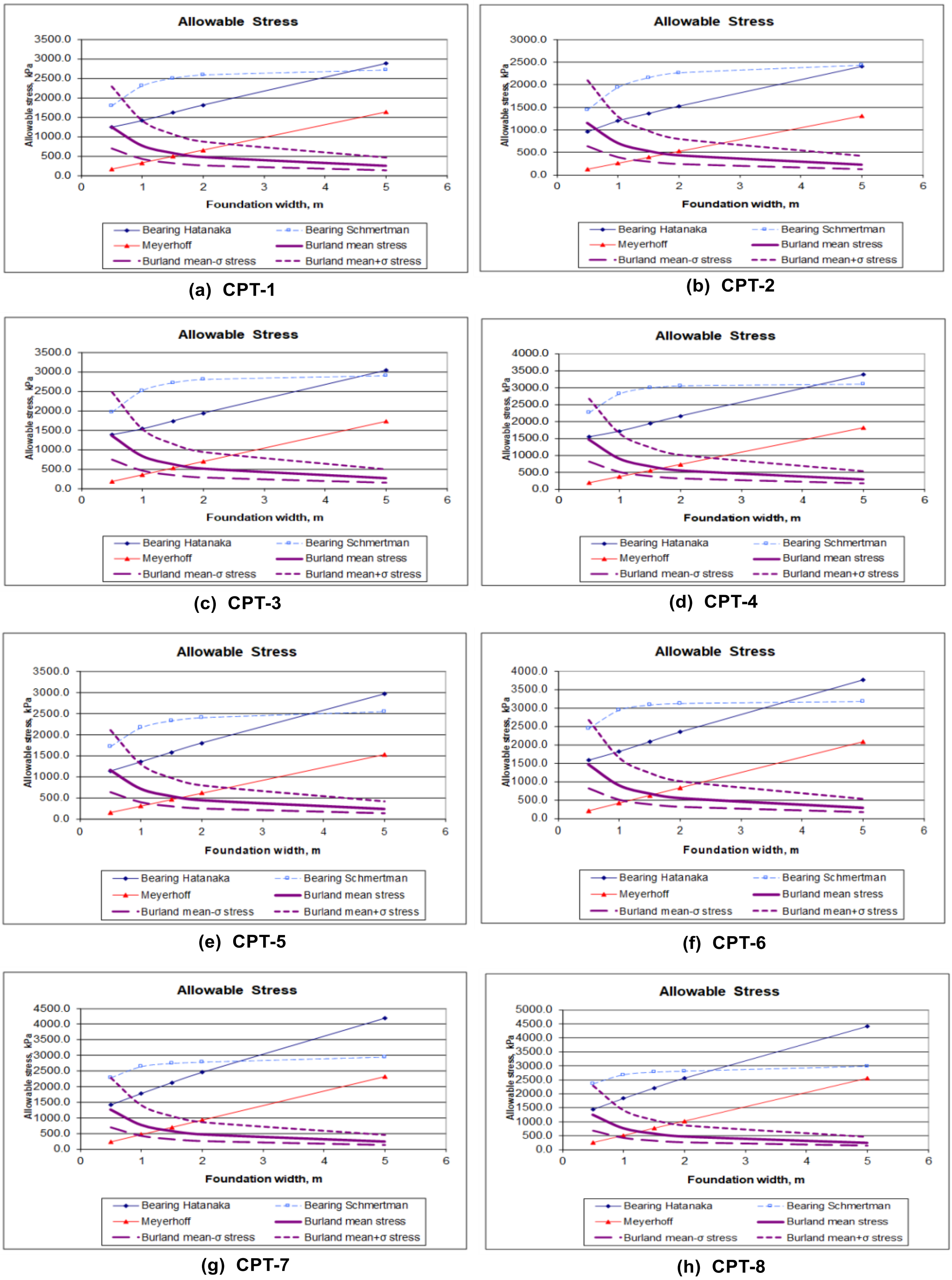

Fig. 14. Model Graph of the bearing pressure and stresses for various footing width using CPT data for maximum allowable settlement of 25 mm

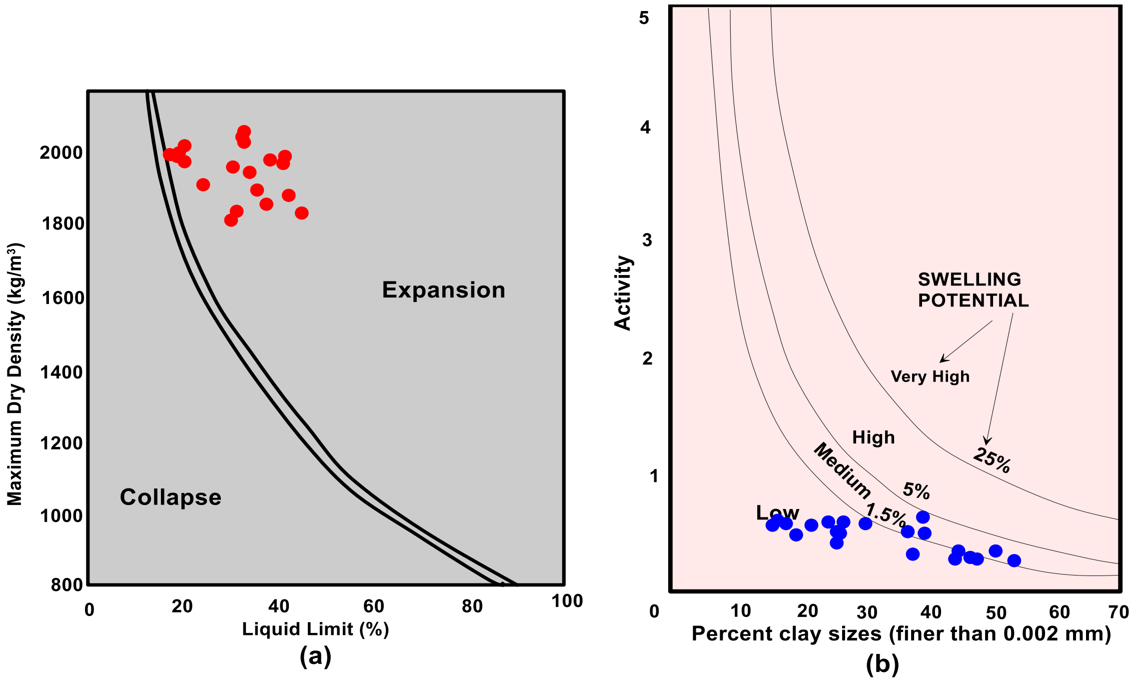

Fig. 15. (a) Workability and expansion potential of the soils (b) Classification chart for swelling potential (modified after [12], [68]

Table 10. Summary of desirability potential of the soil for various engineering uses

Various uses	Properties	Characteristics/relative suitability
Important Engineering parameter/property	Permeability when compacted	Semi impervious-impervious
	Shear strength when compacted saturated	Fair
	Compressibility when compacted saturated	Fair
	Workability as construction material	Fair
Earth fill dams	Rolled Earth fill dams (homogeneous embankment)	S: 3
	Rolled Earth fill dams (core/shell)	S: 3/4
Canal	Canal sections (erosion resistance)	S: 5
	Canal sections (compacted earth lining)	S: 2, where erosion is critical 4
Foundation	Foundations (where seepage is important)	S: 4
	Foundations (where seepage not important)	S: 8
Roadway	Roadway fills	S: 8
	Roadway surfacing	S:2

These rocks are supposed to be contemporaneous with those in the study area, as they both displaced the same structural features in magnitude and direction. The Aggregate Impact Value (AIV) ranged 11.2 (granite gneiss) to 15.2 (porphyritic granite), Aggregate crushed value (ACV) 19.7 – 24.2, and unconfined compressive strength (UCS) varied from 121.1 MPa (porphyritic granite) – 143.1 MPa (granite). Higher UCS values above 150 MPa were recorded for charnockite, grandiorite, quart schist, and quartzite. All the rocks are characterized with AIV, ACV, and UCS, with point load strength index (PLSI) ranged between 7.40 MPa – 8.82 (granite gneiss), and shear strength of 60.5 MPa (porphyritic granite) to 71.6 MPa (granite). Therefore the rocks have high value as foundation constructions, aggregate in pavement, building stone, and armourstones [73], [74], [75

5. CONCLUSION

The engineering properties of Ondo metropolis have been undertaken with a view to developing a robust geo-data set for effective design, construction and Management of Engineering. All methods adopted show some level of agreements, as the topsoil/subsoil is composed of sandy clay and sandy silt (SC-SM) material, with average % fines of 48.25. The depth to groundwater ranged from 2.5 m (in well) – 26 m (in borehole). The depth to basement rock is between 10.8 – 35.1 m (avg. 22.2 m), indicating a moderate to deep weathering profile, able to support burial of engineering utilities such as mast, transformer, gadgets, etc. The soil are generally inactive type with predominant illite-montmorillonite clay mineralogy group, with activity of 0.53. The soil showed good strength/shear characteristics of 186.8 KN/m^2 (USC), 13.1° (angle of friction), 56.56 KN/m^2 (cohesion) with unit weight of 20.1 KN/m^3. The soil has low CC (0.03236) and Cv (0.010342 m^2/yr) and Mv (0.194297 MPa^{-1}) indicative of low compressibility and expanding soil. The CPT revealed the soil to be composed of sandy silt to clayey silt and silty sand to sandy silt within 0.5 m to 2.25 m, with allowable bearing pressure of 164.45 KN/m^2 and average N_{cor} of 18.

Fig. 16. Some major rocks observed in the study area, including granites, gneisses, and migmatites

Table 11. Classification of residual soils by its primary origin [69]

Primary occurrence	Secondary occurence	Typical residual soils
Granite	Saprolite	Low activity clays and granular soils
Diorite		
Gabroo	Saprolite	High activity clays
Basalt		
Dolerite		
Gneiss	Saprolite	Low activity clays and granular soils
Schist		
Phyllite		Very soft rock
Sandstone		Thin cover depends on impurities. Older sandstones would have thicker cover
Shales	Red Black, marine	Thin clayey cover Friable and weak mass high activity clays
Carbonates	Pure Impure	No soil, rock dissolves Low to high activity clays

Findings also showed that the soil is unsuitable for base and sub-base courses with CBR less than 7 % and GI of 6 (avg.), with expected recommended minimum thickness of 79 – 140 mm (avg. 109 mm) obtained from design curves. Thus an inexpensive/economic mechanical stabilization or soil gradation and compaction will help in improving the bearing capacity and drainage characteristics. The average allowable bearing capacity of the soil for square and round foundations varied from 234 – 297 KN/m^2 (avg. 268 KN/m^2) and 232 – 298 KN/m^2 (avg. 268 KN/m^2). The estimated immediate/elastic settlement ranged from 6.76 – 7.66 mm (avg. 7.13 mm); and consolidation settlement varied between 10.9 – 11.62 mm (avg. 11.2 mm). The total settlement obtained is in between 17.69 – 18.88 mm (avg. 18.28 mm) for structural pressure of 100 KN/m^2. These results showed that the soils exhibit more of consolidation settlement than elastic. The embankment, the suitability index of the soil suggests a fair/expanding not collapsible construction material, as shown since the soils have high MDD at moderate OMC (avg. 1997 kg/m^3; 13.2%) greater than 1500 kg/m^3, they are ordinarily considered suitable.

Table 12. General engineering properties of common rocks [69]

Rock origin	Type	Characteristics	Permeability	: Deformability	Strength
Igneous coarse to medium grained – very slow to slow cooling	Granite, granodiorite, diorite, peridiorite	Welded interlocking grains, very little pore space	Essentially impermeable	Very low	Very high
Igneous fine grained – rapid cooling	Rhyolite, trachyte, quartz, dacite, andesite, basalt	Similar to above or can contain voids	With voids can be highly permeable	Very low to low	Very high to high
Igneous glassy – very rapid chilling	Pumice, scoria, vesicular basalt	Very high void ratio	Very high	Relatively low	Relatively low
Sedimentary – arenaceous clastic	Sandstones	Voids cement filled. Partial filling of voids by cement coatings	Low Very high	Low Moderate to high	High Moderate to low
Sedimentary – argillaceous clastic	Shales	Depends on degree of lithification	Impermeable	High to low, can be highly expansive	Low to high
Sedimentary – arenaceous clastic chemically formed	Limestone	Pure varieties normally develop caverns	High through caverns	Low except for cavern arch	High except for cavern arch
Metamorphic	Gneiss	Weakly foliated	Essentially impermeable	Low	High
		Strongly foliated	Very low	Moderate normal to foliations. Low parallel to foliations	High - normal to foliations. Low parallel to foliations
Metamorphic	Schist	Strongly foliated	Low	As for gneiss	
Metamorphic	Phyllite	Highly foliated	Low	Weaker than gneiss	
Metamorphic	Quartzite	Strongly welded grains	Impermeable	Very low	Very high
Metamorphic	Marble	Strongly welded	Impermeable	Very low	Very high

Table 13. Estimate of allowable bearing capacity in rock [68]

	Presumed allowable bearing capacity (kPa)			
	XW	DW	SW	FR
Igneous				
Tuff	500	1,000	3,000	5,000
Rhyolite, Andesite, Basalt	800	2,000	4,000	8,000
Granite, Diorite	1,000	3,000	7,000	10,000
Metamorphic				
Schist, Phyllite, Slate	400	1,000	2,500	4,000
Gneiss, Migmatite	800	2,500	5,000	8,000
Marble, Hornfels, Quartzite	1,200	4,000	8,000	12,000
Sedimentary				
Shale, Mudstone, Siltstone	400	800	1,500	3,000
Limestone, Coral	600	1,000	2,000	4,000
Sandstone, Greywacke, Argillite	800	1,500	3,000	6,000
Conglomerate, Breccia	1,200	2,000	4,000	8,000

Rocks of igneous and metamorphic rock are widespread in the study area, some are outcropped while some are deep seated with the subsurface. The magnetic and geoelectric section showed that the rocks are sometimes fractured. However, it is expected for the rock to have very high compressive/shear strength, modulus of elasticity, high crushing strength, low deformability; and presumable bearing capacity of 8, 000 – 10, 000 KPa especially when fresh (FR), and can be in between 5000 – 7000 KPa when partly or slightly weathered (SW) and thus can be trusted in most construction works, especially as foundation and road stones, as Falowo (2015) reported high values for aggregate impact value, aggregate crushed value, point load strength test, unconfined compression test, and direct shear strength for the rock in northern area of the same geological province which are contemporaneous in history. Therefore the rocks have high value as foundation constructions, aggregate in pavement, building stone, and armourstones.

ACKNOWLEDGEMENTS

The author is grateful to TETFund, Nigeria (under the Institution Based Research) Nigeria. Special appreciation to all students of Civil Engineering Technology Department for the assistance rendered during data acquisition.

REFERENCES

[1] S. Roy and S. K. Bhalla, "Role of geotechnical properties of soil on civil engineering structures". Sci Acad., 2017, Pub 7: 103-109.

[2] S. A. Adejumo, A. O. Oyerinde and M.O. Akeem, "Integrated geophysical and geotechnical subsoil evaluation for pre-foundation study of proposed site of vocational skill and entrepreneurship center at the polytechnic, Ibadan", SW, Nigeria. Int J Sci Eng, Res 2015, 6: 910-917.

[3] O. I. ADewuyi and O. F. Philips, Integrated Geophysical and Geotechnical Methods for Pre-Foundation Investigations. J Geol Geophys, 2018, 8: 453. doi:10.4172/2381-8719.1000453

[4] M. Carter and M. W. Symons, "Site Investigations and Foundations Explained". Pentech Press, London, 1989.

[5] A. A. Alabi, A. O. Adewale, J. O. Coker and O. A. Ogunkoya, Site Characterization for Construction Purposes at FUNAAB using Geophysical and Geotechnical Methods, RMZ – M&G, 2017, vol. 65, pp. 089–102. DOI: 10.2478/rmzmag-2018-0007

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

[7] O. E. Faseki, O. A. Olatinpo and O. B. Taiwo, Ground Investigation of an Engineering Site at Point Road, Apapa, Lagos, Nigeria. Journal of Engineering and Energy Research, 2016, 6(1), pp. 1–20.

[8] A. B. Hawkins (Ed.), "Site Investigation Practice: Assessing BS 5930. Engineering Geology Special Publication" No. 2, Geological Society, London, 1986.

[9] C. Craig (Ed.), Advances in Site Investigation Practice. Thomas Telford Press, London, 1996.

[10] F. G. Bell, "Engineering Geology and Construction" Spon Press, London, 2004.

[11] G. M. Olayanju, K. A. Mogaji, H. S. Lim and T. S. Ojo, "Foundation integrity assessment using integrated geophysical and geotechnical techniques: Case study in crystalline basement complex, southwestern Nigeria". J Geophys Eng, 2017, 14: 675- 690.

[12] R. D. Holtz and W. D. Kovacs, An Introduction to geotechnical engineering, Prentice Hall, 1981.

[13] J. E. Prentice, "Geology of Construction Materials" Chapman and Hall, London, 1990.

[14] P. M. Kearey and I. H. Brooks, "An Introduction to Geophysical Exploration". Blackwell Science Limited, 2002, 262pp.

[15] L. Williams, Fundamental of Geophysics. Cambridge University Press, 1997, 206-217pp.

[16] O. O. Osinowo and M. O. Falufosi, "3D Electrical Resistivity Imaging (ERI) for subsurface evaluation in preengineering construction site investigation." NRIAG Journal of Astronomy and Geophysics, 2018, 1-9pp. https://doi.org/10.1016/j.nrjag.2018.07.001

[17] K. F. Oyedele, L. Adeoti, S. Oladele and A. Kamil, "Investigation of a Proposed Four Story Building Sites Using Geophysical and Laboratory Engineering Testing Methods in Lagos, Nigeria". International Journal of Scientific Research in Knowledge, 2014, 2(2), pp. 83–91.

[18] J. O. Coker, "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, 2015, 1(2), 40-48.

[19] B. T. Kamtchueng, V. L. Onana, W. Y. Fantong, A. Ueda, R. F. D. Ntouala, M. H. D. Wongolo, G. B. Ndongo, A. N. Ze, V. K. B. Kamgang and J. M. Ondoa, Geotechnical, chemical and mineralogical evaluation of lateritic soils in humid tropical area (Mfou, Central-Cameroon): Implications for road Construction International Journal of Geo-Engineering, 2015: 6(1). DOI 10.1186/s40703-014-0001-0, 21pp.

[20] V. N. Vazirani and S. P. Chandola, "Concise Handbook of Civil Engineering" Revised Edition, Rajendra Ravindra Printers Pvt Ltd, New Delhi, India, 2008, 1318pp.

[21] W. M. Telford, L. P. Geldart and R. E. Sheriff, "Applied Geophysics", Cambridge University Press, 1991, 792pp.

[22] K. M. Sudha, S. M. Israil and J. Rai, "Soil characterization using electrical resistivity tomography and geotechnical investigations". Journal of Applied Geophysics, 2009, vol. 67, pp. 74-79.

[23] M. S. Zhdanov and G. V. Keller, "The geoelectrical method in geophysics exploration". Elsevier, Amsterdam, 1994.

[24] I. Adewumi and M. O. Olorunfemi, "Using geoinformatics in construction management". Journal of Applied Sciences, 2005, 5 (4): 761-767.

[25] K. O. Cetin and C. Ozan, CPT-based probabilistic soil characterization and classification. J Geotech Geoenviron Eng, 2009, 135(1):84–107. doi:10.1061/(ASCE)1090-0241(2009)135:1(84).

[26] A. Kezdi and L. Rethati, "Handbook of Soil Mechanics", Volume 3: Soil Mechanics of Earthworks, Foundations and Highway Engineering Elsevier, Amsterdam, 1988.

[27] Federal Meteorological Survey, "Atlas of the Federal Republic of Nigeria", 2nd Edition, Federal Surveys, 1997, 160pp.

[28] P. N. Iloeje, "A new geography of Nigeria". Longman Nigeria Limited, Lagos, 1981.

[29] Nigerian Geological Survey Agency. "Geological and Mineral Map of Ondo State, Nigeria", 2006.

[30] F. G. Bell, "Engineering Geology" Second Edition, Elsevier Limited, 2007.

[31] F. H. Chen, "Soil Engineering: Testing, Design, and Remediation" Edited by Morris, PE, CRC Press LLC, Corporate Blvd, NW, Boca Raton, Florida 33431, 2000, 288pp. ISBN 0-8493-2294-4 (alk paper)

[32] K. Terzaghi, R. B. Peck and G. Mesri, Soil Mechanics in Engineering Practice, 3rd ed., John Wiley Sons Inc., 1996.

[33] J. S. Ojo, M. O. Olorunfemi, O. J. Akintorinwa, S. Bayode, G. O. Omosuyi and F. O. Akinluyi, "Subsoil Competence Characterization of the Akure Metropolis, Southwest Nigeria". Journal of Geography, Environment and Earth Science International. 2015: 3(1): 1-14. DOI: 10.9734/JGEESI/2015/15851

[34] E. G. Imeokparia and O. O. Falowo, "Subsoil Foundation Support Assessment in Owo Area of Ondo State, Southwestern Nigeria" Geotechnical and Geological Engineering, 2020, 38(2), pp 2009-2026 DOI: 101007/s10706-019-01144-0

[35] A. J. Weltman and J. M. Head, Site Investigation Manual. Construction Industry Research and Information Association, Special Publication No. 25, London, 1983.

[36] M. J. Tomlinson, "Foundation Design and Construction". Seventh Edition, Prentice Hall, Harlow, England, 2001.

[37] A. K. Upadhyay, "Soil and Foundation Engineering", Second Edition, S.K. Kataria & Sons New Delhi India 2015, 349pp.

[38] Nigeria Geological Survey, "Geological Map of Southwestern Nigeria", Geological Survey Department, Ministry of Mines, Power and Steel, Nigeria, 1984.

[39] P. B. Attewell and I. W. Farmer, "Principles of engineering geology" John Wiley & Sons, Inc, New York, 1988, 1045pp. ISBN-13: 978-94-009-5709-1 DOl: 101007/978-94-009-5707-7

[40] V. N. S. Murthy, "Advanced Foundation Engineering, Geotechnical Engineering Series". CBS Publishers and Distributors Ltd, New Delhi, India, 2007, 795pp.

[41] B. Vandre, A. Budge and S. Nussbaum, "DCP- A Useful Tool for Characterizing Engineering Properties of Soils at Shallow Depths". Proceedings of 34th Symposium on Engineering Geology and Geotechnical Engineering, Utah State University, Logan, UT, 1998.

[42] ASTM, "Annual Book of ASTM Standards" Sections 4.02, 4.08, 4.09 and 4.13. ASTM International, West Conshohocken, PA., 2006.

[43] BS 1377, "Methods of Tests for Soils for Civil Engineering Purpose", British Standard Institution HMSO London, 1990.

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

[45] P. K. Robertson, "Soil classification using the cone penetration test". Can Geotech J., 1990, 27:151–158.

[46] R. E. S. Moss, R. B. Seed, R. S. Olsen, "Normalizing the CPT for overburden stress". J Geotech Geoenviron Eng, 2006, 132(3):378–387. doi:10.1061/(ASCE)1090-0241(2006)132:3(378).

[47] H. L. Erickson and A. Drescher, "Bearing capacity of circular footings". J. Geotech. Geoenv. Eng., 2002, 128(1), 38-43

[48] M. Hatanaka and A. Uchida, "Empirical correlation between penetration resistance and effective friction of sandy soil". Soils & Foundations, Vol. 1996, 36 (4), 1-9, Japanese Geotechnical Society.

[49] G. G. Meyerhof, "Penetration tests and bearing capacity of cohesionless soils", Journal of the soil mechanics and foundation division, ASCE, Vol. 82, No. SM1, January, 1956, pp. 1-19.

[50] J. H. Schmertmann, Measurement of insitu shear strength, keynote lecture, Proceedings of the conference on in-situ measurement of soil properties, June 1-4, 1975, vol. II, American Society of Civil Engineers.

[51] J. B. Burland and M. C. Burbidge, "Settlement of foundations on sand and gravel", Proceedings of the Institution of Civil Engineers, Part 1, 1985, 78, Dec., 1325-1381.

[52] O. O. Falowo and S. D. Dahunsi, "Geoengineering Assessment of Subgrade Highway Structural Material along Ijebu Owo – Ipele Pavement Southwestern Nigeria", International Advanced Research Journal in Science, Engineering and Technology (IARJSET), 2020, Vol 7, Issue 4, 10pp. DOI 1017148/IARJSET20207401

[53] O. O. Falowo and W. Olabisi, "Engineering Geological and Geotechnical Site Characterization for Economic Design of Structures and Earthwork", Journal of Research and Innovation for Sustainable Society, 2020, 2(2), pp43-60. Doi: 1033727/jriss202027:43-60

[54] O. O. Falowo, O. O. Ojo and A. S. Daramola, "The use of magnetic survey in engineering site characterization". International Journal of Innovative research and development, 2015, vol., 4(13): pp. 74-85.

[55] R. Brassington, "Field Hydrogeology". Wiley, Chichester, 1988.

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

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

[58] G. McNally, "Soil and Rock Construction Materials". Spon Press, London, 1998.

[59] S. F. Brown, "Soil mechanics in pavement engineering". Geotechnique, 1996, 46: 383–426.

[60] K. P. George and W. Uddin, "Subgrade characterization for highway pavement design, final report". Jackson, MS: Mississippi Department of Transportation, 2000.

[61] P. H. Wright, "Highway Engineering", Sixth Edition. John Willey and Sons: New York, NY., 1986.

[62] AASHTO, "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., 2006.

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

[64] J. Putra, A. H. Ramadhansyah and E. S. Norhidayah, "Editorial: Trends and Advanced Materials for Pavement and Road Infrastructure" Frontiers in Materials 2021, (8), 1-2.

[65] P. W. Mayne, "Geotechnical site characterization using Cone, piezocone, SPTu, and VST", Civil and Environmental Engineering Department, Georgia Institute of Technology, 2001.

[66] C. M. Martin, "Exact bearing capacity calculations using the method of characteristics". Issues lecture, 11th Int. Conf. of IACMAG, Turin, in press, 2005.

[67] C. M. Martin, "ABC – Analysis of Bearing Capacity v1.0. Software and documentation", 2004. available online from www-civil.eng.ox.ac.uk/people/cmm/software/abc

[68] M. Carter and S. P. Bentley, "Correlations of soil properties", Pentech Press Publishers, London, 1991, 129pp.

[69] R. E. Hunt, "Geotechnical engineering investigation handbook", second edition, Taylor and Francis, 2005.

[70] M. R. Smith and L. Collis (Eds.), "Aggregates: Sand, Gravel and Crushed Rock Aggregates". Third Edition. Engineering Geology Special Publication No. 9. Geological Society, London, 2001.

[71] M. R. Smith (Ed.), "Stone: Building Stone, Rock Fill and Armourstone in Construction". Engineering Geology Special Publication No. 16, Geological Society, London, 1999.

[72] O. O. Falowo, "Engineering properties of some basement rocks of Nigeria as Aggregate in Civil Engineering Pavement Construction". Asian Review of Environmental and Earth Sciences, 2019, vol. 6, No. 1, 28-37. DOI: 10.20448/journal.506.2019.61.28.37

[73] P. M. Archana and K. R. Padma, "Engineering and Geological Evaluation of Rock Materials as Aggregate for Pavement Construction'. International Advanced Research Journal in Science, Engineering and Technology, Vol. 3, Special Issue 3, August 2016. DOI 10.17148/IARJSET

[74] J. P. Latham (Ed.), "Advances in Aggregates and Armourstone Evaluation". Engineering Geology Special Publication, 1998, No. 13. Geological Society, London.

[75] E. M. Winkler, "Stone: Properties, Durability in Man's Environment". Springer-Verlag, New York.G. Eason, B. Noble, and I. N. Sneddon, "On certain integrals of Lipschitz-Hankel type involving products of Bessel functions," Phil. Trans. Roy. Soc. London, 1973, vol. A247, pp. 529–551, April 1955. *(references)*