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The Usefulness of Engineering Geological and Geotechnical Studies in Civil Engineering Sites Foundation Design Parameters Consideration and Construction: A Case Study in SW Nigeria

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ABSTRACT

Engineering site investigation for shallow foundation design is carried out in Ose Local Government of Ondo State using integrated geotechnical, geophysical, hydrogeological measurements, and borehole logging. The northern part of the area is underlain by rocks of Migmatite-Gneiss Complex of Southwestern Nigeria, while the southern area is Cretaceous Ewekoro Formation. The static water level ranges from 3.1 to 15.9 m, while the hydraulic head varies between 38.9-371.8 m. Low static water level is observed in clay shale/sandstone lithology, while granite, migmatite and granite gneiss are characterized by high static water level. The hydraulic head map shows possible groundwater flow direction of south, as the values of hydraulic head in the study area reduce towards the south. The plasticity chart and clay mineral group indicate predominant high plasticity soil group (CH) with general AASHTO class of A-6 characteristic of poor soil material. All samples fall above the A-line, indicating clayey inorganic material. The mineral group of the soil is closer to illites. The soils are also characterized by high plasticity index (23.8% avg.) and show moderate to high shear parameters. The targeted CPT value of 100 kg/cm² corresponding to 245KN/m² is obtained at a depth of 3.0 m. The allowable bearing capacity of the soils (within upper 3 m) varies from 6 KN/m² to 309 KN/m² respectively, while the ultimate bearing capacity varies between 18 KN/m² and 927 KN/m², respectively. The settlement values obtained from the area are generally less than 20 mm from each of the subsurface layers. The minimum settlement values obtained vary from 0.68 mm to 0.80 mm at depth of 0.8 m and 3 m respectively. These depths would be appropriate for the design of foundation structure because they are characterized with low settlement with allowable bearing pressures greater than 100 KN/m².

1. Introduction

Site investigation or characterization is the process of collection of information, appraisal of data, assessment, and reporting. It also involves location of geological hazards in the ground beneath the site (Krynine and William, 1957). Evaluation of general suitability of a site for any proposed project, to enable an adequate and economical design to be made, and disclose and make provision for difficulties that may arise during construction is an important aspect of civil engineering design process and construction that have to be taking with seriousness it deserves (Falowo, 2020). Lack of the good knowledge of the properties of the subsurface materials leads to the failure of most engineering structures especially roads and buildings (Ochuko, 2013; Daramola et al., 2018). Consequently, an adequate and properly structured site investigation is essential for successful design/construction of civil engineering structure. The design of structure (such as highway, dam, tunnel, buildings) that is economical, safe to construct, and durable, depends upon an adequate understanding of the nature of the ground by assessing the distribution or stratification of the materials in the ground, their properties and behavior under various influences and constraints during the construction (Atat et al., 2012; Oyedele et al., 2016). Soil strength or competence to support civil engineering structure depends on the cohesion and friction between soil particles (Krynine and Williams, 1957; Kumar, 2003). Determination of bearing capacity and behavior of foundation under applied load is one of the most important consideration in the design process of structures (Akinlabi and Adeyemi, 2014). It is of great importance to carry out engineering site characterization of a proposed site to ascertain the bearing capacity of the foundation (soil) material to support any intended structure (Shiva and Darga, 2016). The investigation may involve direct mechanical boring, pitting and trenching for subsoil sequence delineation, groundwater table mapping, soil sampling, and geotechnical laboratory analysis (Falowo and Aliu, 2020; Aka et al., 2020; Adewuyi and Philips, 2018; Essex, 2007; Geotechnical Engineering Office, 1987).

In order to furnish adequate information for settlement prediction, the boring should penetrate all strata that could shear or consolidate under the load of the structure (Oyeyemi and Olofinnade, 2016; Kumar, 2003; Evinemi et al., 2019). However, relying on a single method or technique of site investigation, could render deficient and misleading result. Hence an integrated method would render more efficient probing.

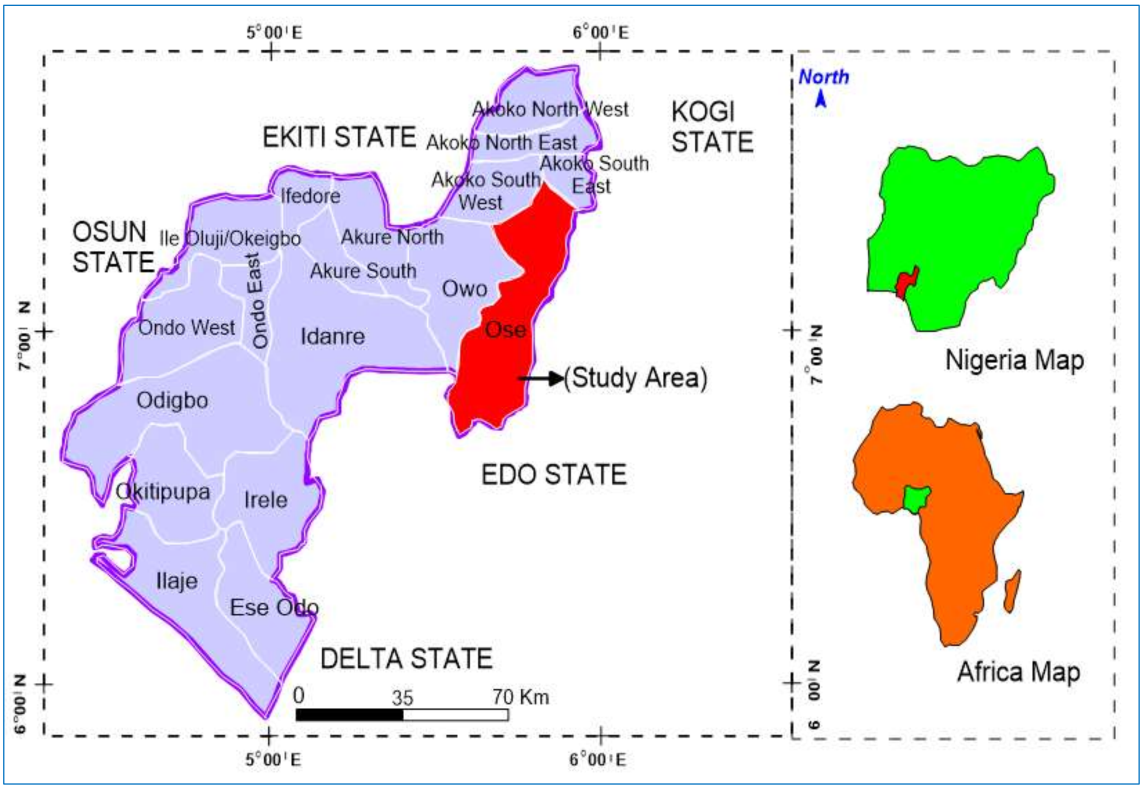

Fig. 1. Location of the study area on the map of Africa and Nigeria

In recent time, geotechnical analysis is becoming increasingly useful in subsurface engineering study to obtain information about the physical and engineering properties of the subsoil which may include the strength, stability and competence of the material that make up the subsoil materials, which serves as host for foundations of engineering structures (Aina et al., 1996; Adewumi and Olorunfemi, 2005; Idornigie et al., 2006; Mundher, 2016). However geotechnical boring gives point source (1-D) information and hence lack continuity in subsurface imaging which a geophysical method can provide. Such investigations (such as cone penetration and standard penetration tests) can be very expensive if it has to be representative or detailed. Involvement of a cheaper method like surface geophysical method will reduce cost without compromising quality. Geophysical surveys have shown to be efficient and cost effective in providing the required geotechnical information (Olayanju et al., 2017; Adeoti et al., 2016; Fajana et al., 2016; Adejumo et al., 2015; Osinowo and Falufosi, 2018; Olorunfemi et al., 2000) and in mineral exploration (Oyedele et al., 2016; Oyedele et al., 2016; Ehinola et al., 2012; Bayowa et al., 2016).

In addition, geophysical techniques especially electrical methods of prospecting are generally quick, inexpensive and generally non-destructive to provide information about the subsurface properties, depth to bedrock, location and distribution of conductive fluids, location and orientation of fractures and faults with accuracy in the shallow subsurface

(Reynolds, 2011). Environmental and engineering applications of geophysical techniques such as the electrical resistivity (ER), seismic refraction, electromagnetic (EM), magnetic and ground penetrating radar are used singly or in combinations for engineering site investigation (Anomohanran, 2012; Abiye et al., 2019; Adeoye-Oladapo and Oladapo, 2011; Onoja and Osifila, 2015; Faseki et al., 2016). The applications of such geophysical investigation were determination of depth to bedrock, structural mapping and evaluation of subsoil competence electrical resistivity surveys can provide a rapid evaluation of subsurface conditions. Consequently, this research is aimed at determining the engineering site condition of Ose local government area of Ondo State for civil engineering foundation design and construction. The research work is expected to give important geotechnical parameters in relation to geology that are valuable for infrastructure development planning. This would invariably help in proper location and design of (structure) foundation footing and suitable construction materials needed for any anticipated project in the study area.

2. Description of the Study Area

2.1. Location

The study area is Ose Local Government Area of Ondo State, Nigeria. It is located within longitudes 5˚20ᴵ E and 6˚00ᴵ E and latitudes 6˚30ᴵ N and 7˚30 ᴵ N (Fig. 1). The study area is easily accessible by roads such as Ikare-Owo highway, Benin-Ifon highway and Akure-Owo highway. It is bounded by Kogi and Ekiti State in the North, Edo in the east, and Delta States in the south. Over 60% of the State is underlain by basement migmatites, gneisses and granites which form rugged hills (around Elegbeka, Ido Ani, Idogun, Imeri) and rolling plains (Ifon, Okeluse). The area lies geographically within the tropical rain forest belt of hot and wet equatorial climatic region characterized by alternating wet and dry climate seasons (Iloeje, 1981), which is strongly controlled by seasonal fluctuation in the rate of evaporation.

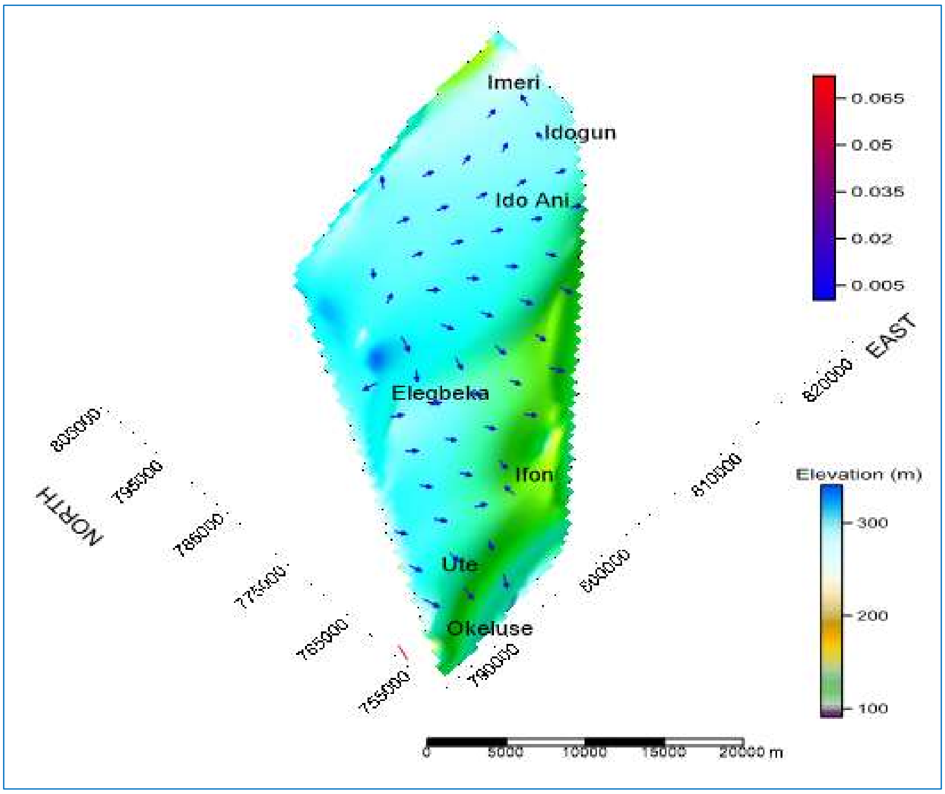

Fig. 2. 2-D Surface map overlay by1-Grid Vector Map showing potential Groundwater flow direction

2.2. Climate and Geology

The available rain data shows that mean annual rainfall ranges from 1000-1800 mm and mean temperature of 24 °C to 27 °C (Federal Meteorological Survey, 1982). There is rapid rainfall during the month of March and cessation during the month of November. June and September are the critical months when rainfall is usually on the high side. The vegetation is of tropical rainforest and is characterized by thick forest of broad-leaved trees that is ever green. The vegetation of the area is dense and made up of palm trees, kolanut trees and cocoa trees. However teak and Gmelina trees are also predominant in the area. The surface elevation generally varies between less than 40 m and 250 m except at Ido Ani area where surface elevation is between 300 m and 400 m (Fig. 2), with potential groundwater flow direction due south (Fig. 2). The area of study falls within the Southwestern basement rock, which is part of Nigerian Basement complex (Obaje, 2009).

The area is underlain mainly by rocks of the Migmatite-Gneiss Complex which is predominated by quartzite, granite gneiss and schist (Fig. 3). Granite gneiss and gneiss are the most widespread rock in the area; which mineralogically contain quartz and feldspar dominating mineral, other minerals such as muscovite, tremolite, microcline and biotite are common as well (Obaje, 2009; Jones and Hockey, 2010). Quartzites which are prominent as ridge vary in texture from massive to schistocity due to the presence of flaky minerals like mica. However, the southern part is basically underlain by cretaceous sediments in Imoru, Ajagba, Arimogija, Ute and Okeluse. These areas are characterized by thick lateritic clay/sand (more than 10 m in places) especially around Imoru and Ute in Ose local government area; and predominant lateritic clay and kaolinite in Okeluse. Also, oolitic sand and sharp sand are observed (Fig. 4). Field observations also show the presence of shale and sand stone.

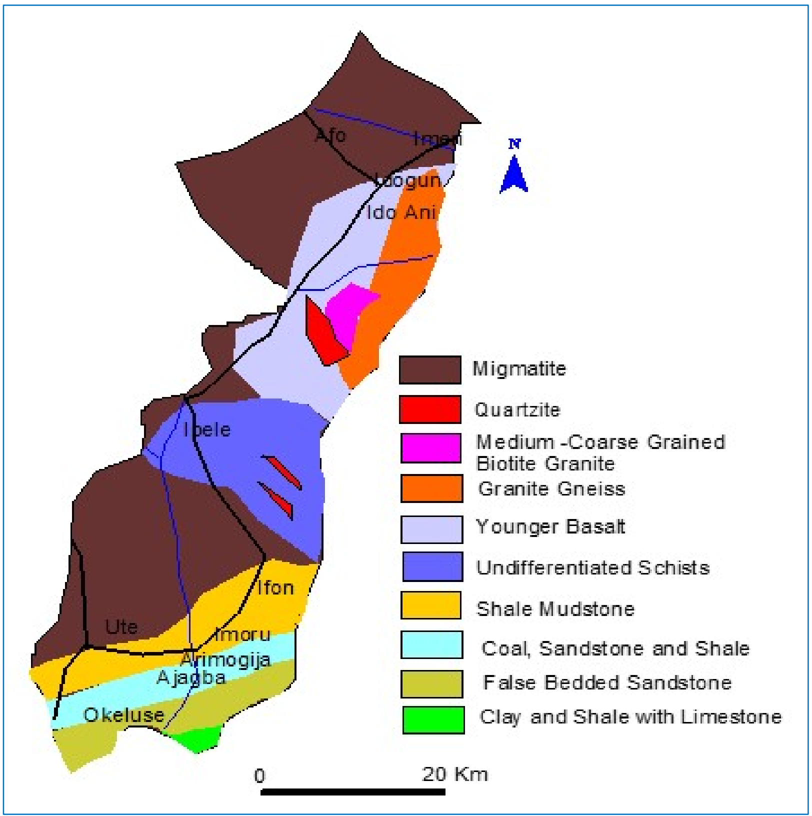

Fig. 3. Geology map of the study area (Modified after Geological Survey of Nigeria, 1984)

3. Materials and Methods

3.1. Field mapping and laboratory analysis

Detailed geo-environmental site evaluation technique usually involved determining subsurface conditions by examining soil samples taken from various depths in exploratory boreholes drilled at closely-spaced points over proposed site. The borehole(s) must be deep enough to penetrate all strata and terminate possibly on the bedrock. In addition, both in situ and laboratory tests should be conducted on foundation soil, in order to obtain information about the subsurface geology and engineering properties (Das, 1997; Blyth and De Freitas, 1984).

This research work involved detailed reconnaissance survey, soil exploration, water table determination, borehole drilling and detailed geological/geophysical investigation. During the reconnaissance survey, locations available for sampling were established. Exploratory method follows with excavation of ten (10) test pits for sampling. Location of boreholes was based on the preliminary geological conditions form test pits, the dimensions of the area and the observed engineering problems. The testing points were arranged in such a pattern that soil profiles can be assessed across the site (Fig. 5).

This study adopted combined geophysical survey (ER, EM, and combined VES and Horizontal profiling (HP)), geotechnical investigation (field and laboratory studies), and hydrogeological measurements. The geophysical survey was carried out along twelve traverses with maximum length (spread) of 300 m. The geographic co-ordinates of data stations were taken using GARMIN'S GPS 12-Channel model. The use of the ER method for geophysical exploration seems to be the most applied for geophysical technique in shallow subsurface investigation. This is applied through the use of the vertical electrical sounding (VES, HP, or combined VES/HP) technique which measures vertical or lateral or combined lateral and vertical changes of electrical resistivity (Lowrie, 2007).

In terms of field logistic, it's economical, easy and straight forward to use. In electromagnetic (EM) surveying, the electrical conductivity of the ground is measured as a function of depth and/or horizontal distance. Different rocks (and buried structures/objects) exhibit different values of electrical conductivity. Mapping variations in electrical conductivity can identify anomalous areas worthy of further geophysical or intrusive investigation (Gadallah and Fisher, 2009).

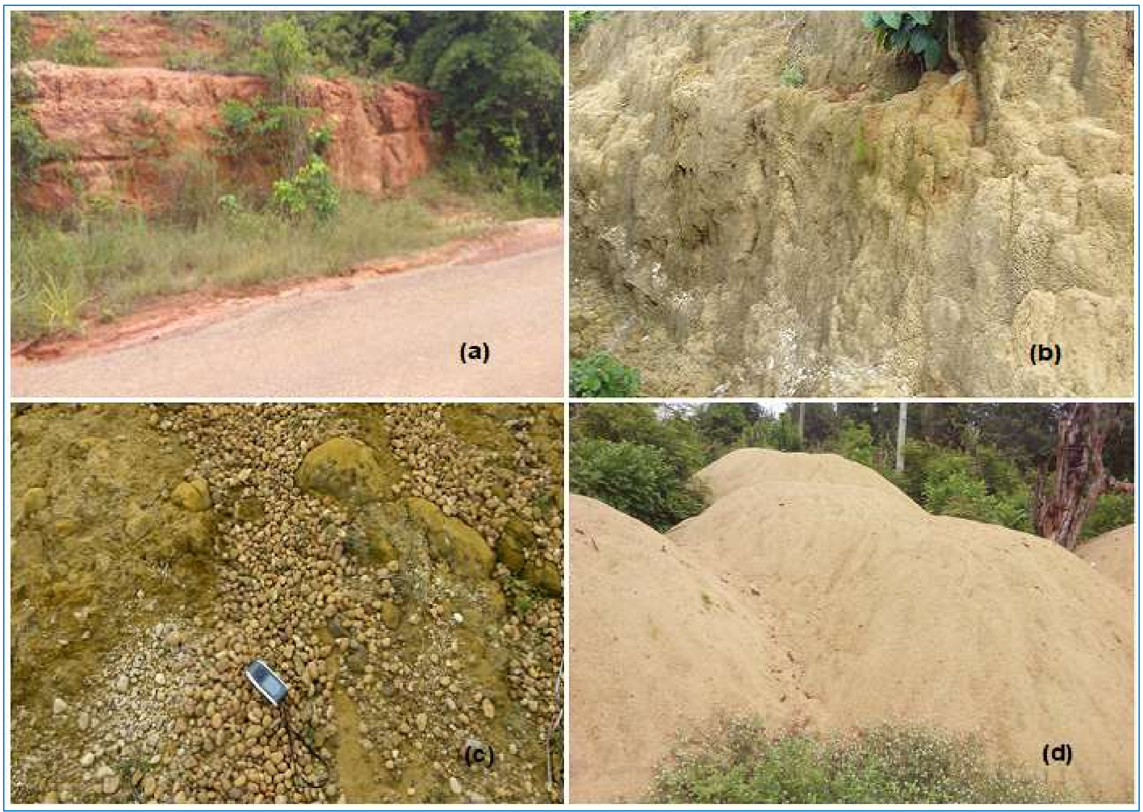

Fig. 4. Site pictures of some sedimentary deposits observed in southern parts of the study area

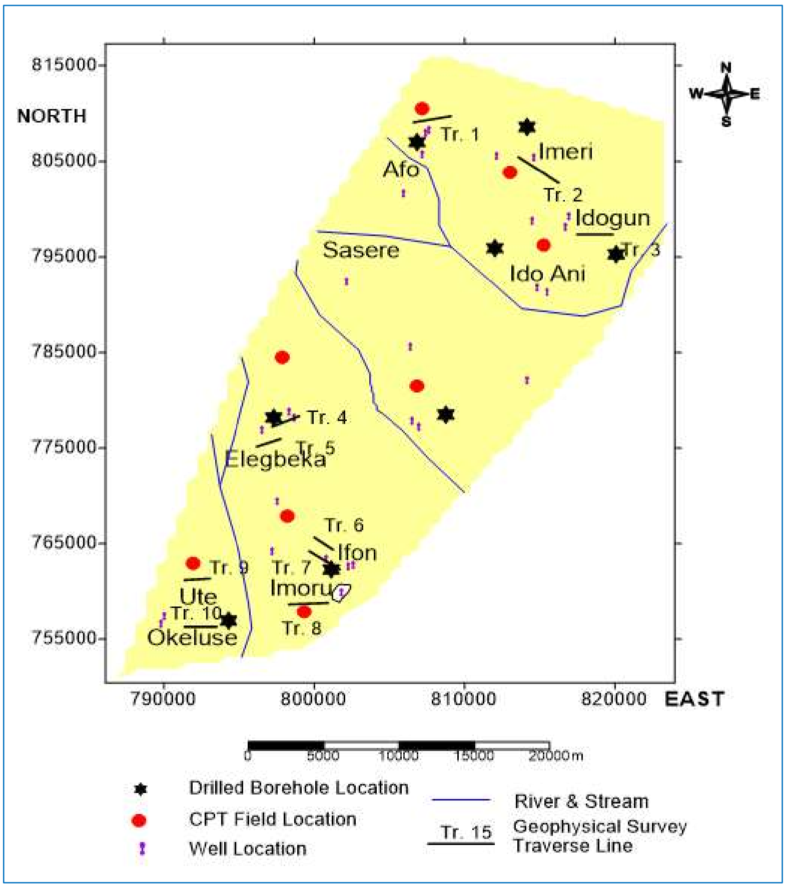

Fig. 5. Base map for geotechnical, geophysical, and borehole sampling

The electromagnetic method is based on the induction of electric currents in the ground by the magnetic component of electromagnetic waves generated at the surface. An alternating current, of variable frequency, is passed through a coil of wire (a transmitter coil). This process generates an alternating primary magnetic field which, in turn, induces very small eddy currents in the earth, the magnitude of which is directly proportional to the ground conductivity in the vicinity of the coil. These eddy currents then generate a secondary magnetic field, a part of which is intercepted by a receiver coil. The interaction between the primary and secondary magnetic flux and the receiver coil generates a voltage that is related to the electrical conductivity of the subsurface, expressed as milliSiemen/metre (Lowrie, 2007; Gadallah and Fisher, 2009).

Twenty-Five vertical electrical sounding were done in the study area using Ohmega Resistivity meter with current electrode spacing of 1-225 m. The apparent resistivity measurements at each station were plotted against half electrode spacing on bi-logarithmic graph sheets. Geoelectric sections along the traverses were produced from the interpretation results of the partial curve matching. The combined horizontal profiling and vertical electrical sounding (VES/HP) utilized dipole-dipole array. The inter-electrode spacing of 5 m was adopted while inter-dipole separation factor (n) was varied from 1 to 5.

The apparent resistivity values were calculated using geometric factor and plotted at an intersection of lines drawn at 45° from mid-points of the potential and current di-poles. The VLF-EM method utilized the inline profiling technique. The ABEM-WADI equipment was used for the measurements, while the measured raw real and filtered real components were plotted against station position. The 2-D modeling of the filtered real was carried out using "KHFFILT" software. The hydrogeological measurements were taken from thirty-four open-wells and eight boreholes, from which hydraulic head and static water level measurements. In addition, eight boreholes were drilled in the study area to the basement rock/competent soil.

Field activities include eight (8) Cone Penetrometer Tests (CPT) using Dutch Cone Penetrometer which measures the resistance of penetration into soils using a 60% steel cone. The cone has an apex angle of 60° and a base area of 10.2cm². The test was carried out by securing the winch frame to the ground by means of anchors which provided the necessary power to push the cone into the ground (Robertson, 1990).

The cone and the tube were pushed together into the ground for 20 to 25 cm; the cone was pushed ahead of the tube for 3.5 cm at a uniform rate of about 2 cm/s. The resistance to the penetration of the cone registered on the pressure gauge connected to the pressure capsule was recorded. The tube was then pushed down and the procedure described above is repeated. This process was continued until the anchors start to lift out of the ground. Successive cone resistance readings were plotted against depth to form a resistance profile. The layer sequences were interpreted from the variation of the values of the cone resistance with depth. These were followed by collecting ten disturbed sampling of the soils and consequently taken to the laboratory for analysis. All field and laboratory tests were in accordance with the British Standard 1377 (1990).

The soil samples were subjected to the following laboratory tests; Atterberg limits (liquid and plastic limits), grain size analysis for both sieve and hydrometer tests, unit weight determination, moisture content, undrained triaxial test at cell pressures of 30, 60 and 90 Kpa, and specific gravity tests. The degree of saturation of the soils was computed from the unit weight, water content and specific gravity data, based on phase relationship equation. The shear strength parameters; undrain cohesion (Cu) and angle of internal friction (ϕu) of the soil samples were obtained from the relation between the principal stresses at failure over three Mohr's circles. Consequently, calculated allowable bearing capacity was obtained using Meyerhof (1974) equation which covers all foundations irrespective of the width:

$$q_a = 2.7q_c \text{ (KN/m}^2\text{)} \tag{1}$$

where; q_a is the allowable bearing capacity and q_c is the cone penetration resistance value. The ultimate bearing capacity was calculated by multiplying the factor of safety of three (3) on the allowable bearing capacity. A simple and rapid method of estimating settlement of footings using CPT tip resistance value by Meyerhof (1974) was adopted in this study for settlement analysis of the subsoil for foundation footing width of 1.5 m:

$$s = {}^{qB}\!/_{2q_c} \tag{2}$$

where; s is settlement, q is net foundation stress, B is footing width and q_c is cone CPT value. Where;

$$q = \frac{Maximum\ Exerted\ Pressure}{Cross\ Sectional\ Area} \tag{3}$$

4. Results and Discussion

4.1. Hydrogeological measurement

The result of the measurements taken from the open wells and boreholes in the study area is presented in Table 1 and Fig. 6. The static water level ranges from 3.1 to 15.9 m, while the hydraulic head varies between 38.9 - 371.8 m. Low static water level is observed in clay shale/sandstone lithology, while granite, migmatite and granite gneiss are characterized by high static water level. The hydraulic head map (Fig. 7) shows the possible groundwater flow direction of south, as the values of hydraulic head in the study area reduce towards the south. Groundwater conditions play an important part in the stability of foundations. If the water table lies very close to the base of footings, the bearing capacity and settlement characteristics of the soil would be affected. The general groundwater flow direction is south. Consequently, the SWL in the northern areas are very high indicating a likelihood of high-water levels during the rainy season (which could even lead to spring condition), that may affect basement/ foundation footings, and subsequently compromise the integrity of such structures.

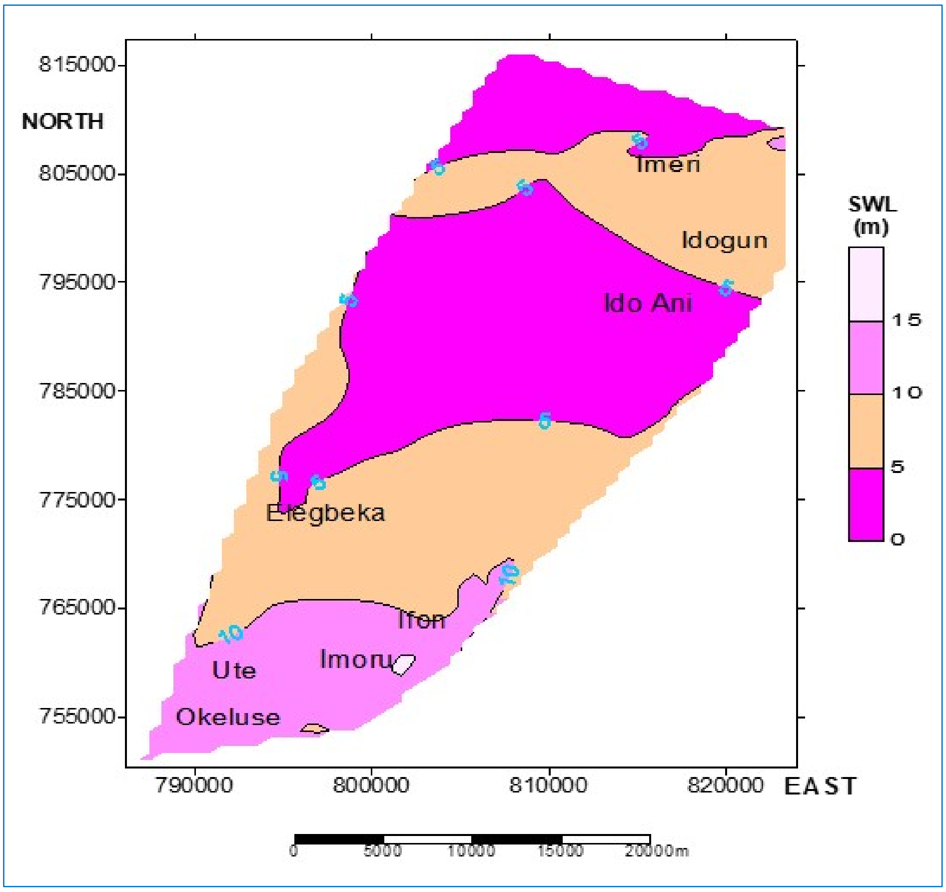

Fig. 6. Static water level map generated for the study area

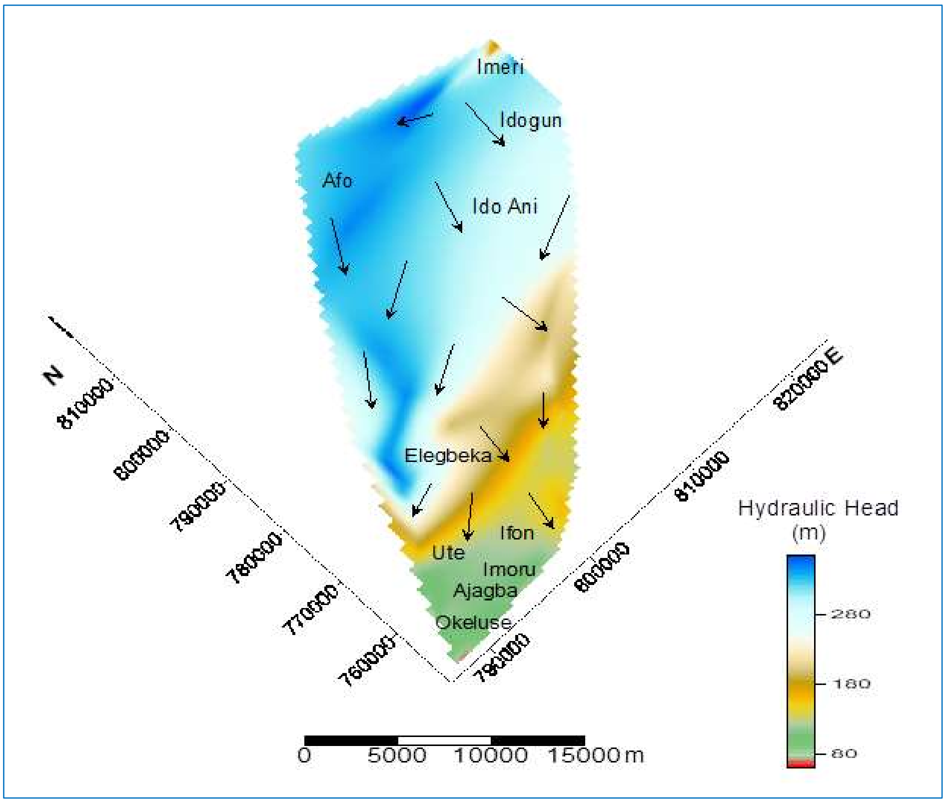

Fig. 7. Map of Hydraulic Head Measurements obtained in the study area

4.2. Geotechnical investigation

The summary of the geotechnical results is presented in Tables 2 and 3. The various values of Natural Moisture Content (NMC) obtained from laboratory tests are presented in Table 2. The natural moisture content gives information on the condition of the soil. The natural moisture content of soils varied from 6.4 % to 14.4 %, with an average of 10.7 %. The samples have low moisture content in their natural state. The results of the grain size distribution analyses are summarized in Table 2. The tested soils show % fines (percentage passing 0.002 mm) variation of 32.6 – 54.6 %, with an average of 48.4 %. The % of sand and gravel in the sampled soils vary from 42.2 % to 66.2 % and 1.1 % to 3.2 % respectively. Therefore, the soils are dominated by sand and clay (clayey sand). This classification correlates well with clayey sand/sandy clay delineated on the geoelectric sections and the 2-D resistivity structure of geophysical investigations. The plot of the characteristics of the soil samples on the Casagrande plasticity chart (Casagrande, 1947) and clay mineral group are shown in Fig. 8 and indicate predominant high plasticity soil group (CH) with general AASHTO class (AASHTO, 1982) of A-6, characteristic of poor soil material.

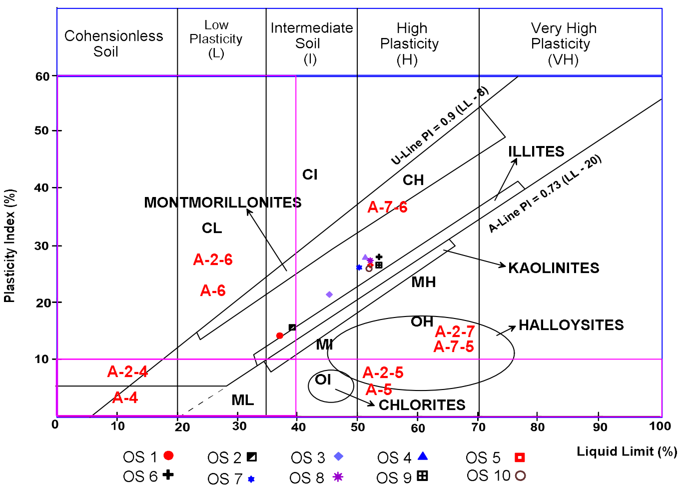

Fig. 8. Casagrande Plasticity Chart for the sampled soils showing predominantly High Plasticity

All the samples fall above the A-line, indicating that they consist of clayey inorganic material (Jegede, 2000; Adewuyi and Philips, 2018; Falowo, 2020). Based on British Standard BS 1377 (1990) if percentage fine is less than 35 % it is adjudged a good foundation material. Therefore, the soil samples can be generally classified as unsuitable foundation material since the mean % of fines (i.e., 48.4 %) is greater than 35%, hence a high degree of stabilization is required (Okogbue and Onyeobi, 1999) due to high % of fines (clay) with high plasticity. The mineral group of the soil is closer to illites (Fig. 8) which has non-expansive lattice (Grim, 1953). The engineering performance of clay soil is affected by the total moisture content and by the energy with which the moisture is held. It develops as an alteration product of feldspars, micas or ferromagnesian silicates upon weathering or may be form from other clay minerals during diagenesis (Bell, 2007).

Table 1. Hydrogeological data obtained from the study area

No	Easting	Northing	Elevation (m)	S.W.L (m)	H.H (m)	Geology
1	786123	750451	65	14.9	50.1	CSH
2	786178	750530	61	14.2	46.8	CSH
3	786379	750651	54	15.1	38.9	CSH
4	786488	750700	63	12.3	50.7	CSH
5	786653	750782	78	15.9	62.1	CSH
6	786820	750850	79	15.4	63.6	CSH
7	787729	751284	78	13.8	64.2	SD
8	788082	757635	59	10.7	48.3	SD
9	788589	751633	73	9.1	63.9	CSH
10	788016	757522	93	13.1	79.9	ST/SH
11	789286	757450	132	13.4	118.6	ST/SH
12	804279	759249	124	9.8	114.2	SH
13	804811	759762	123	8.8	114.2	SH
14	804551	760192	143	11.7	131.3	CSH
15	806434	766213	168	12.1	155.9	SH/MS
16	806294	766507	174	14.1	159.9	SH/MS
17	806400	766504	188	13.4	174.6	SH/MS
18	806411	766633	155	12.8	142.2	SH/MS
19	806464	767025	173	9.9	163.1	SH/MS
20	805937	766978	169	14.9	154.1	SH/MS
21	805342	767566	188	10.3	177.7	SH/MS
22	800229	775443	201	5.4	195.6	MG/SH
23	800010	775782	210	6.2	203.8	MG/SH
24	794157	784455	233	7.1	225.9	MG/SH
25	815129	808313	366	5.5	360.5	MG/SH
26	816697	808779	376	4.2	371.8	MG/SH
27	819845	808353	276	5.9	270.1	MG/SH
28	820480	808381	286	5.2	280.8	MG/SH
29	820755	808420	287	4.9	282.1	MG/SH
30	823974	809267	260	3.1	256.9	MG/SH
31	823627	809120	262	5.7	256.3	MG/GG
32.	823833	809775	263	3.9	259.1	MG/GG
33.	807112	816226	322	3.4	318.6	MG/GE
34.	807225	817321	311	3.3	307.7	MG/GE

Note; SWL: Static Water Level, H.H: Hydraulic Head, MG: Migmatite, GG: Granite Gneiss, CSH/SD: Clay Shale/Sand, SH/ST: Shale/Sandstone, MS: Mudstone, SH: Shale, GE: Granite

The specific gravity correlates well with the mechanical strength of sub grade (Mesida, 1981) and depends on the amount of sand and also on mineral constituents and mode of formation of the soil. Table 2 presents the results of the specific gravity (GS) for all the soil samples and vary between 2.67 - 2.76 with an average of 2.72. These values portray resistant soil material in line with Brink et al. (1992). The liquid limit, plastic limit, and plasticity index of the soil samples vary from 37.4 % to 54.2 %, 23.5 % to 26.6 %, and 13.3 % to 29.0 %, with average values of 49.1 %, 25.3 %, and 23.8 % respectively. Good foundation materials must among other significant criteria be of low plasticity such that its resistance to swelling, total expansion and linear shrinkage should be minimal (Okogbue, 1985; Okogbue and Ene, 2008; Oyedele et al., 2011). The high plasticity index (23.8% avg.) and liquid limit values are indicative of poor engineering properties (Federal Ministry of Works and Housing, 1972). Linear shrinkage is an important parameter in the evaluation of material soils for foundation construction. It has been suggested that a linear shrinkage (LS) value below 8 % is indicative of a soil that is good for foundation material (Brink et al., 1992; Mededor, 1983). The lower the linear shrinkage, the lesser the tendency of the soil to shrink when desiccated. The results of the linear shrinkage tests are presented in Table 2. The values range between 7.7 % and 10.1 % with an average value of 8.2 %. Using Table 4, the soils can be classified as medium-good material.

Table 2. Summary of the geotechnical results

Sample No	NMC (%)	GSD			SG	Consistency Limits			LS (%)	Compaction		AASHTO Group	Rating
		Fines (%)	Sand (%)	Gravel (%)		LL (%)	PL (%)	PI (%)		OMC (%)	MDD (Kg/m³)		
OS 1	6.4	40.8	59.2	-	2.67	37.4	24.1	13.30	10.1	16.3	1855	A-6	Poor
OS 2	8.3	41.3	55.5	3.2	2.69	39.5	25.2	14.30	9.6	17.2	1810	A-6	Poor
OS 3	6.7	54.3	44.2	1.5	2.68	45.0	23.5	21.50	8.7	25.9	1586	A-7	Poor
OS 4	11.3	54.6	42.2	3.2	2.68	52.4	24.3	28.10	7.7	27.0	1549	A-7	Poor
OS 5	11.1	53.3	45.4	1.3	2.75	53.4	24.6	28.80	7.7	27.5	1532	A-7	Poor
OS 6	12.4	52.8	46.1	1.1	2.76	54.2	25.2	29.00	7.7	26.3	1572	A-7	Poor
OS 7	11.4	32.6	66.2	1.2	2.76	50.4	26.3	24.10	7.7	19.0	1741	A-7	Poor
OS 8	12.2	48.1	50.6	1.3	2.76	52.4	26.6	25.80	7.7	19.8	1711	A-7	Poor
OS 9	12.3	52.8	47.2	-	2.74	53.8	26.6	27.20	7.7	18.8	1749	A-7	Poor
OS 10	14.4	52.9	45.9	1.2	2.74	52.5	26.6	25.90	7.7	19.7	1715	A-7	Poor

Note; NMC: Natural Moisture Content, GSD: Ground Size Distribution, SG: Specific Gravity, LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, LS: Linear Shrinkage

Table 3. Bearing Capacity and settlements obtained for all the CPT locations at an interval of 0.2 m depth

Depth (m)	CPT 1			CPT 2			CPT 3			CPT 4		
	q_a (KN/m²)	q_u (KN/m²)	S (mm)	q_a (KN/m²)	q_u (KN/m²)	S (mm)	q_a (KN/m²)	q_u (KN/m²)	S (mm)	q_a (KN/m²)	q_u (KN/m²)	S (mm)
0.2	14	42	9.20	13	39	15.10	21	63	10.25	8	24	9.67
0.4	18	54	10.10	15	45	12.30	20	60	10.37	6	18	6.78
0.6	24	72	9.02	19	57	11.95	26	78	8.05	10	30	4.86
0.8	35	105	4.77	25	75	10.44	32	96	6.67	31	93	5.01
1.0	52	156	4.59	36	108	7.32	43	129	4.89	43	129	2.79
1.2	74	222	3.90	55	165	5.40	70	210	2.99	42	126	21.00
1.4	80	240	2.52	62	186	4.22	88	264	2.41	76	228	19.13
1.6	83	249	2.35	72	216	4.10	83	249	2.55	22	66	10.13
1.8	91	273	2.21	81	243	3.90	98	294	2.16	11	33	6.47
2.0	112	336	1.52	95	285	2.89	116	348	1.82	21	63	5.01
2.2	121	363	1.44	105	315	2.21	123	369	1.72	33	98	3.44
2.4	155	465	1.28	119	357	1.90	156	468	1.35	82	249	2.54
2.6	160	480	1.25	126	378	1.70	172	516	1.23	61	183	1.88
2.8	216	648	0.99	154	462	1.42	215	645	0.98	42	126	1.29
3.0	222	666	0.62	176	528	0.94	309	927	0.68	112	336	0.80
Depth (m)	CPT 5			CPT 6			CPT 7			CPT 8		
	q_a (KN/m²)	q_u (KN/m²)	S (mm)	q_a (KN/m²)	q_u (KN/m²)	S (mm)	q_a (KN/m²)	q_u (KN/m²)	S (mm)	q_a (KN/m²)	q_u (KN/m²)	S (mm)
0.2	8	24	10.40	6	18	11.10	10	30	10.85	6	18	9.88
0.4	10	30	10.20	9	27	10.44	14	42	10.47	7	21	9.70
0.6	12	36	11.50	13	39	8.65	18	54	8.50	13	39	8.75
0.8	16	48	9.95	18	54	6.70	112	336	7.77	20	60	6.60
1.0	21	63	7.70	26	78	5.89	165	495	5.44	80	240	2.22
1.2	27	81	5.20	44	132	3.90	174	522	3.56	98	294	1.99
1.4	39	117	2.55	56	168	2.42	198	594	1.91	118	354	1.66
1.6	58	174	2.10	64	192	2.05	218	654	1.72	133	399	1.55
1.8	76	228	1.80	79	237	2.86	232	696	1.55	162	486	1.33
2.0	92	276	1.75	95	285	2.22	-	-	-	188	564	1.00
2.2	104	312	1.71	114	342	1.98	-	-	-	215	645	0.92
2.4	125	375	1.52	123	369	1.62	-	-	-	-	-	-
2.6	143	429	1.40	152	456	1.08	-	-	-	-	-	-
2.8	166	498	1.22	184	552	0.80	-	-	-	-	-	-
3.0	202	606	0.80	212	636	0.55	-	-	-	-	-	-

Table 4. Soil Classification according to Shrinkage Limit (After Murthy, 2007)

Shrinkage Limit (%)	Quality of Soil
<5	Good
5 – 10	Medium good
10 – 15	Poor
>15	Very poor

The importance of compaction test is to improve the desirable load bearing capacity properties of a soil as foundation material. The best for foundation engineering structures is one with high MDD at low OMC (Jegede, 1999). The Optimum Moisture Content (OMC) and the Maximum Dry Density (MDD) obtained from the tested soil are presented in Table 2. The OMC varies from 16.3 % and 27.5 % with an average of 21.8 %. The MDD ranges from 1532 - 1855 kg/m³, with a mean value of 1682 kg/m³. The degree of compaction is sensitive to moisture content, thus the higher the value of MDD and the lower OMC, the more suitable the material to sustain any load imposed. All the soil samples have MDD at moderately low OMC.

The result of the undrained triaxial test is presented in Table 5. Cohesion is the ability of the soil to resist shearing stress. The cohesion of the studied soils varies between 85.1 kPa -

96.5 kPa. The values of angle of friction are between 22.1° and 29.1°. This range of value is classified as hard soil material by Holtz and Kovacs (1981). These values indicate moderately cohesive material with moderately high shear strength. Also, these range of values make the soil samples moderately competent to accommodate civil engineering foundation structures, especially shallow foundation of 3 m. The cone penetrometer test was carried out in order to obtain geotechnical parameters required for the design of the foundation support for civil engineering structures.

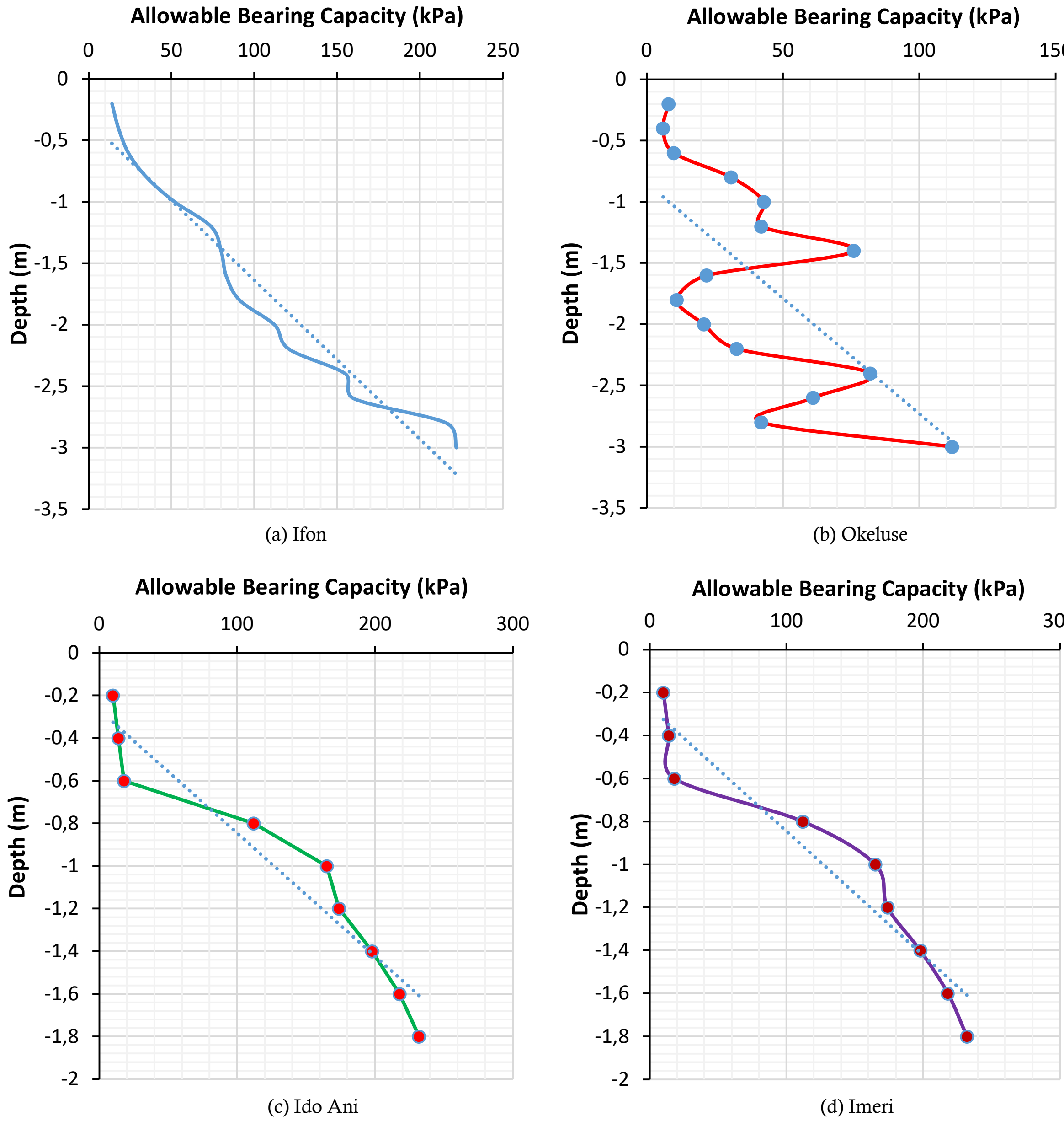

Fig. 9. Typical plots of cone resistance values against depth

Groundwater table was encountered only at all the locations between 1.9 and 2.5 m depths. The range of water levels observed during penetration agrees with shallow water level measured in Table 1. Sediments like sand, silt, gravel, clay, and laterite were encountered. Figure 8 shows the plots of cone resistance with depth in the study area. All the plots except at Okeluse which shows alternation of low and high cone resistance values, while the targeted CPT value of 100 kg/cm² corresponding to 245kN/m² was obtained at a depth of 3.0 m. The graphs generally show increase in cone resistance with depth. Six major geological layers are delineated comprising clay, clay silt, silty clay, sandy clay, clay sand, and lateritic clay. Therefore, using Meyerhof's equation, bearing capacity (pressure) of the soil units and the corresponding settlements were obtained and presented in Tables 3. The allowable bearing capacity of the soils (within upper 3 m) varies from 6 kN/m² to 309 kN/m² respectively, while the ultimate bearing capacity varies between 18 kN/m² and 927 kN/m² respectively. The settlement values obtained from the area are generally less than 20 mm from each of the subsurface layers. The minimum settlement values obtained vary from 0.68 mm to 0.80 mm at depth of 0.8 m and 3 m.

These depths would be appropriate for the design of foundation structure as they are characterized with low settlement with allowable bearing pressures greater than 100 kN/m².

Table 5. Results of the quick undrained triaxial test

Sample No	Deviator stress at different cell pressures, σ (kPa)			Cohesion, c (kPa)	Angle of friction (θ °)
	30	60	90		
OS 1	395	470	545	85.1	29.1
OS 2	395	468	542	86.2	28.8
OS 3	337	383	429	91.3	23.5
OS 4	337	381	426	92.7	23.1
OS 5	337	378	419	96.1	22.1
OS 6	337	384	431	90.4	23.7
OS 7	337	382	427	92.3	23.2
OS 8	395	459	523	93.2	27.3
OS 9	395	455	515	96.5	26.6
OS 10	337	380	422	94.7	22.5

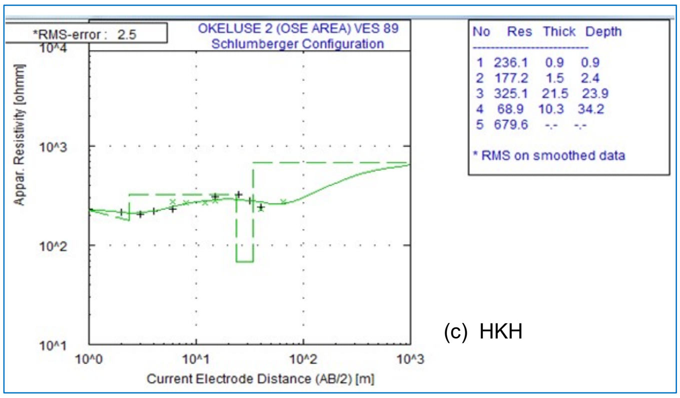

Fig. 10. Typical Curve Types obtained in the study area

In fact, the commonly accepted basis of design is that the total settlement of a footing should be restricted to about 25 mm (Bell, 2007; Skempton and Bjerrum, 1957). Therefore, in order to achieve this, the depth of footing in the study should be between 0.8 (in Ido Ani area) - 2.0 m (in Okeluse/Ifon environment). The minimum allowable bearing capacity of at least 100 kN/m² (corresponding to 80 - 100 kg/cm² cone resistance values) for a raft, simple spread foundation and/or structure was recommended by Adewumi and Olorunfemi (2005) and Oyedele and Okoh (2011). These allowable bearing pressures are considered appropriate for use in the design of bases, strips or raft foundations. Subsequently, shallow foundation such as pad or raft foundation of reinforced concrete can be adopted in the study area (Schmertmann, 1978).

4.3. Geophysical results

Table 6 gives a summary of the results of the VES curves obtained from the study area. The number of layers varies between three (3) layers and six (6) layers; while four-layer configuration is the most dominant. Seven curve types have been identified: HKH, H, HA, KQ, KH, QH, and KHK (Table 7). The most occurring curve types identified is KH. Typical curve type is shown in Fig. 10. These type curves can be classified into four distinct classes in relation to their engineering competence (Table 8) by using their interpreted resistivity and thickness values (Olorunfemi et al., 2005) as Class 1, 2, 3, and 4 and are rated Very Good, Good, Moderate, and Fair respectively. A-curve type is rated "Very Good" and widely found in Akoko area; KH, KHK, and KQ are rated "Good"; HKH is rated "Moderate"; and HA, and QH are rated "Fair". Since the KH curve is the most preponderant of all the curve types, therefore the area can be classified as "Class 2" signifying a Good subsoil material at a depth below the topsoil.

Table 6. Summary of the VES data obtained in the study area

VES No	Resistivity (Ohm-meter)						Thickness (m)					Depth (m)					Curve Type
	ρ_1	ρ_2	ρ_3	ρ_4	ρ_5	ρ_6	h_1	h_2	h_3	h_4	h_5	d_1	d_2	d_3	d_4	d_5	
71	161	107	178	5094	-	-	4.6	1.2	3.7	-	-	4.6	5.8	9.5	-	-	HA
72	610	111	160	752	-	-	0.6	3.8	18.5	-	-	0.6	4.4	22.9	-	-	HA
73	302	64	922	-	-	-	0.5	5.9	-	-	-	0.5	6.4	-	-	-	-
74	264	41	9662	-	-	-	1.2	6.4	-	-	-	1.2	7.6	-	-	-	H
75	188	18	361	4351	-	-	0.7	2.0	5.4	-	-	0.7	2.7	8.0	-	-	HA
76	272	32	127	1102	-	-	2.5	0.8	3.5	-	-	2.5	3.3	6.8	-	-	HA
77	238	23	275	-	-	-	1.6	12.8	-	-	-	1.6	14.4	-	-	-	H
78	464	157	5175	-	-	-	2.2	10.6	-	-	-	2.2	12.8	-	-	-	H
82	142	143	56	5532	99	-	1.1	0.5	1.7	8.0	-	1.1	1.6	3.3	11.3	-	KHK
83	90	201	60	8199	-	-	2.0	1.4	5.4	-	-	2.0	3.4	8.8		-	KH
84	675	207	379	657	1400	-	0.5	0.4	0.7	23.8	-	0.5	0.9	1.6	25.4	-	HAA
85	464	140	125	698	-	-	0.7	1.6	11.2	-	-	0.7	2.3	13.5		-	QH
86	384	152	316	283	5203	-	0.8	1.0	4.5	11.9	-	0.8	1.8	6.3	18.2	-	HKH
87	346	32	172	113	5372	-	0.7	1.1	2.2	14.2	-	0.7	1.8	3.9	18.1	-	HKH
88	342	189	154	877	-	-	0.9	23.7	15.2	-	-	0.9	24.6	39.8		-	QH
89	236	177	325	69	670	-	0.9	1.5	21.5	10.3	-	0.9	2.4	23.9	34.2	-	HKH
90	689	1277	49	10392	-	-	0.7	21.4	30.2	-	-	0.7	22.2	52.3	-	-	KH
91	3389	11220	7966	207	-	-	0.3	3.7	10.3	-	-	0.3	4.0	14.3	-	-	KQ
92	182	1147	139	28840	-	-	0.4	9.8	28.2	-	-	0.4	10.1	38.3	-	-	KH
93	117	172	60	245	-	-	1.5	11.4	19.8	-	-	1.5	12.8	32.6	-	-	KH
94	46	265	43	13239	-	-	1.0	7.0	13.2	-	-	1.0	8.0	21.2	-	-	KH
95	268	61	1599	-	-	-	4.0	27.6	-	-	-	4.0	31.6	-	-	-	H
96	200	2134	141	436	-	-	0.5	0.4	45.7	-	-	0.5	0.9	46.5	-	-	KH
97	271	353	69	853	-	-	1.0	9.6	16.1	-	-	1.0	10.6	26.7	-	-	KH
98	73	114	69	910	-	-	0.5	1.5	35.3	-	-	0.5	2.0	37.3	-	-	KH

Table 7. Curve Types and their Statistical Frequency obtained from the study area

Location/Curve type	HKH	H	HA	HK	KHKH	KQ	KH	A	KHK	QH	Total no of VES curves
Ose Area	3	5	5	-	-	1	8	-	1	2	25

Table 8. Engineering Competence Rating of the Curve Types

Class	Curve type	Engineering competence characteristics	Rating	Overall rating (%)
1	A	Increased resistivity/competence with depth	Highly stable	70 (Very Good)
2	KH, KHK, KQ	Highly resistive hardpan underlies low resistivity clayey topsoil, and weathered layer underlies the former, then follow by fresh basement. In KHKH type, there is confined fracture basement	Stable at depth below the topsoil with appreciable thickness	60 (Good)
3	HKH	Succession of resistive topsoil followed by a more conductive horizon with appreciable thickness, and then another less conductive layer underlies the latter, which overlies the fresh basement. HKH is associated with confined fracture basement	Stable at shallow depth, within the topsoil with thin thickness	50 (Moderate)
4	HA, QH	Low resistivity, high porosity, and low weathered zone sandwiched between high resistivity topsoil (but thin thickness) and basement rock	Stable at shallow depth, within the topsoil with thin thickness	40 (Fair)

Along Traverse 29, the VLF-EM model shows some strongly conductive targets at distances 60 m and 75 m, and 180 m to 190 m (Fig. 11 a-b). The topsoil and the weathered layer show resistivity values in the range of 41 Ω-m - 610 Ω-m (Fig. 11c) and combined thickness ranging from 2.7 m to 7.6 m (Fig. 11d). This range is suggestive of incompetent subsoil with poor geotechnical characteristics. The 2-D VLF-EM model shows a very conductive targets which is not represented on the geoelectric section and 2-D resistivity structure along Traverse 32 (Fig. 12). The topsoil and the weathered layer along this traverse are thicker at the center than at both flanks. The resistivity of the topsoil varies from 90 Ω-m to 675 Ω-m and the thickness of 0.5 to 2.0 m (Fig. 12c). The wide variation of the resistivity values is due to variation in degree of compaction, composition, and fluid saturation. The underlying weathered layer is clayey in nature with resistivity variation of 56 Ω-m to 207 Ω-m (Fig. 11c-d). Along Traverse 34, the 2-D model (Fig. 13a) reveals multiple conductive zones suspected to be a water filled geological formation at depth less than 10 m and greater than 40 m. The geoelectric section (Fig. 13b) and the resistivity structure (Fig. 13d) show an alternation of sedimentary deposit consisting of clay sand, laterite, clay, sandy clay, and sand. The upper layer shows high degree competency for civil engineering foundation structure judging from their resistivity values. The overburden is moderately thick to distribute the imposed load to the basement or bedrock.

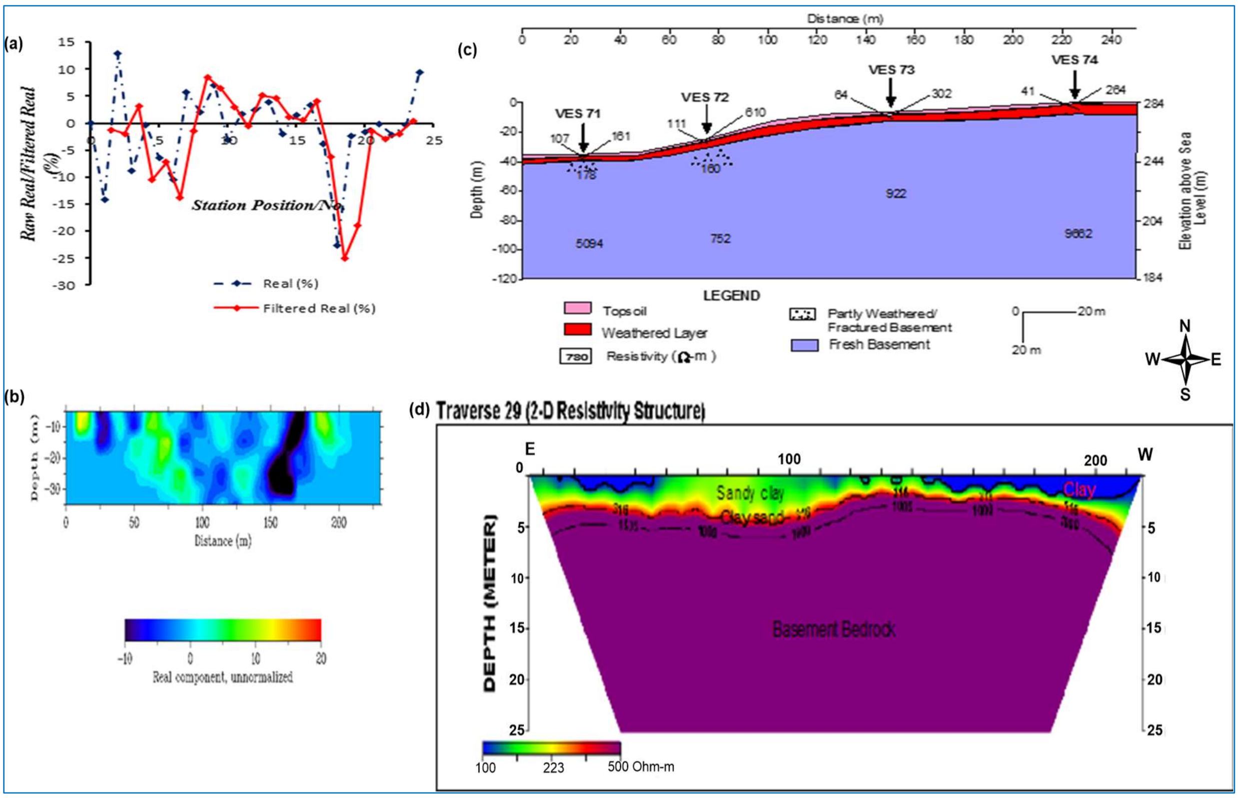

Fig. 11. (a) VLF-EM Real and Filtered Real Components, (b) VLF-EM 2-D Model, (c) Geoelectric Section, (d) Dipole-Dipole Resistivity Structure along Traverse 29 (Ido Ani-Ose)

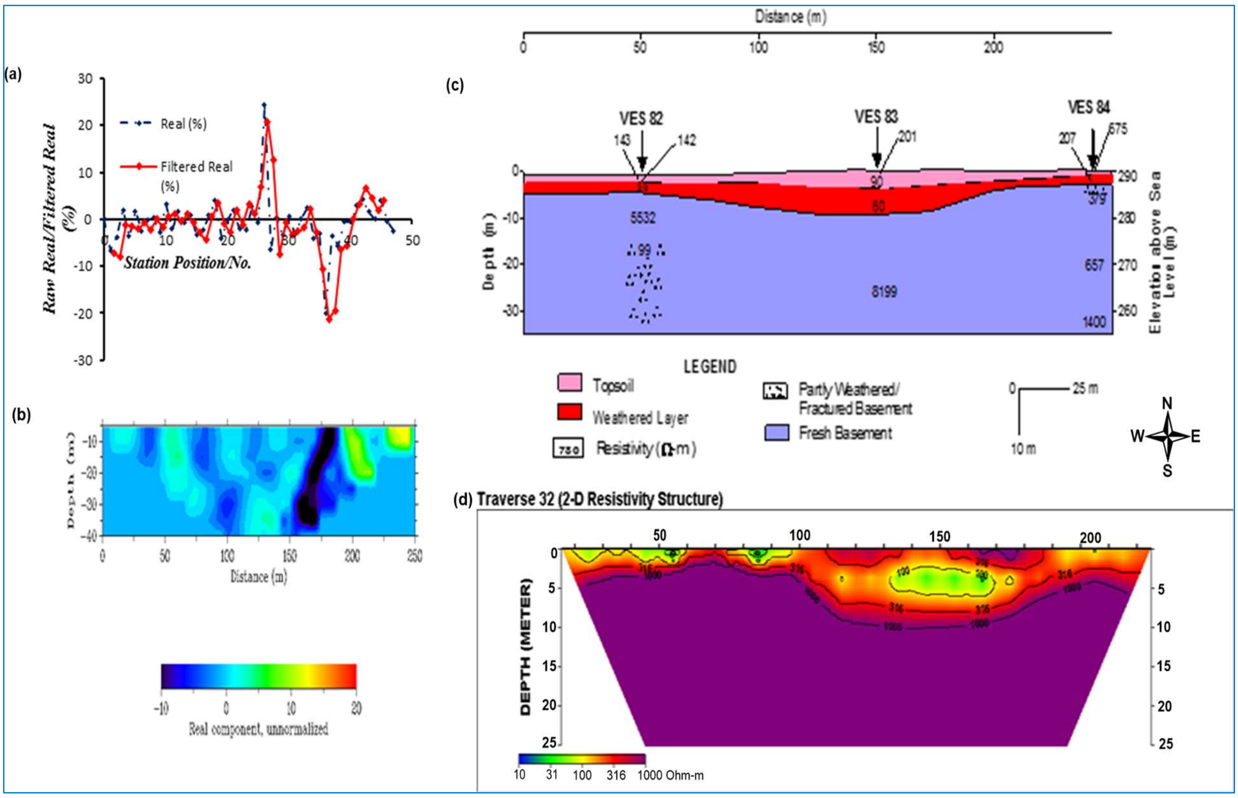

Fig. 12. (a) VLF-EM Real and Filtered Real Components, (b) VLF-EM 2-D Model, (c) Geoelectric Section, (d) Dipole-Dipole Resistivity Structure along Traverse 32 (Ifon-Ose)

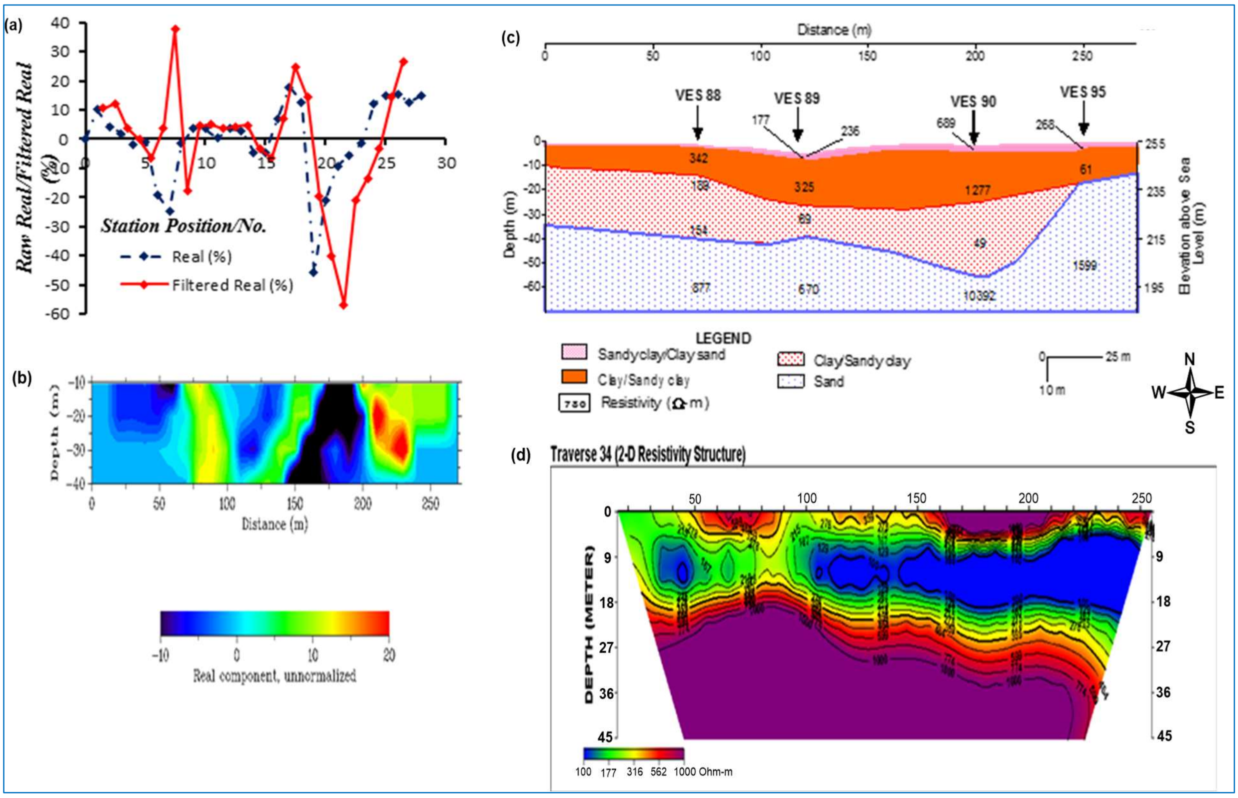

Fig. 13. (a) VLF-EM Real and Filtered Real Components, (b) VLF-EM 2-D Model, (c) Geoelectric Section, (d) Dipole-Dipole Resistivity Structure along Traverse 34 (Okeluse-Ose)

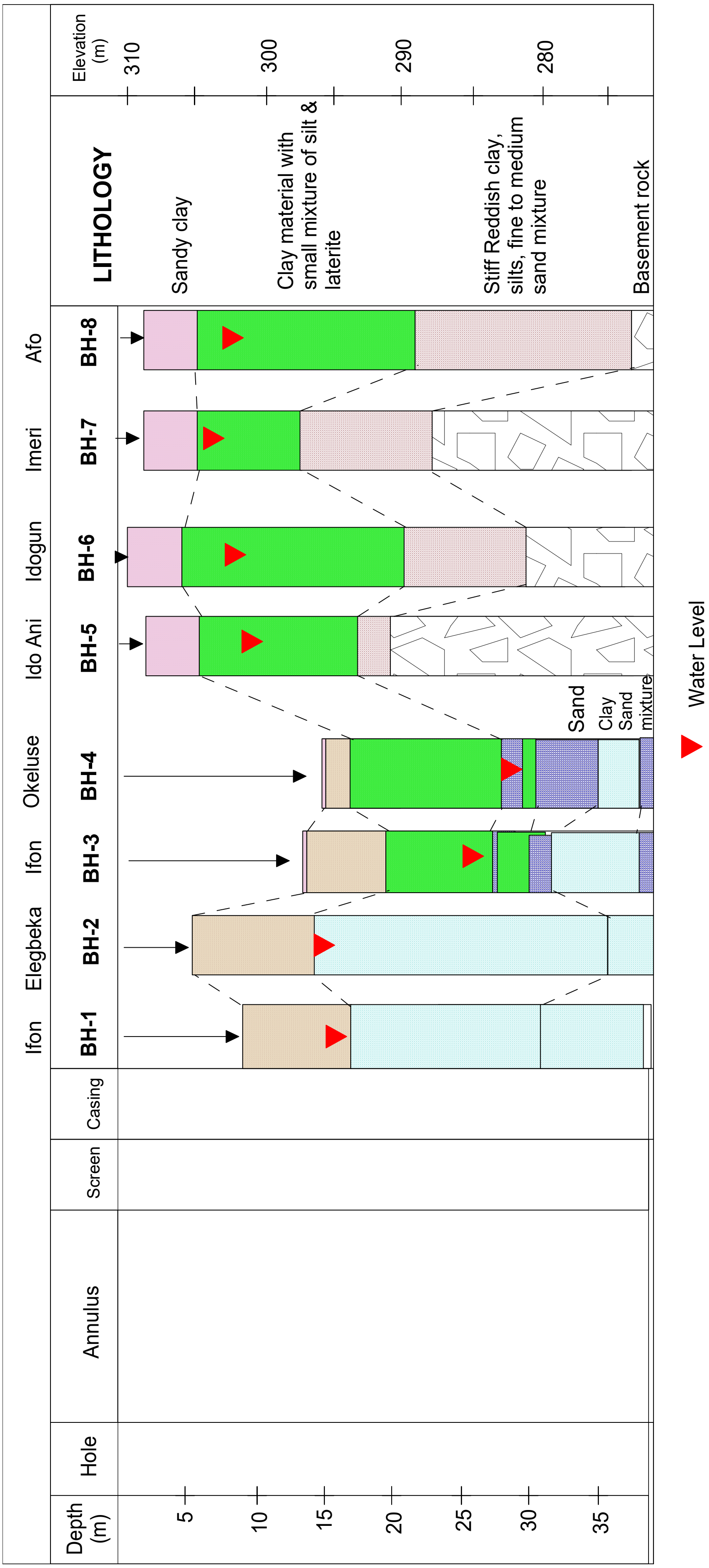

Fig. 14. Borehole logs

4.4. Borehole Sections

The results of the borehole sections generated through the recording of the cuttings ejected during drilling are presented in Fig. 14. Major geological units are delineated, which comprises stiff reddish clay silt – fine to medium sand mixture, sandy clay, clay material with small mixture of silt & laterite, clay sand, sandy clay, laterite, sand, and basement rock. However, in Okeluse and some part of Ifon there's an alternation of clayey and sand material represented by BH-3 and BH-4 respectively. This is generally the case of sedimentary environment (Ojo et al., 2007).

The basement rock was only observed in the northern part of the study area embracing Ido Ani, Imeri, Idogun and Afo at depth not less than 18 m (between 18 and 35 m). However, the borehole sections agree very well with the geophysical and geotechnical results which depicted a clay sand topsoil and clayey subsoil material. From the section, the uppermost 5 m is made up of competent clay sand, sandy clay, and laterite (in some cases hardpan).

5. Conclusion

From the results, it can be concluded that the soils are dominated by clay and sand, with mean % of fines (i.e. 48.4 %) greater than acceptable BS standard of 35%. The soils are accompanied by high liquid limit, plastic limit, plasticity index and linear shrinkage index, of illites (clay mineralogy) which has non-expansive lattices. The soils are moderately cohesive with moderate - high shear strength. The settlement values obtained from the area are generally less than 20 mm from each of the subsurface layers. The minimum settlement values obtained vary from 0.68 mm to 0.80 mm at depth of 0.8 m and 3 m. These depths would be appropriate for the design of foundation structure as they are characterized with low settlement with allowable bearing pressures greater than 100 KN/m². This allowable bearing pressures are considered appropriate for use in the design of bases, strips or raft foundations. Groundwater conditions play an important part in the stability of foundations. If the water table lies very close to the base of footings, the bearing capacity and settlement characteristics of the soil would be affected. Low static water level is observed in clay shale/sandstone lithology, while granite, migmatite and granite gneiss are characterized by high static water level. The hydraulic head map shows the possible groundwater flow direction of south, as the values of hydraulic head in the study area reduce towards the south. Consequently, the SWL in the northern areas are very high indicating a likelihood of high-water levels during the rainy season (which could even lead to spring condition), that may affect basement/foundation footings, and subsequently compromise the integrity of such structures. The geophysical result show KH curve as the most preponderant of all the curve types, therefore the area can be classified as "Class 2" signifying a good subsoil material at a depth below the topsoil. Subsequently, shallow foundation such as pad or raft foundation of reinforced concrete can be adopted in the study area.

Conflict of Interest

The author declares no conflict of interest exist in this publication.

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