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Full Length Research

Geo-environmental engineering site evaluation for shallow foundation design in Akoko area of Ondo State, southwestern Nigeria

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ABSTRACT: Civil engineering structures like buildings, dams, bridges, roads etc. are built or rest on soils, hence the bearing capacity and settlement characteristics of foundation soil to support the structure is important in effective structural design. Consequently, engineering site evaluation was carried out in Akoko area of Ondo State, Nigeria with the aim of investigating the qualitative engineering properties of the subsurface material for shallow foundation design by geological field studies, geophysical, static water level measurement and field/geotechnical laboratory analysis of collected samples. The geotechnical laboratory results show that the soils are of good foundation material with American Association of State Highway and Transportation Officials classification of A-2-4 to A-2-6 and A-2-7 types, with percentage fines less than 35%. The most of VES-curve types are H pattern (which accounts for 58%) which is very resistive at shallow depth. The static water level ranges from 1 to 15 m (generally less than 2 m) while the hydraulic head varies between 300 to 530 m. In the upper 2 m, sediments delineated are clay, silty clay, sandy clay, clayey sand, and basement. All the CPT-plots show the same signature with increasing resistance values with depth. The CPT value of 100 kg/cm² corresponding to 245 KN/m² which signifies moderately competent soil material was obtained at depths of 1.2 to 1.9 m, with immediate settlement values of 0.78 to 21 mm. Consequently, shallow foundation such as simple pad/raft or strip shallow foundation and spread footing are very feasible in the study area. However, substantial additional settlement may occur in the area with high water table, which could exceed tolerable limit and threatens the integrity of the foundation structure.

Keywords: Allowable bearing capacity, borehole lithology, cone penetrometer test, electrical resistivity, hydraulic head.

INTRODUCTION

One of the major environmental/engineering problems prevalent in Nigeria is instability of civil engineering foundation structures (Omali et al., 2010). Therefore, for Nigeria to achieve the set millennium goals/objectives in the area of environmental sustainability and structural stability, this must involve corporate synergy especially government at all levels and private sectors. They must engage in extensive studies using the state-of-the-art equipment. However, movement of rural dwellers to urban areas is not helping the matter, as it puts more pressure on the existing inadequate facilities and utilities. Beside this, in view of the growing demand and competition for space utilization (Akintorinwa and Adesoji, 2009), urban degradation is of major concern in Nigeria (Imasuen and Onyeobi, 2013; Imeokparia and Falowo, 2015).

On this premise engineering site investigation of Akoko area of Ondo State, Southwestern Nigeria was investigated with the aim of characterizing the area sub-soil to support civil engineering structures such as road, buildings, dam etc. Foundation studies usually provide subsurface information that assists builders, town planners, civil engineers in the design and construction of

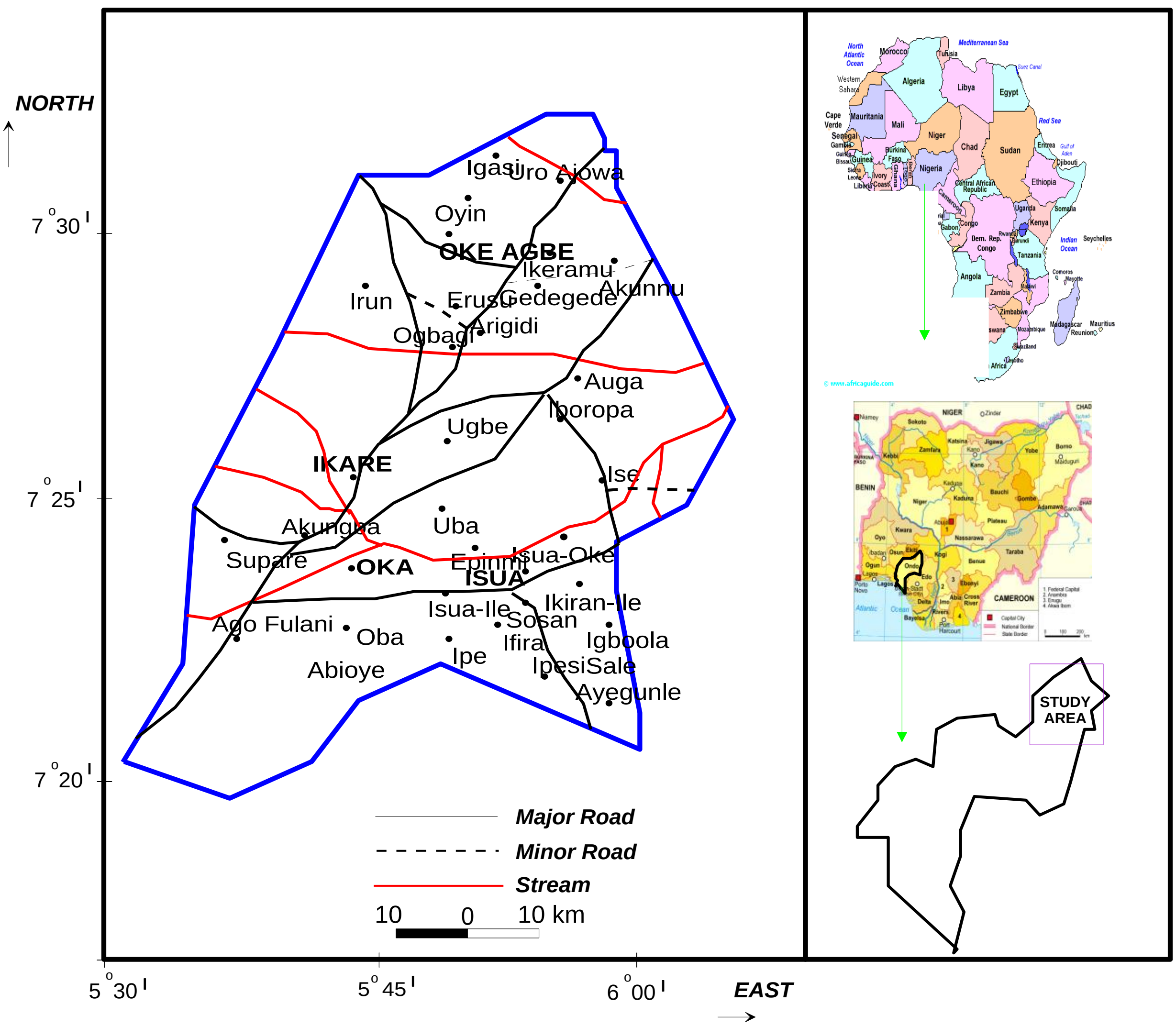

Figure 1. Map of Nigeria showing the study area.

civil engineering structures (Akintorinwa and Adeusi, 2009). The study area lies within the southwestern part of Nigeria (Figure 1). It is located within longitudes $5°30^1$ and $6°10^1$ and latitudes $7°20^1$ and $7°32^1$. The elevation of the area ranges between 305 and 610 m above the mean sea level (Figure 2).

Akoko area forms part of the Precambrian Basement Complex of Southwestern Nigeria, comprising mainly of the Migmatite-Gneiss-Quartzite Complex which is of Archean – Proterozoic age (>2Ga) (Annor, 1995; Dada et al., 1998), the Late Proterozoic Schist Belts (Turner, 1983; Fitches et al., 1985) and the Older Granitoids of Pan-African age (500-750Ma) which intruded the former two units (Rahaman, 1976; Ajibade, 1982; Ekwueme, 1990). The area comprises mainly of gneisses in association with other rock types which include porphyritic granite, pegmatite, aplite, quartz vein and amphibolite. The gneisses are of two types: granite gneisses and grey gneisses. Rahaman (1976) and Rahaman and Ocan (1988) referred to the grey gneisses as early or quartzo-feldspathic gneisses and explained that they are granodioritic to quartz-dioritic or tonalitic in composition.

The granite gneiss forms part of the felsic components of the Migmatite-Gneiss complex (Rahaman, 1976). The grey gneisses are the second most abundant rock type in the area forming enclaves within the granite gneisses. They are dark grey in colour and are medium-coarse grained with well-developed mineralogical bands. The light coloured bands are quartzo-feldspathic while the dark bands are biotite-rich. The grey gneisses contain intrusions of pegmatites and quartzo-feldspathic veins and are regarded as the oldest rocks in the area into which all other rocks in the area were intruded.

The granite gneisses are light coloured, medium-coarse grained and characterised by weak foliation defined by the alignment of streaks of light and dark coloured minerals.

Figure 2. Elevation Map of Nigeria, showing the topographical variation between 305 and 610 m above the mean sea level.

The granite gneisses contain xenoliths of the grey gneisses and amphibolites. This indicates that the granite gneisses post-date the grey gneisses in the study area. These gneisses (grey and granitic varieties) are widespread in the area constituting about 90% of the rock types found in the area and have been intruded by the Pan-African granites. They occur as massive rugged hills and rolling plains assuming batholithic dimensions and forming impressive outcrops which tower few hundred metres above the surrounding lowlands and showing different types of geological structures such as folds, faults, foliation, joints, veins, etc. These structures show that the area has been subjected to at least two phases of deformation. Metamorphism in this area is of granulite facies grade (Rahaman and Ocan, 1988).

MATERIAL AND METHOD

The scope and method of investigation includes geological field studies, geophysical, static water level measurement, geotechnical and laboratory analysis of collected soil samples. Eight soil samples were collected from pits dug in different locations within the study area (Figure 3). The samples were collected from each of the weathering profile from top to the depth of about 3.0 m with the aid of shovel and a digger. Typical soil unit in the area is shown in Plate 1. Consequently, the samples were labelled as AK-1, AK-2, AK-3, AK-4, AK-5, AK-6, AK-7, and AK-8 representing samples 1 to 8 respectively. The samples collected were air-dried and disaggregated for geotechnical tests which include grain size analysis, specific gravity, plasticity indices, compaction, triaxial compression test and linear shrinkage determination which were conducted at the Engineering Geology Laboratory, Federal University of Technology Akure, Ondo State, Nigeria.

The geophysical data were acquired by adopting very low frequency electromagnetic (VLF-EM) and electrical resistivity methods. Specifically, the vertical electrical sounding (VES); and combined vertical electrical sounding and horizontal profiling techniques were adopted for the electrical resistivity method using Schlumberger and dipole-dipole configurations respectively.

The VLF-EM survey was conducted in the area at an interval of 10 meters along 13 traverses using ABEM WADI VLF receiver. Meanwhile, forty-three (43) VES were occupied across the study area at 20 to 50 m inter-VES station. The electrode spacing was varied between 1 to 65 m. The Ohmega resistivity meter was used to acquire the field data and the position of the occupied sounding stations in Universal Traverse Mercator (UTM) was recorded using the GARMIN ’12 channel personnel

Plate 1. Exposed soil unit along Iboropa – Ise Highway.

navigation Geographic Positioning System (GPS) unit. The hydrogeological measurements were taken from thirty-four open-wells and eight boreholes, from which static water level were determined and hydraulic head calculated. In addition, eight boreholes were drilled in the study area to the basement rock/competent soil.

Eight (8) Cone Penetrometer Tests (CPT) using Dutch Cone Penetrometer which measures the resistance of penetration into soils using a 60% steel cone was carried out. The cone has an apex angle of 60° and a base area of 10.2 cm². 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 was 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 using Microsoft Office Excel 2007. The layer sequences were interpreted from the variation of the values of the cone resistance with depth.

Consequently, calculated allowable bearing capacity was obtained using Meyerhof (1974) equation which covers all foundations irrespective of the width:

$q_a = 2.7q_c$ (KN/m²) (1)

where q_a was the allowable bearing capacity and q_c was 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 sub-soil for foundation footing width of 1.5 m:

$$s = {}^{qB}/_{2q_c}$$

where s = settlement, q = Net foundation stress, B = Footing width and q_c = Cone CPT value

RESULTS AND DISCUSSION

Geological field observation

Granite and granite gneiss

Granite is observed predominantly in Akoko area, and some parts of Owo especially Ogbese, Amurin and Emure-Ile. They are usually found in association with granite-gneiss. They exhibit varying structural and textural characteristics (Plate 2). Some are fine grained (Plate 2a). Lenses, quartz veins and feldspathic veins of varying length (could be more than 15 m in places) are also observed in the study area. They show great irregularities in their forms. Plates of muscovite and at times biotite may be found associated with large quartz veins. Some of the veins occur conformably within their host rock and have been involved in the tectonism affecting the host while most are structureless, discordant with respect to the host. The rocks are essentially rich in quartz, alkaline feldspar, and very common mica (muscovite and/or biotite). A number of accessory minerals are also included such as aplite, zircon and magnetite. The granite is generally coarse grained in texture.

The granite gneiss observed showed mafic and felsic gradation/alternation (Plate 2d). The mafic bands may be early gneiss and/or mafic to ultramafic rocks, and felsic bands are granitic. Quartz, biotite, plagioclase feldspar and orthoclase feldspar are the most common minerals observed in the granite gneiss rocks of the study area. Along with the lithology, the structural features of the granite and granite-gneiss like veins, joints, fractures, folding which can modify the engineering properties of the rock to a great extent (by rendering the rock weak and enhance permeability) are sparsely developed and of a close nature. Therefore, the massive granite/granite-gneiss may be classified as the most excellent category of the rocks studied, which showed more compaction, and less fractures. This implies that it is likely they have high compressive strength, shear strength, and modulus of elasticity, which might be responsible for stability of structures in Akoko area. The engineering performance of gneiss usually is similar to that of granite (Bell, 2007). However, the gneiss in the study area are strongly foliated, and a texture with preferred orientation.

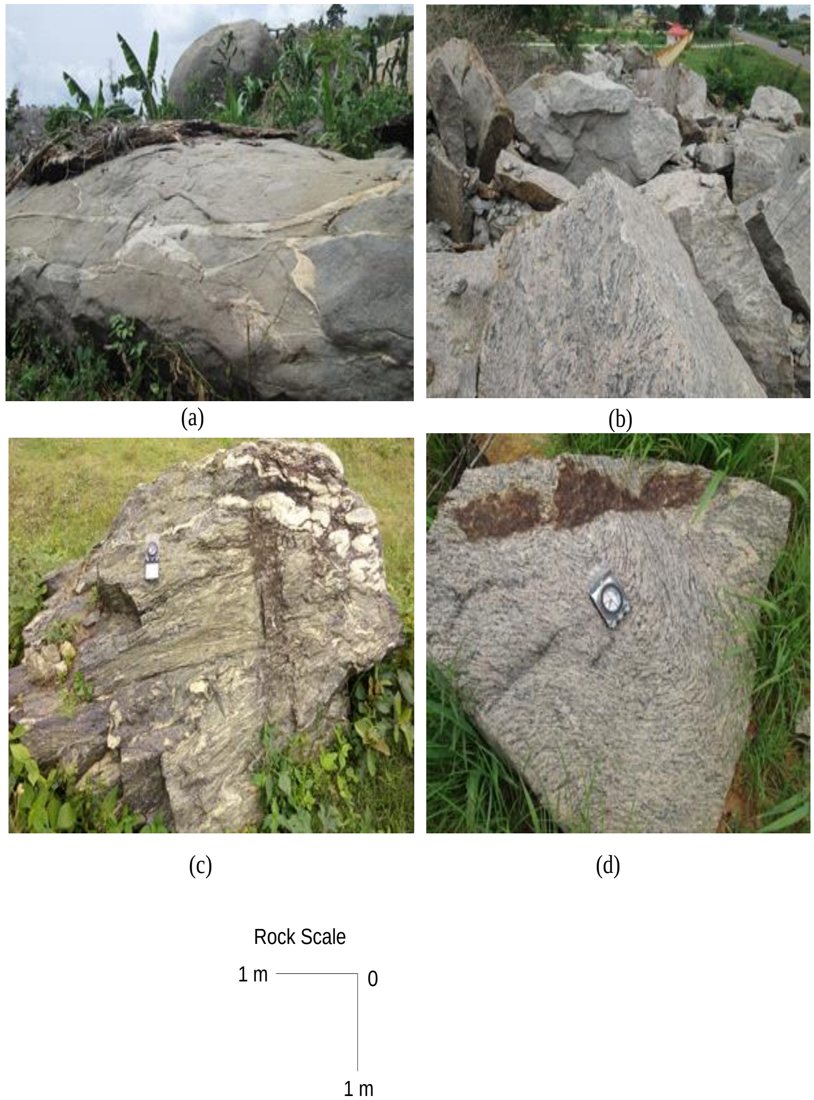

Plate 2. (a) Massive Granitic rocks observed in Iwaro-Oka showing some quartzo-feldspathic intrusion (b) Granite gneiss (c) Isolated intensely weathered Gneiss (d) Gneiss showing micro-folded alternation of felsic and map mineral alternation.

Migmatite

Migmatite – Gneiss Complex is the most widespread of the component units in the Nigerian basement. It consists of variably migmatised, undifferentiated biotite and biotite hornblende gneiss with intercalated amphibolites. It has a heterogeneous assemblage comprising migmatites, orthogneises, paragneisses, and a series of basic and ultrabasic metamorphosed rocks. Petrographic evidence indicates that the Pan-African reworking led to recrystallization of many of the constituent minerals of the Migmatite – Gneiss Complex by partial melting with majority of the rock types displaying medium to upper amphibolite facies metamorphism. The Migmatite – Gneiss Complex has ages ranging from Pan-African to Eburnean (Obaje, 2009). Migmatite gneiss/biotite granite is the most common rock type in the study area (Plate 3). The minerals observed in the rock are quartz, biotite, hornblende and plagioclase feldspar. Fractures, occurring as block joints/columnar joints, quartz and feldspathic intrusion, and folds (Plate 4) are commonly observed.

Weathering leads to decrease in density and strength, and increase deformability of rocks (Bell, 2007). The rocks are highly weathered in some areas. The degree of resistance that rock offers to weathering and other deformational agents depends on its mineralogical composition, texture, porosity, amount and type of cement/matrix, and the presence of any planes of weakness (Imasuen and Onyeobi, 2013).

The migmatite in the area have undergone intense

Plate 3. Surface Exposures of some Migmatite / Migmatite Gneiss Rocks observed in the study area showing high degree of fracturing showing columnar jointings/block joints, quartz vein and quartzo-feidspathic intrusions of different sizes.

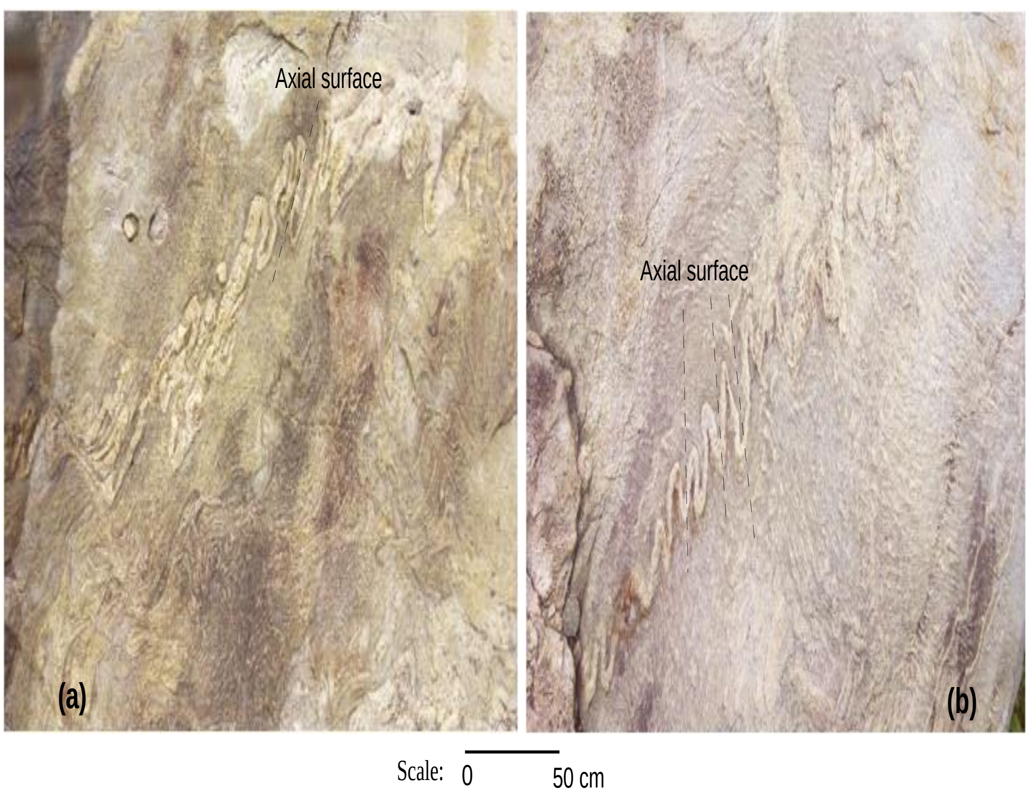

Plate 4. Folds observed on migmatite exposure in Supare (a) Asymmetrical Fold (b) Symmetrical Fold.

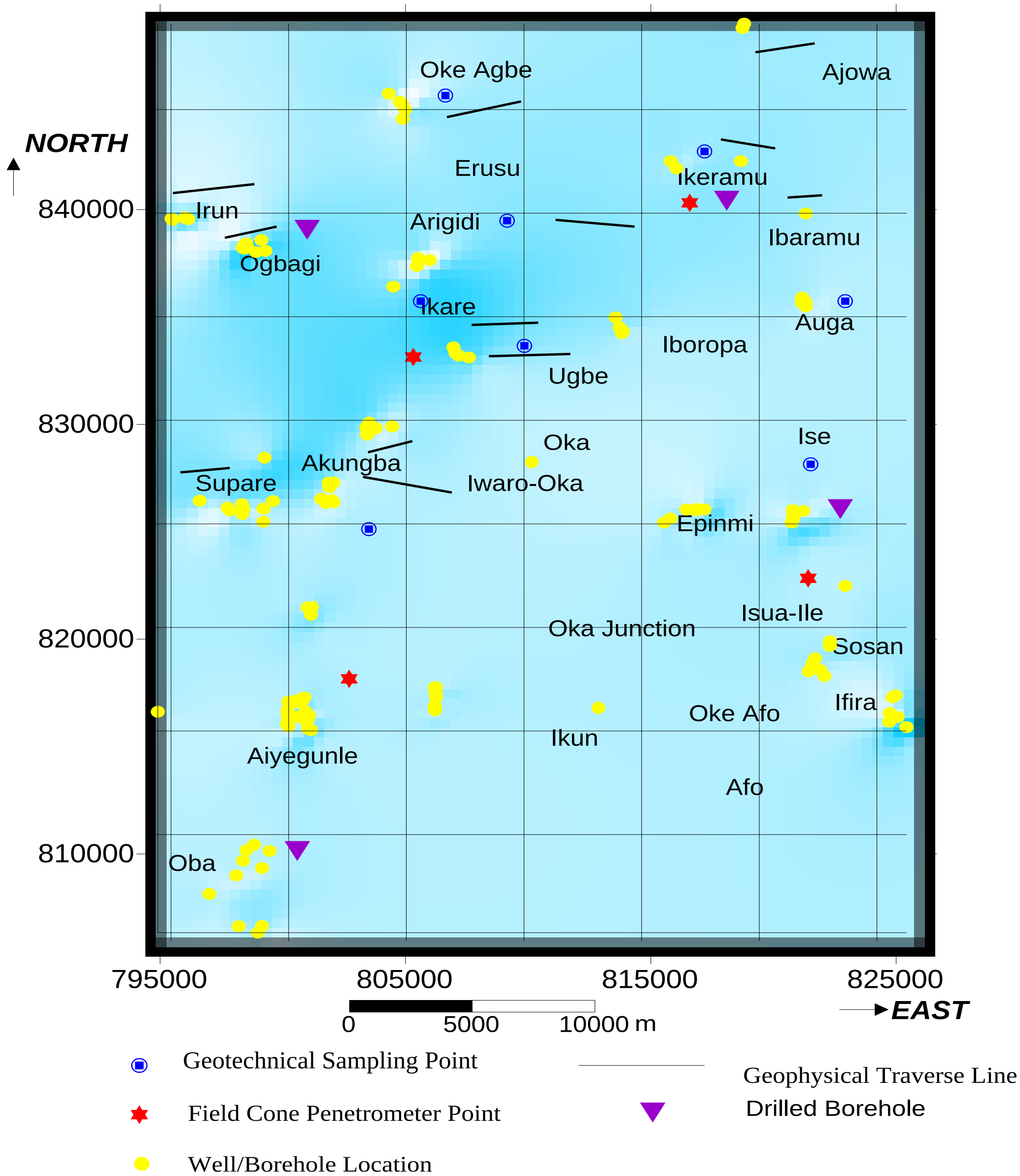

Figure 3. Base map for the investigation.

deformation resulting into joints, fractures, and faults. These structural features reduce the engineering capability/stability and competence of the rocks as foundation material by increasing porosity. Therefore, adequate safety-steps must be taken to ameliorate or remedies these zones of weakness in order to ensure safety of civil engineering structures and utilities to be built on the rocks.

Geophysical investigation

Table 1 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 four (4) layers. Four curve types have been identified: A, H, HA, and KH (Table 1). The most occurring curve types identified is H (58%). The root mean square (RMS) error of the generated curves ranges between 1.8 and 6.1. These types of curves can be classified into three distinct classes in relation to their engineering competence by using their interpreted resistivity and thickness values as: A-curve type is rated “Very Good”, KH and H are rated “Good”, while HA is rated “Fair” (Table 1). Therefore, the area can be regarded or rated as “Good” for civil engineering construction.

The VLF-EM profiles/model, geologic section, and resistivity structure along Traverse 22 are shown in Figure 4. A conductive steeply dipping (NE – SW direction) linear feature is delineated on the VLF – EM model at distances between 200 to 250 m. However, this linear feature is not

Table 1. Curve types and their statistical frequency obtained from the study area.

Curve Type	H	HA	KH	A
Frequency (unit)	25	9	1	8
Frequency (%)	58	21	2	19
Stability (%)	50	40	50	70
Rating	Good	Fair	Good	Very Good

Table 2. Summary of Geotechnical Results.

Sample No.	G. S. D				Consistency Limits				Compaction			AASHTO Group	Rating
	N.M.C (%)	% Fines	% Sand	% Gravel	S. G	L.L (%)	P.L. (%)	P.I (%)	L.S (%)	OMC (%)	MDD (Kg/m³)		
AK 1	16.4	16.5	55.6	-	2.72	37.2	22.4	14.80	10.1	14.2	1910	A-2-6	Good
AK 2	12.2	16.1	64.7	-	2.73	30.0	22.2	7.85	12.0	12.1	1988	A-2-4	Good
AK 3	12.7	16.5	51.4	5.5	2.72	41.2	23.2	18.00	9.6	15.0	1879	A-2-7	Good
AK 4	9.3	14.5	70.9	2.1	2.73	27.0	23.3	3.70	10.4	11.1	2027	A-2-4	Good
AK 5	17.4	17.4	51.4	1.3	2.73	47.2	23.4	23.80	8.7	16.0	1824	A-2-7	Good
AK 6	16.2	20.9	49.7	2.2	2.70	48.4	23.7	24.75	8.7	16.5	1804	A-2-7	Good
AK 7	14.4	17.2	51.3	2.5	2.70	40.8	23.4	17.45	9.6	14.8	1885	A-2-6	Good
AK 8	8.2	15.1	68.7	3.3	2.69	27.9	11.0	6.85	11.0	11.8	1988	A-2-4	Good

NMC- Natural Moisture Content; GSD- Grain size distribution; S.G.- Specific gravity; L.L.- Liquid Limit; P.L.- Plastic Limit; P.I.- Plasticity Index; L.S.- Linear Shrinkage; OMC- Optimum Moisture Content; MDD- Maximum Moisture Content.

identified on the geologic section and dipole-dipole resistivity structure. Consequently, using the resistivity values, both the topsoil and weathered layer are identified as clayey along this traverse with resistivity values ranging from 34 to 119 Ω-m. However, the topsoil under VES 49 shows fairly competent sub-soil (119 Ω-m) and appreciable thickness of 6.3 m, hence suitable for foundation construction.

Along Traverse 24, multiple low amplitude filtered real positive peaks are shown on the filtered real profile (< 8%). The geoelectric section (Figure 5c) and 2-D resistivity structure (Figure 5d) three geoelectric sequence comprising the topsoil, weathered layer and fresh basement. On the resistivity structure, the basement rock nearly outcrops at distances between 100 to 110 m. The topsoil is very thin (< 1.5 m), in fact almost unrepresented on the geoelectric section. The topsoil resistivity ranges between 164 and 429 Ω-m, and thickness of 0.4 to 5.4 m, hence, moderately competent to host structure.

Traverse 26 is characterized by an almost vertical strongly conductive linear feature on the 2-D model of the VLF-EM. The linear feature is shown as relatively thick clayey material on the geoelectric section and 2-D resistivity structure (Figures 6c and d). Subsequently, the topsoil with resistivity range of 53 to 327 Ω-m and the weathered layer of 27 to 167 Ω-m are generally composed of clay/sandy clay which is rated as poor/fair competent material for foundation construction of building, road etc. Therefore, some degree of stabilization/improvement is required on the soil. However, the fracture zones must be avoided. The groundwater direction follows the surface hilly topographical elevation.

Geotechnical investigation

The summary of the laboratory analysis and cone penetrometer field test are presented in Tables 2 to 4 respectively. The natural moisture content of the samples ranges from 8.2 to 16.4%. This shows that the moisture content of the soil in the area is relatively low in their natural state. Moisture variation is generally determined by intensity of rain, depth of collection of sample and texture of the soil (Jegede, 2000). Samples AK-4 and AK-8 show the lowest moisture content value, which could be as a result of increased porosity and permeability as the soils could not retain appreciable amount of water. This is because unconsolidated soils loose moisture very quickly on exposure to heat as a result of little or no matrix. Low water retention, high porosity and permeability make the soil to be easily washed away; a condition that favours gully erosion development. The tested soils show percentage fines (percentage passing 0.002 mm) variation of 14.5 to 20.9%. The percentage of sand and gravel in the sampled soils ranges between 49.7 and 68.7% and 1.3 to 5.5% 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 soils are less than 35%

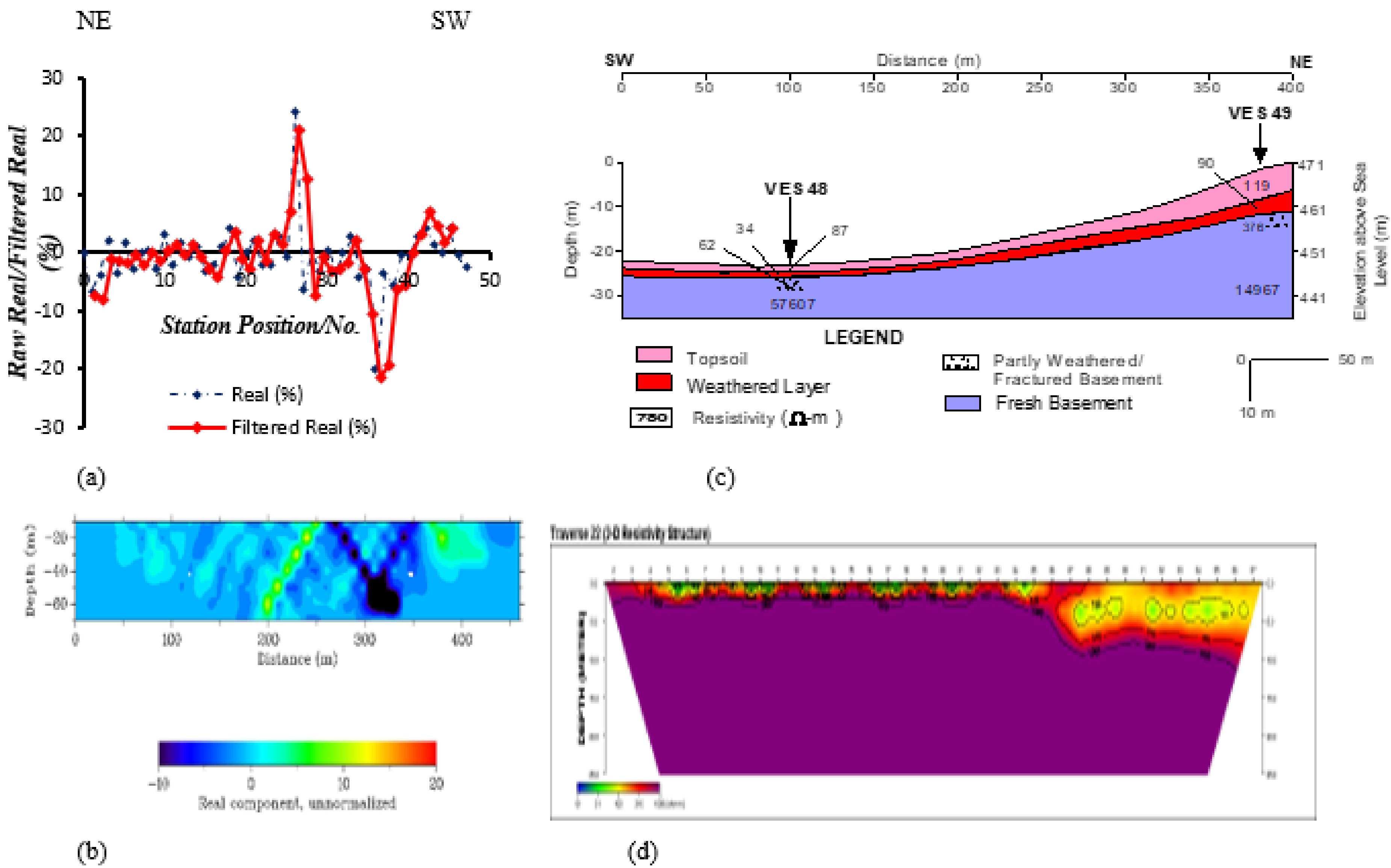

Figure 4. (a) VLF – EM Real and Filtered Real Components (b) VLF – EM 2-D Model (c) Geoelectric Section (d) Dipole – Dipole Resistivity Structure along Traverse 22 (Ikare – Ugbe Road, Akoko).

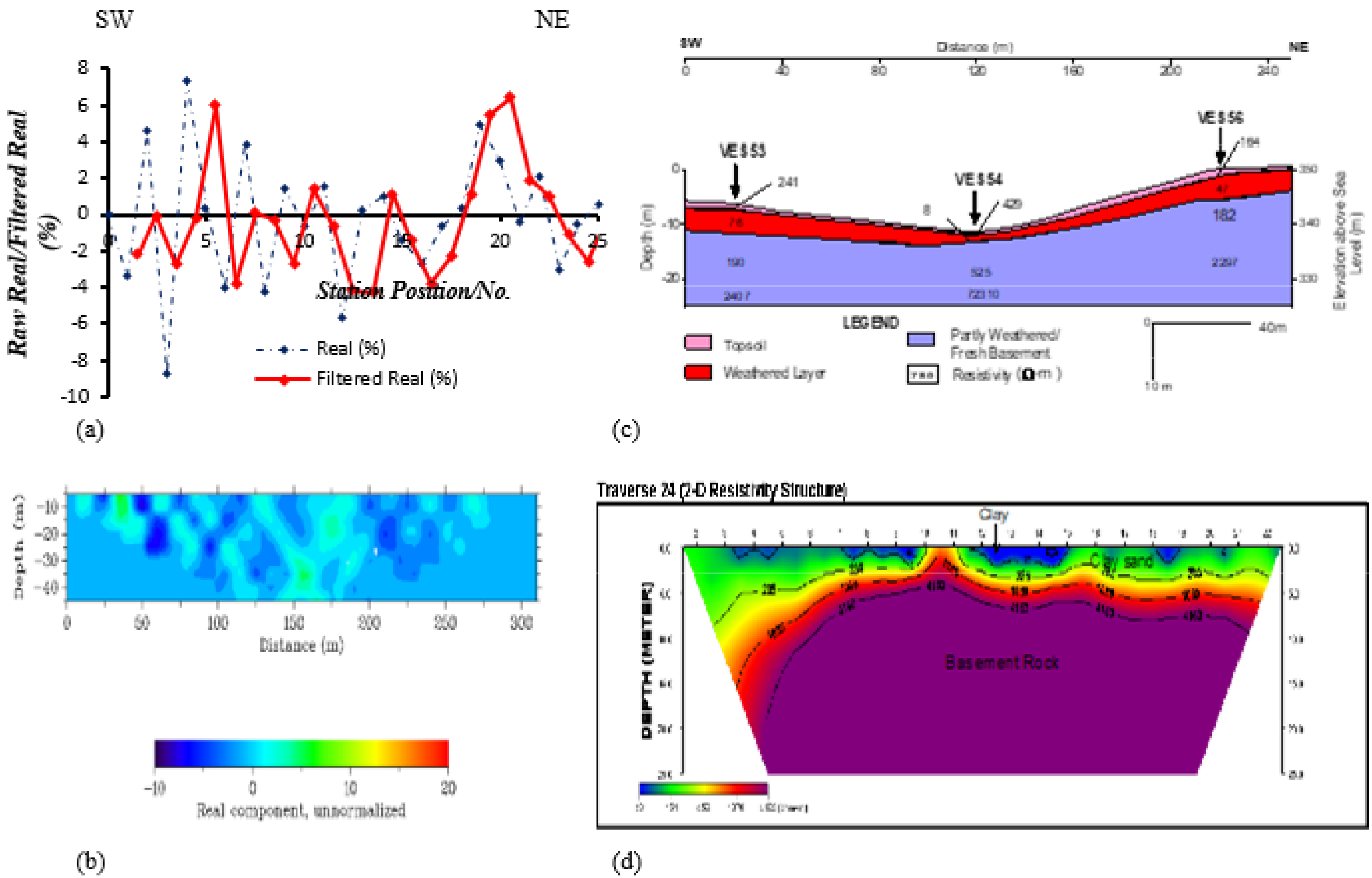

Figure 5. (a) VLF – EM Real and Filtered Real Components (b) VLF – EM 2-D Model (c) Geoelectric Section (d) Dipole – Dipole Resistivity Structure along Traverse 24 (Akungba – Akoko).

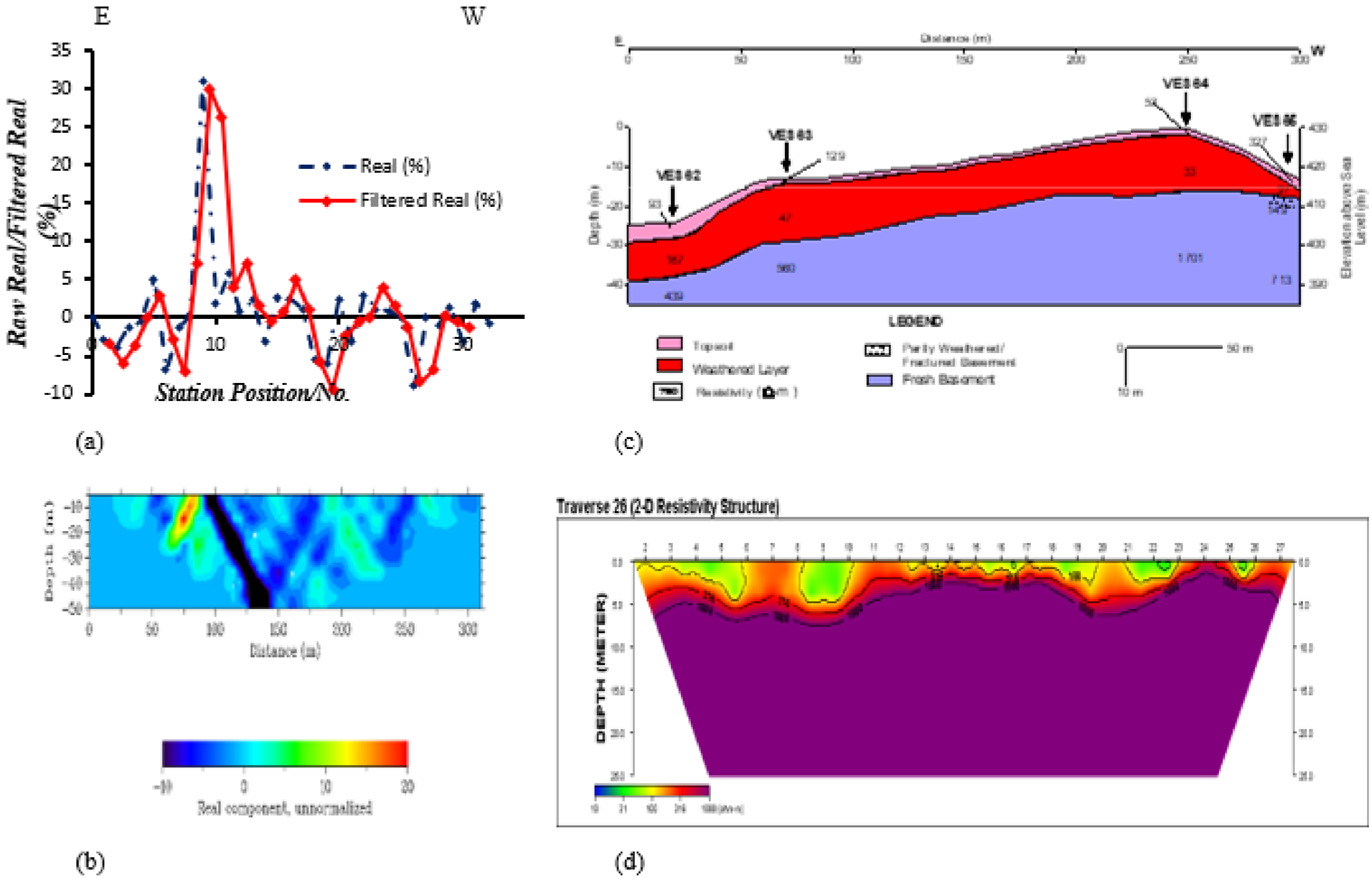

Figure 6. (a) VLF – EM Real and Filtered Real Components (b) VLF – EM 2-D Model (c) Geoelectric Section (d) Dipole – Dipole Resistivity Structure Along Traverse 26 (Irun – Akoko).

Table 3. Results of the Quick Undrained Triaxial Test.

Sample No.	(σ) Deviator stress at different cell pressures (Kpa)			Cohesion (c) (Kpa)	(θ °) Angle of Friction	Undrained compressive strength @ Max. Cell Pressure (Kpa)
	30	60	90			
AK 1	337	380	422	94.7	22.5	422
AK 2	296	360	425	65.3	27.3	425
AK 3	323	381	425	82.9	24.6	425
AK 4	269	339	419	51.5	29.1	419
AK 5	337	384	431	90.4	23.7	431
AK 6	337	382	427	92.3	23.2	427
AK 7	395	459	523	93.2	27.3	523
AK 8	395	455	515	96.5	26.6	515

Table 4. Summary of the CPT obtained in the study area.

Depth (m)	CPT 5 (OBA)			CPT 6 (IKARE)			CPT 7 (IKERAMU)			CPT 8 (ISUA)		
	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	22	66	9.78	10	30	21.00	61	183	3.44	11	33	19.57
0.4	20	60	10.50	22	66	9.67	39	117	5.38	25	75	8.36
0.6	33	99	6.43	52	156	4.06	21	63	10.01	54	162	3.90
0.8	43	129	4.86	82	246	2.59	54	162	3.90	121	363	1.75
1.0	53	159	3.99	163	489	1.29	125	375	1.69	162	486	1.30
1.2	73	219	2.91	195	585	1.08	153	459	1.38	207	621	1.02
1.4	82	246	2.59	249	747	0.85	197	591	1.07	248	744	0.85
1.6	109	327	1.94	-	-	-	245	735	0.86	-	-	-
1.8	173	519	1.22	-	-	-	-	-	-	-	-	-
2.0	270	810	0.78	-	-	-	-	-	-	-	-	-

q_a - Allowable Bearing Capacity; q_u – Ultimate Bearing Capacity; S – Settlement.

Table 5. Soil Classification according to Shrinkage Limit (Murthy, 2007).

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

maximum specified for subgrade/foundation material for civil engineering structures based on British Standards Institution BSI 1377 (1990). According to AASHTO (1982) classification, the soils are of A-2-4 to A-2-6 and A-2-7 types which are regarded as "good" soil material.

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. The results of the specific gravity (*Gs*) for all the soil samples and vary between 2.69 to 2.73. These ranges of values portray resistant soil material (Brink et al., 1982). The liquid limit, plastic limit, and plasticity index of the soil samples vary from 27.0 to 48.4%, 11 to 23.7%, and 3.7 to 24.8% 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 low (Okogbue, 1985; Okogbue and Ene, 2008). Therefore, low liquid limit (less than 50%), plastic limit (less than 30%) and plasticity index (less than 20%) are indicative of good engineering properties (FMWH, 1972; Terzaghi et al., 1996) as depicted in all the soil samples.

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; Madedor, 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 8.7 and 12.0%. Using Table 5, the soils can be classified as medium good material. The Maximum Dry Density (MDD) of the studied soils ranges from 1804 Kg/m³ at Optimum Moisture Content (OMC) of 11.1% to 2027 Kg/m³

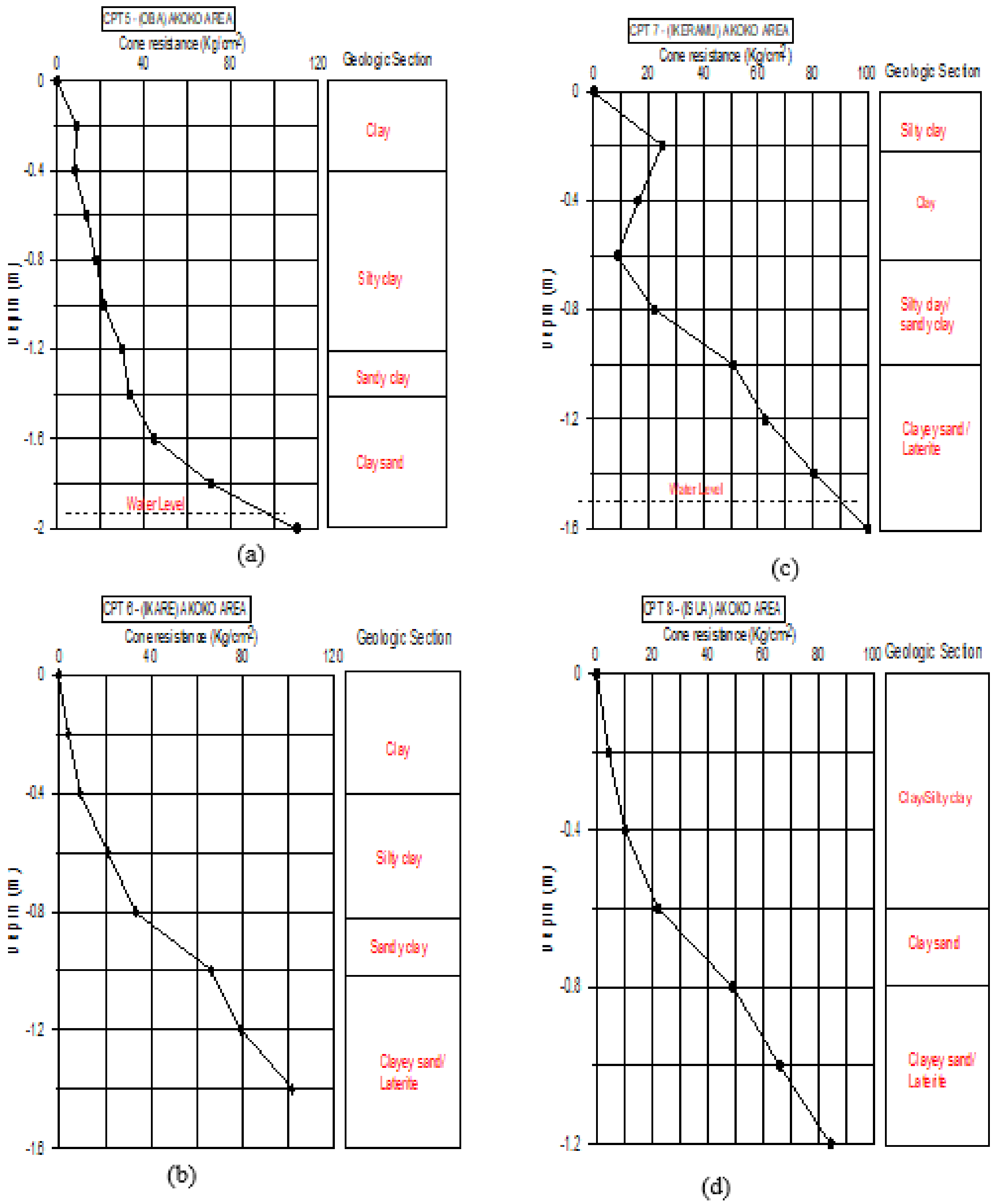

Figure 7. Plots of cone resistance with depth in Akoko area (a) Oba (b) Ikare (c) Ikeramu (d) Isua, with minimum cone resistance value of 100 Kg/cm² for competent subsoil observed to be between 1.2 and 1.9 m.

at OMC of 16.5%. 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 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.

Cohesion is a function of silt or clay fraction in soil. Fines content are known to contribute to the strength of a soil as their small sizes and mineralogy increase the bond between the grains (Das, 2011). The fewer fines a soil has, the greater the ease with which moving water can detach the particles. The cohesion of the studied soils varies between 51.5 to 96.5 Kpa. The values of angle of friction

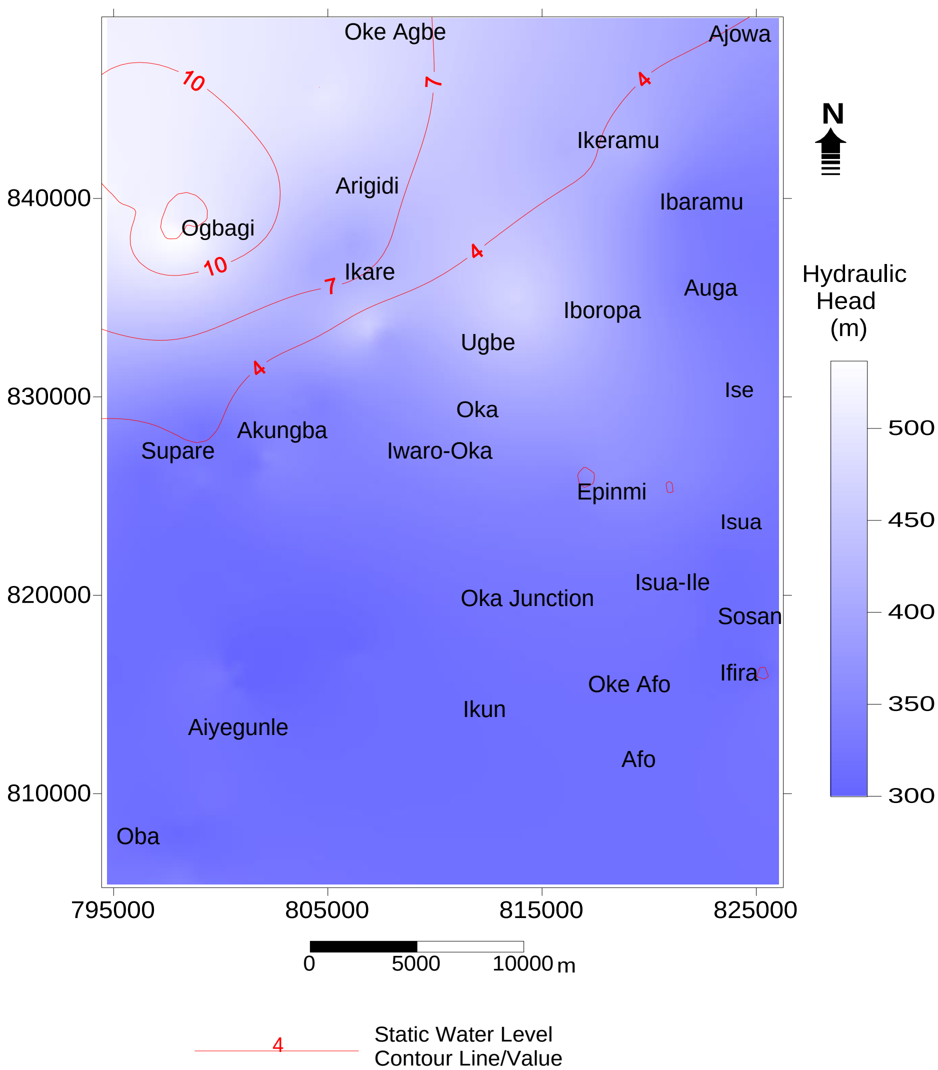

Figure 8. Hydraulic head and static water level map of the study area.

are between 22.5° and 29.1°. These ranges of values are classified as hard soil material by Holtz and Kovacs (1981). These values indicate moderately cohesive material with moderately high shear strength. Also, these ranges of values make the soil samples moderately competent to accommodate civil engineering foundation structures, especially shallow foundation at a depth of 3 m (which is sample depth). The undrained compressive strength at maximum cell pressure (90 Kpa) varies from 419 to 523 Kpa. Figure 7 shows the plots of cone resistance with depth in the study area. 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, groundwater table was encountered at CPT-5 and 7 (in Ikeramu and Oba) at depths of 1.5 and 1.9 m respectively.

The range of water levels observed during penetration agrees with shallow water level measured in Figure 8. In the upper 2 m, sediments delineated are clay, silty clay, sandy clay, and clay sand (Figure 9). All the plots show the same signature with increasing resistance values with depth. The CPT value of 100 kg/cm² corresponding to 245 KN/m² which signifies moderately competent soil material was obtained at depths of 1.2 to 1.9 m. The allowable bearing capacity ranges from 10 to 270 KN/m², with settlement values of 0.78 to 21 mm. The settlement values are below the 25 mm maximum specified by Bell (2007) and Shahin et al. (2002). The result of the measurements taken from the open wells and boreholes in the study area is presented in Figure 8. The static water level ranges from 1 to 15 m, while the hydraulic head varies between 300 to 530 m. Generally, the area is characterized by low static water level (less than 2 m), except in Irun, Ogbagi, Arigidi, and part of Ikare which have high water levels greater than 5 m. It is also observed that many of the areas with high hydraulic head values are also characterized by high static water level. The hydraulic head map (Figure 8) shows the possible groundwater flow direction is northwest. However,

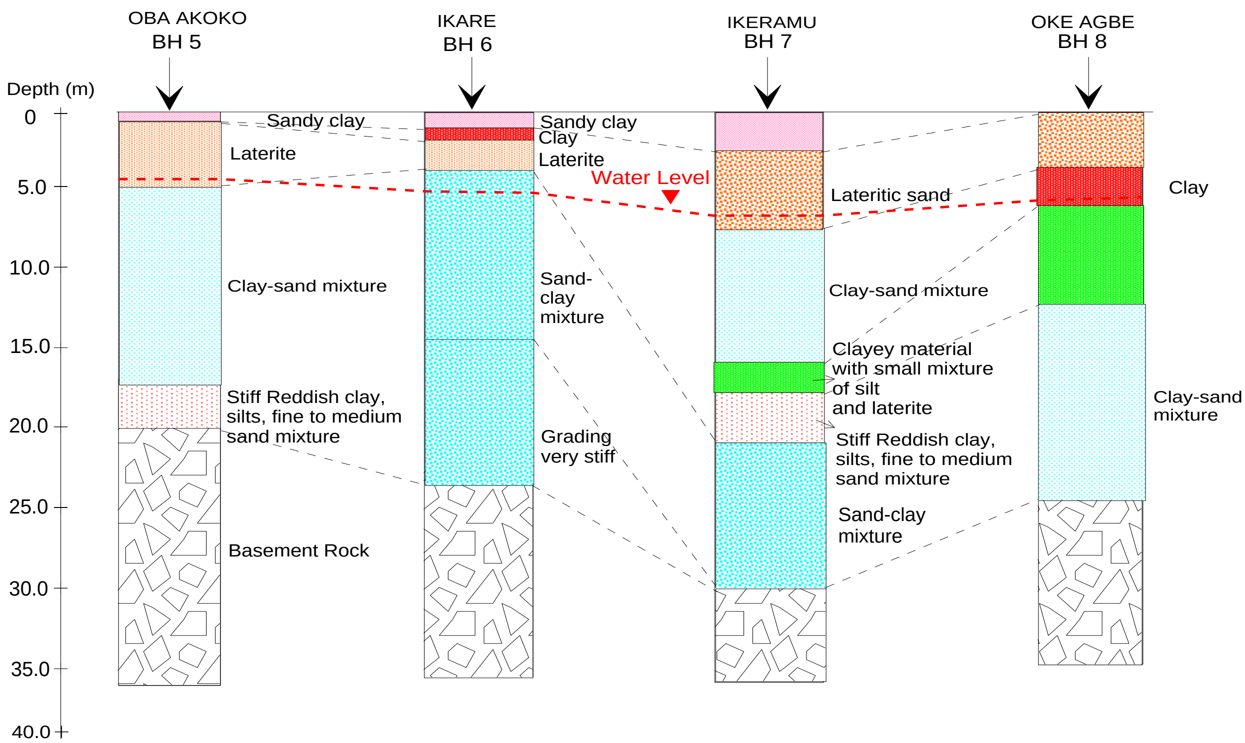

Figure 9. Borehole Sections obtained from the records of the cuttings during drilling.

substantial additional settlement may occur in the area due to low static water level values/high water table, which could exceed tolerable limit and threatens the integrity of structures. Consequently, shallow foundation such as simple pad/raft or strip shallow foundation and spread footing are very visible in the study area.

The results of the borehole sections generated through the recording of the cuttings ejected during drilling are presented in Figure 9. The depth range of the boreholes are between 30 to 35 m. Major geological units are delineated, which comprises sandy clay, clay sand, clay, laterite, sand, and basement rock. From the sections, the uppermost 5 m is made up of competent clay sand, sandy clay, and laterite (in some cases hardpan). The clay-sand mixture dominated the subsoil which graded stiffly as drilling continues to a depth of 30 m in some places (BH-7). The result of the borehole drilling also shows that the area has relatively moderate-high overburden thickness which would assist in evenly distribution of an imposed load to the bedrock. This thickness can also accommodate utilities/materials such as electrodes, communication masts and gadgets, transmitter, cables and wires, water reticulation pipes. The groundwater levels of the drill wells in the area are generally less than 10 m.

Conclusion

Granite gneiss and Migmatite are most common rocks in the area. Most of the rocks have undergone intense deformation resulting into joints, fractures, and faults. These structural features reduce the engineering capability/stability and competence of the rocks as foundation material by increasing porosity. Therefore, adequate safety-steps must be taken to ameliorate or remedies these zones of weakness, in order to ensure safety of civil engineering structures and utilities to be built on the rocks.

Therefore, the massive granite/granite-gneiss may be classified as the most excellent category of the rocks studied, which showed more compaction, and less fractures. This implies that it's likely they have high compressive strength, shear strength, and modulus of elasticity. The engineering performance of gneiss usually is similar to that of granite. The dominant curve type in the study area is H-curve which accounts for 58% of the total. Three major geological layers were delineated comprising the topsoil, weathered layer, and partly weathered/fracture basement/fresh basement. The depth to basement rock is predominantly between 5 to 20 m. Also, most of the studied soils fall within the Federal Ministry of Works and Housing specification of subsoil engineering material. The soil samples exhibit moderate to high shear strength parameters. The CPT value of 100 kg/cm² corresponding to 245 KN/m² which signifies moderately competent soil material was obtained at depths of 1.2 to 1.9 m. The allowable bearing capacity ranges from 10 to 270 KN/m², with settlement values of 0.78 to 21 mm. The settlement values are below the 25 mm maximum. This allowable bearing pressure of 245 KN/m² is considered appropriate for use in the design of bases, footings, strips or raft foundations in the study area.

CONFLICT OF INTEREST

The authors declare that no conflict of interest exists.

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