Subsoil Engineering Characterization and Foundation Design in Derived Geological Coastal Sands of Ilaje Area of Ondo State, Nigeria

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Abstract **Engineering subsoil evaluation and foundation design have been undertaken at Ilaje area of Ondo State, Nigeria. The aim of this study was to examine the geo-electrical and geotechnical parameters of the subsoil to sustain building structures and also provide appropriate foundation design alternatives. A total of six VES stations were occupied and complemented with geotechnical analysis of seven soil samples collected at two cone penetration test locations. The result showed that all the determined geotechnical parameters of the subsoil fall within the specification recommended for foundation material by federal ministry of works and housing of Nigeria. The VES showed a predominant (66.67%) HKQ curve type. The upper 5 m is characterized by moderate thickness and high resistivity (average of 450ohm-m) values to sustain structural load. An average allowable bearing capacity of 150 KN/m^2 was recommended for design of bases/footings for shallow foundation at a depth not less than 1.0 m. The obtained settlement values are less than 50 mm and within tolerable limit, for foundation width ranging from 0.5 to 3.0 m. However there was drastic reduction in settlement values (below 25mm) when the foundation width was increased to 2 and 3m. The allowable capacity of the driven pile ranges from 64 – 115 KN, 206 – 347 KN, and 418 – 677 KN at 5m, 10 m and 15 m respectively. The allowable bearing capacity for bored piles ranges from 34 – 69 KN, 85 – 165 KN, and 146 – 268 at depth levels of 5 m, 10 m, and 15 m respectively.**

Keywords — CPT, Ilaje, square and strip footing, structural load, **bearing capacity,** *vertical electrical sounding*

I. INTRODUCTION

The stability of engineering structures depends on the support offered by the foundation materials that bear the load of the structure [1]. The ability of foundation materials to offer necessary bearing support for engineering structures depend on the bearing capacity of the soil and/or rock [2]. Geological and structural factors such as mineralogical composition, rock fabric, rock association, degree of weathering, fluid saturation, rock deformation and occurrence of joints and faults would not only define the bearing strength of the foundation rocks, it could also impact spatial variability in the strength of the foundation material arising from heterogeneity in rock's physical properties [3]. In recent times, there has been a spate of building distresses and/or collapse in some areas in Ilaje, Ondo State. This failure could be investigated to be as a result of faulty foundation design. This motivated the researcher to carry an in-situ test in the area to ascertain the competence of the subsurface geology to host structures and possibly provides design alternatives to intending builders and engineers in the area. The pre-construction

investigation may involve direct mechanical boring, pitting and trenching for subsoil sequence delineation, groundwater table mapping, soil sampling, and geotechnical analysis. It may also involve non-invasive geophysical investigation. The use of cone penetrometer test (CPT) and standard penetration test (SPT) have become increasingly useful in subsurface engineering study to obtain information about physical and engineering properties of the subsoil which may include strength and competence of the material that make up the subsoil materials.

Cone penetration test (CPT) is considered the most frequently used method for characterization of geomedia. The CPT is basically advancing a cylindrical rod with a cone tip into the soil and measuring the tip resistance and sleeve friction due to this intrusion [4]. The resistance parameters are used to classify soil strata and to estimate strength and deformation characteristics of soils. Different devices added to cone penetrometers made it possible to apply this test for a wide range of geotechnical applications. The CPT is a simple, quick, and economical test that provides reliable in situ continuous soundings of subsurface

soil. Due to the alluvial nature of soil deposits in Ilaje, CPT is considered as a perfect tool for the site characterization. The application of cone penetration test, CPT has been increased in foundation engineering due to supplying continuous and accurate soil profile in recent years. A static analytical model based on general shear failure mechanism of logarithm spiral has been developed for calculating directly bearing capacity of footings (q_{ult}) from cone resistance q_c [5]. One of the main step for safe and economic design of foundations is ultimate bearing capacity determination. The maximum load that can be applied to subgrade soil from foundation with no occurrence of shear failure and limiting settlement in an allowable upper bond to avoid serviceability damages of superstructure. Four approaches currently are used to determine the bearing capacity of shallow and deep foundations; static analysis, in-situ testing methods, full-scale loading tests and using presumed values recommended by codes and handbooks. Among these approaches, theoretical solution (static analysis) is more common and applied first [6].

The advantages of CPT in contrast to other common in-situ tests is that, it's simple, fast, relatively economical, and it supplies continuous records with depth. The results are interpretable on both empirical and analytical basis, and a variety of sensors can be incorporated with cone penetrometer. Evaluating bearing capacity from CPT data is one of the earliest applications of this sounding and includes two main approaches: direct and indirect methods [7-8]. Direct CPT methods apply the measured values of cone bearing for toe resistance with some modifications regarding scale effects (influence of foundation width to the cone diameter ratio). Indirect CPT methods employ friction angle and undrained shear strength values estimated from CPT data based on bearing capacity and/or cavity expansion theories. The present study/research is aimed at determining important geotechnical parameters that are valuable for foundation design and infrastructural development.

However, supplementing the in-situ cone penetrometer with geophysical investigation will give adequate knowledge of the engineering properties of the subsoil materials that would have direct interaction with the proposed structure. For geophysical studies, electrical resistivity method is the most common technique used for such purpose [9-12]. Electrical resistivity technique of geophysical investigation is an effective tool in delineating subsurface layers and to reliably identify underground structures [13]. This is because the method is reliable, efficient and cost effective. Information such as thickness from vertical electrical sounding (VES) and CPT are some of the commonly applied geophysical and geo-engineering techniques in foundation studies. Authors [14], [15] and [16] have successfully employed VES and CPT in subsurface investigation for engineering purposes. Hence, this study

was focused on the characterization of subsurface for foundation studies using VES and CPT techniques at Ilaje coastal environment of Ondo State, Nigeria.

II. RESEARCH METHOD

A. Study Area

The study area is Ilaje local government area of Ondo State, which falls within southwestern Nigeria (Figure 1) between 670000 and 740000mE and 630000 and 720000mN. Major part of the study area is devoted to fishing activities. Also oil exploration and exploitation are presently carried out onshore in the study area. The people of the area depends on government boreholes and streams for drinking and other domestic uses. The area is within the tropical rain forest region of Nigeria characterized by wet and dry seasonal variations, with a mean annual rainfall of 180 cm, mean temperature of 24 °C, and mean humidity of 80 % [17]. The study area is generally characterized by flat and gently undulating topography. Topographic elevations vary from about 2 to 10 m above sea level. The area is drained by many perennial streams and rivers such as Ominla, Akeun, Ufara, Okomu, Ofara and others, which form a network of dendritic drainage pattern and empty their waters into the Atlantic Ocean to the south [18]. The rivers and streams in the area are being fed by several lagoons, ponds, canals, creeks and small streams scattered across the study area. There are major and minor rivers in the area. The major rivers include rivers Oluwa and Siluko. Others include Oloto, Salawa and Erifa, Iju-Osun, Otu Rivers [19]. The Atlantic Ocean terminates the Ilaje end of the study area. The elevation the area and closeness to the sea make the study area prone to flooding/sea water incursion which normally threatened integrity of foundation especially shallow foundation. In places where pile foundation is used, they are usually constructed by timber of soft woods which could easily undergo serious tension/compression under loading (Figure 2).

The area is characterized by heavy annual rainfall averaging about 2,000 mm. Rainfall is distributed virtually over all the months of the year with the minima occurring between November and March. Plant type is generally mangrove in the costal part of the study area, typical of swamp forest, while the mainland area is characterized by oil palm, rubber plantation and other broadleaved species, typical of rainforest vegetation. The different ethnic groups that live in the area are the Egbados, Ikales, Ilajes and Ijaws. They live in hamlets, villages and towns which are closely separated from each other although mostly connected by fairly-good to poor roads.

The eastern Dahomey basin, geologically where the study area located, as beginning with the Abeokuta Group [20], made up of three Formations from oldest to the youngest namely; the Ise, Afowo and Araromi Formations (Figures 3

and 4). The Ise Formation unconformably overlies the basement complex of southwestern Nigeria and consists of conglomerates and grits at base and in turn overlain by coarse-to-medium grained sands with interbedded Kaolinite. The conglomerates are unimbricated and at some locations ironstones occur [21]. The age is Neocomian to Albian. Overlying the Ise Formation is the Afowo Formation, which composed of coarse to medium grained sandstones with variable but thick interbedded shales, siltstones and claystones. The sandy facies are tar-bearing while the shales are organic-rich [22]. The lower part of this Formation is transitional with mixed brackish to marginal horizons that alternate with well sorted, sub-rounded sands indicating a littoral or estuarine near-shore environment of deposition. Using palynological assemblage, Billman [23] assigned a Turonian age to the lower part of this Formation, while the upper part ranges into the Maastritchian. Araromi Formation overlies the Afowo Formation (Figure 3) and has been described as the youngest cretaceous sediment in the eastern Dahomey basin [24]. It is composed of fine to medium grained sandstone at the base, overlain by shales, siltstone with interbedded limestone, marl and lignite. This Formation is highly fossiliferous containing abundant planktonic foraminifera, Ostracods, pollen and spores. Omatsola and Adegoke [24] assigned a maastritchian to palaeocence age to this Formation based on faunal content.

Figure 2: Diagram showing the closeness of the area to the Atlantic Ocean and how flooding is threatening the integrity of the foundation structures
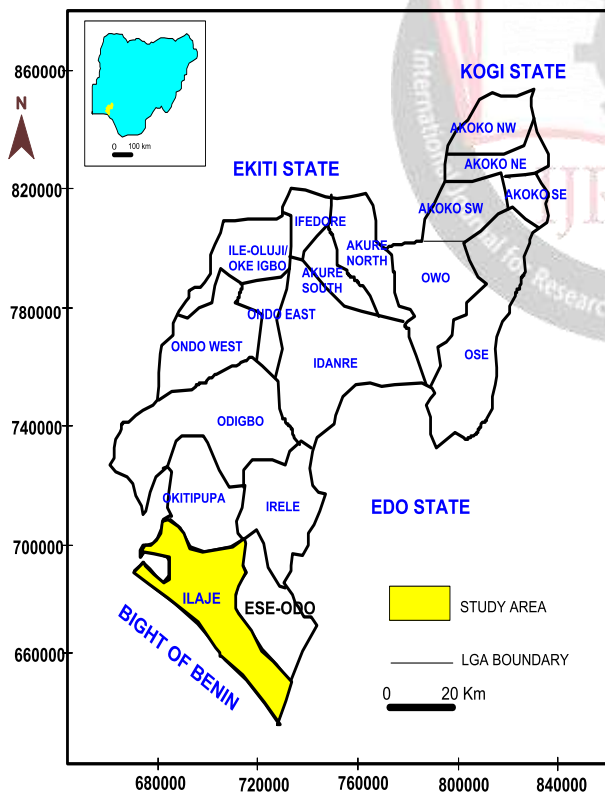

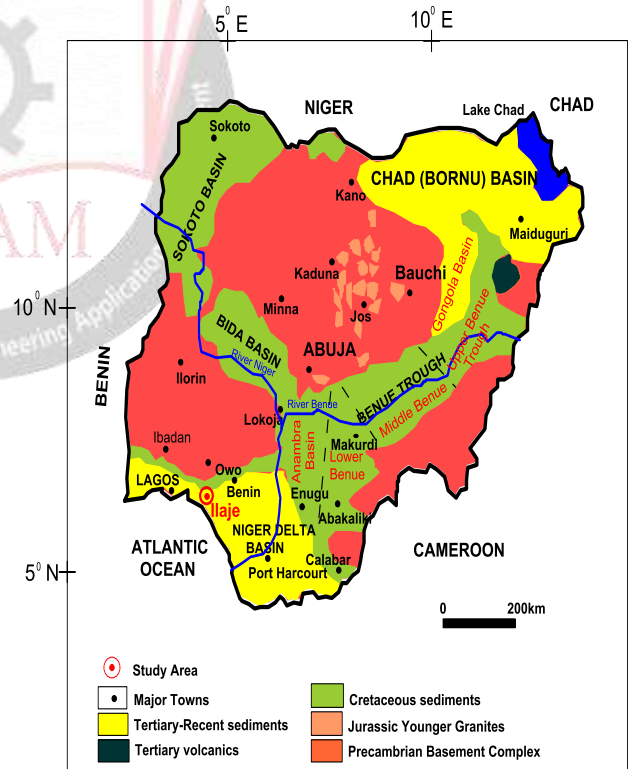Figure 1: Location map of the Study Area

Figure 3: Geological map of Nigeria showing the Study Area on Tertiary – Recent sediments (Modified after Obaje [25])
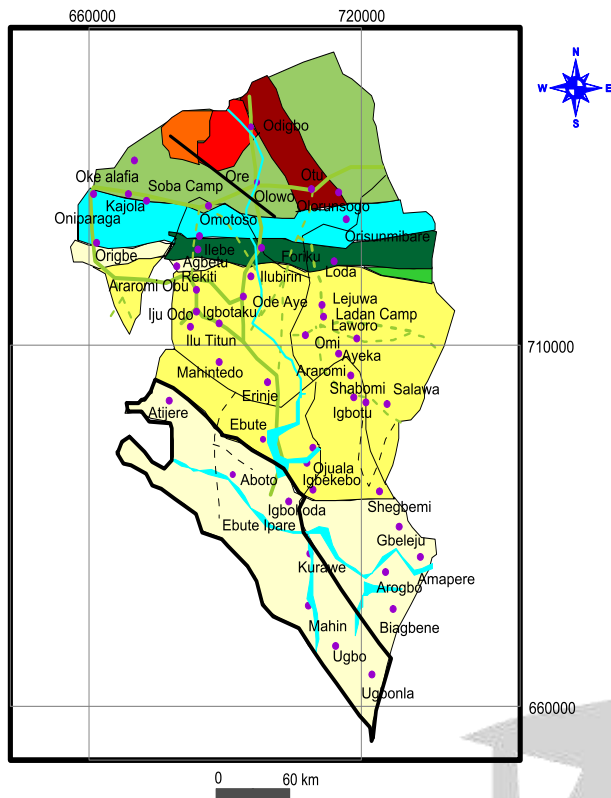
Figure 4: Local Geology map of the Area with predominant Alluvium sand

B. Vertical Electrical Sounding (VES)

The VES were acquired on the proposed site with Ohmega resistivity meter using Schlumberger array with maximum AB/2 of 750 m. The geodetic coordinates of the locations were obtained using Garmin 72 Global Positioning System. A minimum of 2 and maximum of 4 stacks measurement were adopted to ensure high signal/noise ratio. Resist software was used to interpret the VES data. Two (02) parallel traverses trending approximately in the East - West direction were established in the study area (Figure 3). The established traverses length is 120m each. Six (06) Vertical Electrical Sounding (VES) stations were occupied along these traverses using the Schlumberger electrode array (Figure 5). The distance between the VES points was 50 m. The quantitative interpretation of the VES curves involved partial curve matching and computer assisted 1-D forward modelling with the RESIST software [26]. The VES interpretation results were used to generate geoelectric sections. Figure 4 shows the typical VES curves and the interpretation models.

C. Cone Penetration Test (CPT)

Cone Penetration Tests were performed at two (02) locations within the study area (Figures 5 and 6). CPT points were collocated with the VES locations (i.e. VES 2 and 4). The Dutch static penetration 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 10.2 cm^2. 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. 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 20 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. Skin friction was not determined because small machine was employed in the investigation. 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.
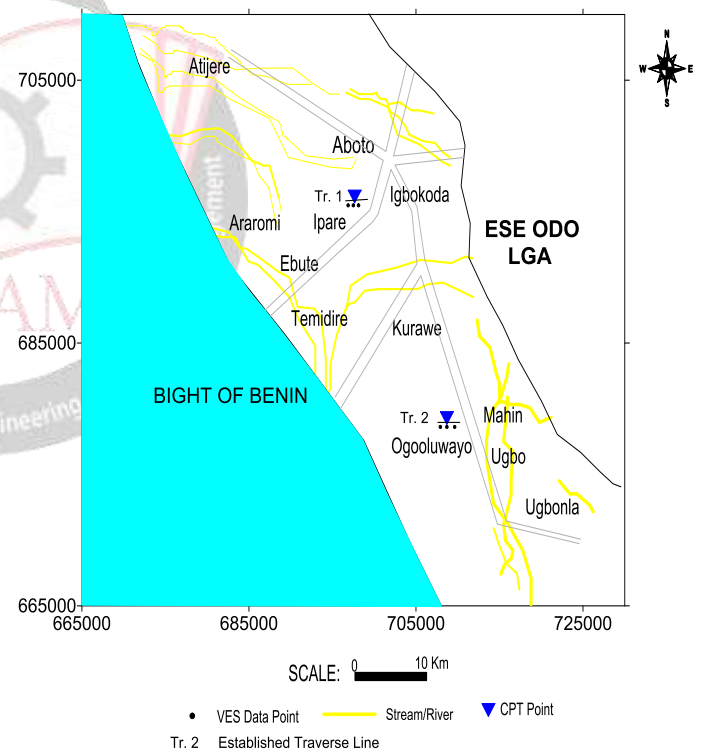

Figure 5: Data Acquisition Map

The layer sequences were interpreted from the variation of the values of the cone resistance with depth. The allowable bearing pressure of the soil layers on each location was calculated using Meyerhof [27] and Schmertmann [28] equation direct method for estimating ultimate bearing capacity (q_{ult}) from cone resistance for square and strip footings.

Figure 6: Field Pictures of in-situ CPT and Trial pits geotechnical tests

Factor of safety at least 3 is recommended by Meyerhof [27] to obtain the allowable bearing pressure.
For cohensionless soils:

Strip $q_{ult} = 28 - 0.0052(300 - q_c)^{1.5}$ kg/cm^2
Square $q_{ult} = 48 - 0.0052(300 - q_c)^{1.5}$ kg/cm^2

For clay:

Strip $q_{ult} = 2 + 0.28 q_c$ kg/cm^2
Square $q_{ult} = 5 + 0.34 q_c$ kg/cm^2

All samples obtained in the field were carefully preserved and subjected to more detailed visual inspection (Figure 6) and descriptions at the laboratory. Thereafter, representative samples were selected from each stratum for laboratory analysis in accordance with relevant geotechnical engineering standards including BS 1377 [29]. The disturbed soil samples were appropriately subjected to the following laboratory classification tests: natural moisture content; Atterberg limits (liquid and plastic limits); grain size analysis; and unconsolidated undrained triaxial tests at different cell pressures. Sieve analysis of cohesive soils were done by soaking oven-dried samples in water overnight and washing through sieve No. 200 (75 microns opening) while remnants retained on sieve No. 200 were oven-dried and sieved mechanically. Materials finer than sieve number 200 were analyzed using the hydrometer method based on Stoke's law. The settlement analysis of the granular soil is determined using Schmertmann [30] equation;

$$\delta = C_1 C_2 C_3 q' \sum \frac{I_e \Delta z}{E_s}$$

where;

C_1 = depth factor

C_2 = secondary creep factor
C_3 = shape factor
q' = net bearing pressure
I_e = strain influence factor
Δz = thickness of soil layer (at midpoint of soil layer)
E_s = equivalent modulus of elasticity

Empirical correction for depth of embedment, secondary creep, and footing shape:

$$C_1 = 1 - 0.5\left[\frac{\sigma'_{zp}}{q'}\right]$$
$$C_2 = 1 + 0.2\log\left(\frac{t}{0.1}\right)$$
$$C_3 = 1.03 - 0.03\frac{L}{B} \geq 0.73$$

In order to use the Schmertmann [28], [30] method, it is necessary to estimate the stiffness of the soil in terms of equivalent Young's modulus at various depths. In the case of normally loaded cohensionless materials (not prestressed significantly to pressures above the present in in-situ overburden pressure, the CPT bearing capacity q_c has been correlated with Young's modulus E_s by DeBeer [31] and Webb [32]. The relationship suggested by Schmertmann [30] is

$$E_s = 2q_c$$

where q_c = CPT bearing capacity

Using Meyerhof [27] equation, the allowable and ultimate bearing capacity was calculated using this equation:

$$q_a = 2.7q_c \left(\frac{KN}{m^2}\right)$$
$$q_a = \frac{q_c}{40} \ (kg/cm^2)$$

Subsequently a factor of safety of 3 was applied on the allowable bearing capacity to get the ultimate bearing capacity [33]. For this study CFEM [34] equation was used for modulus of elasticity determination. A modified version of the Terzaghi bearing capacity equation is widely used for pile design [35]. The third term, or the density term, in the Terzaghi bearing capacity equation is negligible in piles and hence usually ignored [36-37]. The lateral earth pressure coefficient, K, is introduced to compute the skin friction of piles.

$$P_{ultimate} = (\sigma'_t \times N_q \times A) + (K + \sigma'_v \times \tan\delta \times A_p)$$

where:

$P_{ultimate}$ = ultimate pile capacity
σ'_t = effective stress at the tip of the pile
N_q = bearing factor coefficient
A = cross sectional area of the pile at the tip
K = lateral earth pressure coefficient
σ'_v = effective stress at the perimeter of the pile
$\tan\delta$ = friction angle between pile and soil
A_p = perimeter area of the pile

For round piles, $A_p = (\pi d L)$

where:

d - diameter
L = length of the pile

The American Petroleum Institute (API) method [38] uses the following equation for end bearing capacity, which was adopted for this study
where

$$q = N_q \times \sigma'_t \times A$$

q is end bearing capacity of the pile
σ'_t is effective stress at pile tip

A is cross sectional area of the (circular) pile at the tip = $\pi \times \frac{D^2}{4}$; D is the pile diameter

The maximum effective stress used in the computation is within the 240 kPa recommended [39-40]. The value of N_q depends on the soil:

N_q = 8 to 12 for loose sand
N_q = 12 to 40 for medium dense sand
N_q = 40 for dense sand

Meyerhof [27] suggested the following equation for driven/bored piles:

$$s = \beta \times \sigma_v' \times A_p$$

where
s = skin friction of the pile
σ_v' = effective stress at the midpoint of the pile
A_p = perimeter surface area of the pile

For driven piles β = 0.44 for Ø = 28°
= 0.75 for Ø = 35°
= 1.2 for Ø = 37°
For bored piles β = 0.10 for Ø = 33°
= 0.20 for Ø = 35°
= 0.35 for Ø = 37°

III. RESULTS AND DISCUSSION

The interpreted results were presented in the form of geoelectric section along the two traverses in Figures 7 and 8. The VES results showed four to five geoelectric layers across the study area but are distinguished into four major geological layers, comprising the topsoil, silty sand, surficial aquifer unit, and intermediate aquifer unit. Four (04) different curve types were delineated across the study area: HK, HKH, and HKQ. Curve types. The HKQ dominates the study area with frequency of 4 accounting for 66.67 %.

This geoelectric section along Traverse 1 in Igbokoda consist of alluvium sand topsoil, surficial aquifer and intermediate aquifer unit. The topsoil is characterized by resistivity ranging from 155 - 11213 ohm-m with thickness of 2.0 – 5.0 m. The weathered layer (which is the surficial aquifer) has resistivity varying from 198 - 423 ohm-m, with thickness of 68.2 – 122.4 m. The intermediate aquifer underlies the surficial aquifer with resistivity in between 56 and 219 ohm-m, and depth range to this geologic unit is in between 76.8 – 125.1 m. Both the surficial and intermediate aquifer form the major aquifer system in the area. Consequently, the topsoil (the upper 5 m) has high resistivity and thickness values that can sustain structural foundation load. The presence of groundwater in the soil pores has a very significant impact on the engineering behavior or characteristics of the soil, and very important where deep excavation are to be carried out. The groundwater level along this traverse measured during the CPT survey under VES 2 records value of 0.5 m, which is believed to be connate water trapped during sedimentation / lithification process. This depth to this water level is within the targeted foundation depth of 5 m for shallow foundation design in the area. Consequently, this observation needs to be incorporated during the design process, as groundwater level may seriously affect the bases of the foundation footing along this traverse [41].

The geoelectric section along Traverse 2 in Ugbonla also showed four distinct subsurface layering, comprising the topsoil, silty sand, weathered layer (surficial aquifer) and intermediate aquifer unit. The topsoil is made of coastal white sand with resistivity varying from 411 - 111 ohm-m and thickness of 4.8 – 5.8 m. The silty sand underlying the coastal sand has resistivity values varying from 448 – 612 ohm-m, and thickness variation of 8.3 to 15.8 m. The surficial sand aquifer unit is the most prolific aquiferous zone, which is the depth of most boreholes in the area, has layer resistivity ranging from 38 - 102 ohm-m and thickness of 72.1 – 95.2 m. The intermediate aquifer has resistivity ranging from 56 - 92 ohm-m. The depth to this aquifer unit is in between 86.1 and 116.8 m. The low resistivity values recorded for these aquifer system could attributed to high degree of water saturation. The existing well and borehole under VES 4 and 5 record groundwater level of 12 m and 23 m respectively, with an average of 17.5 m. This depth is far below the combined targeted foundation depth (D_f) of 5m and foundation width maximum of 5 m for shallow foundation design. This implies that the groundwater level may not or seriously affect the bases of the foundation footing along this area [41]. Hence the topsoil and silty sand material is moderately competent to harbor the foundation structure along this traverse.

The results of the CPT and laboratory analysis of samples collected during trial test/soil examination is presented in Tables 1 and 2. The result shows increase in cone resistance and sleeve resistance with depth, ranging from 15 – 88 kg/cm^2 and 28 – 110 kg/cm^2 at CPT 1, and 20 - 70 kg/cm^2 and 30 - 125 kg/cm^2 at CPT 2 respectively. The friction ratio ranges from 1.19 – 1.87 (CPT 1) and 1.33 – 1.79 (CPT 2). At CPT 1 the soil seems to be competent at a depths of between 0.2 and 1.0 m. The Robertson [42] soil chart classification shows wide variation of zones 5 to 8 corresponding to clayey silt to silty clay, sandy silt to silty clay, silty sand to sandy silt and sand to silty sand respectively (Figure 9). The plots of cone resistance and sleeve resistance against depth (Figure 10) show high degree of heterogeneity in the upper 1.4 m, with geological succession of clayey silt to silty clay (0-0.2 m), sandy silt to clayey silt (0.2-0.6 m), silty sand to sandy silt (0.6-1.0m) and sand to silty sand (1.0-1.2 m) for CPT 1, while for CPT 2, three geologic layering were interpreted consisting of clayey silt to silty clay (0-0.2 m), sandy silt to clayey silt (0.2-0.6m) and silty sand to sandy silt (0.6-1.0 m).

Consequently in order to avoid clay mixed soil, a depth of 0.6 m would be appropriate as founding depth for design and construction of shallow foundation and bases. The analyzed soil samples at both locations (Tables 2 and 3) shows liquid limits of 20 – 42.3 % (within recommended 50 %), plastic limits of 18 – 22.1 % (within recommended 30 %), plasticity index of 9.2 – 24.26 %, and shrinkage limits of 0.5 – 8.2 % (low/medium good soil quality). The natural moisture content ranges from 10.2 – 15.9 % which is moderately low. The soil recorded no value for gravel, %

sand varies from 62.1 – 93.8, % silt is in between 5.2 – 22, clay ranges from 0.2 – 16 %. Based on this result, the soil is generally silty sand which corroborates the geoelectric sections and CPT geologic section. The average clay content in the soil is less than 10 %. Consequently it is expected that such soil will undergo immediate/elastic settlement than consolidation settlement, since most silty (except plastic silt) and granular soil undergo total settlement even shortly after construction.
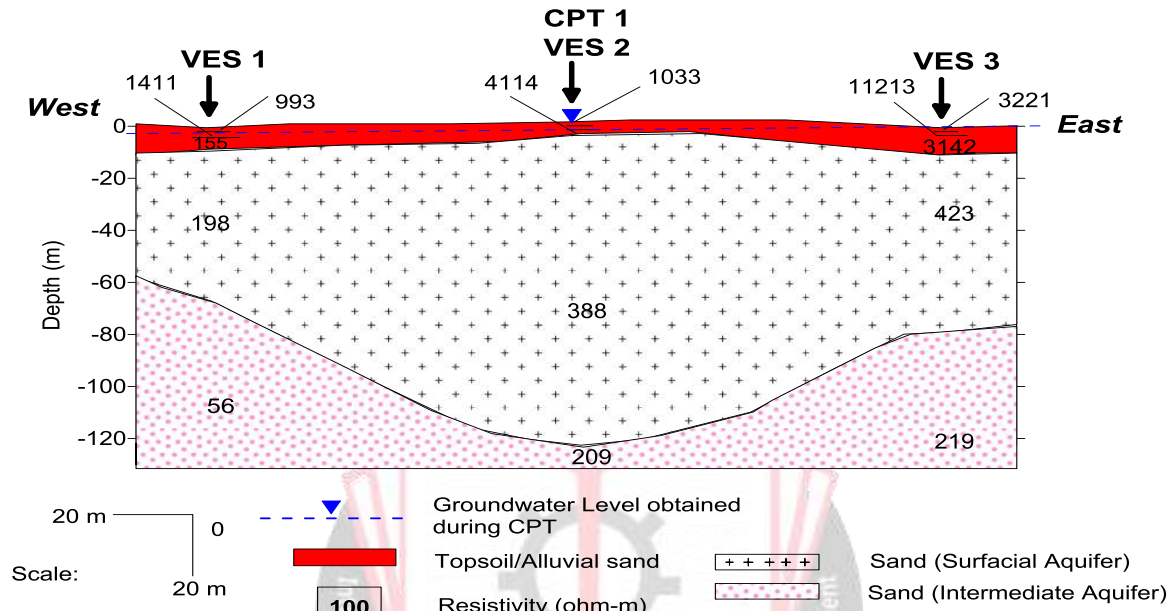

Figure 7: Geoelectric Section along Traverse 1 in Igbokoda also showing measured water level during the CPT
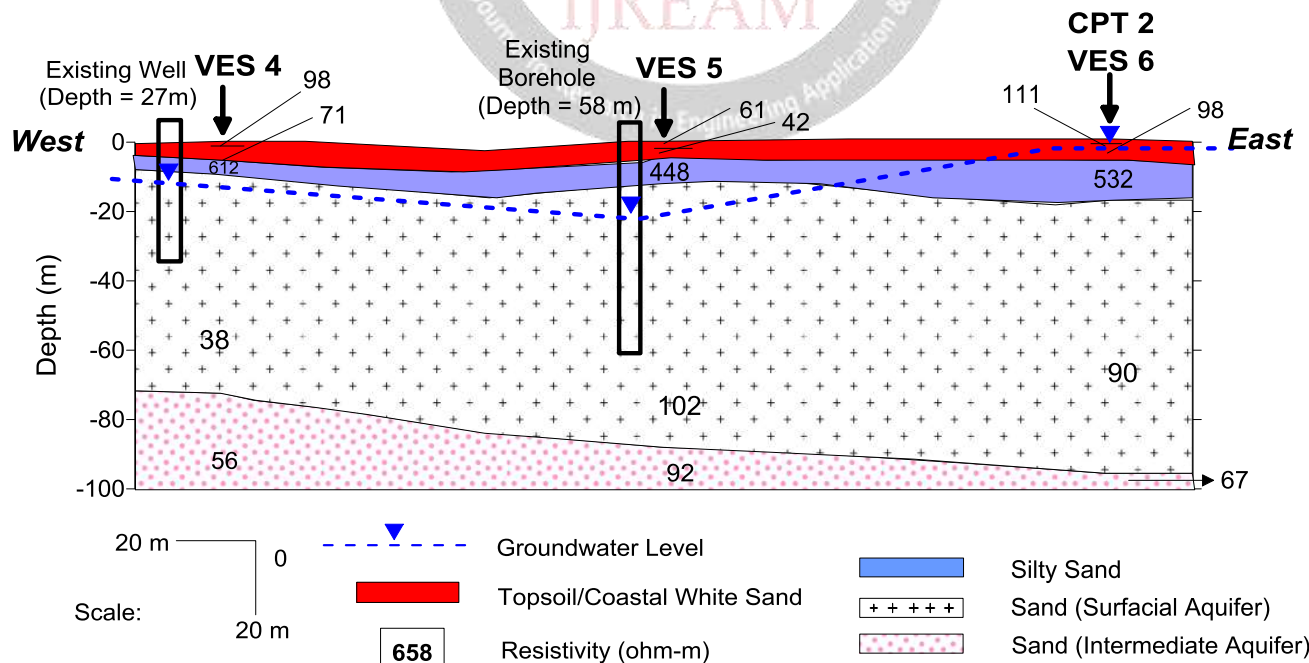

Figure 8: Geoelectric Section along Traverse 1 in Ugbonla/Ugbo and points of the existing wells with measured/inferred static water level

Table 1: Geotechnical/Engineering Properties of Soil in Ojuala (Location 1)

Depth (m)	Cone Resistance	Sleeve Resistance	Friction Ratio	L.L (%)	P.L (%)	P.I (%)	S.L (%)	M.C (%)	% Gravel	% Sand	% Silt	% Clay	S.G
0.2	15	28	1.87	22.0	-	-	0.7	10.3	-	86.8	10.2	3.0	2.68
0.4	25	35	1.40										
0.6	30	42	1.40	20	-	-	0.5	11.1	-	82.0	17.0	1.0	2.67
0.8	55	68	1.24										
1.0	60	80	1.33	31.3	22.1	9.2	0.5	15.1	-	93.8	6.0	0.2	2.68
1.2	80	95	1.19										
1.4	88	110	1.25	32.2	22.1	10.1	4.3	15.3	-	93.4	5.2	1.4	2.68

Table 2: Geotechnical/Engineering Properties of Soil in Igbekebo (Location 2)

Depth (m)	Cone Resistance	Sleeve Resistance	Friction Ratio	L.L (%)	P.L (%)	P.I (%)	S.L (%)	M.C (%)	% Gravel	% Sand	% Silt	% Clay	S.G
0.2	20	30	1.50	42.3	18	24.26	8.2	15.9	-	68.3	20.1	11.6	2.69
0.4	30	40	1.33										
0.6	35	50	1.43	32	31.9	10.10	7.2	11.9	-	69.9	19.5	10.9	2.69
0.8	55	80	1.45										
1.0	70	125	1.79	28	19.2	8.8	5.5	10.2	-	62.1	22.0	16.0	2.68

The ultimate and allowable bearing capacity estimated from the cone resistance using Meyerhof [27] equation as presented in Table 3. The calculated bearing capacities could be used in determining the foundation type for structures [43]. The allowable bearing of the soil varies between 37 to 216 KN/m^2 for CPT 1, and 49 – 172 KN/m^2 for CPT 2. Consequently an average allowable bearing capacity of 150 KN/m^2 (ultimate bearing capacity of 450 KN/m^2) would be appropriate for design of shallow foundation in the area, at a depth not less than 1.0 m.

The total settlement is the total compression of a saturated soil layer over a long period of time under static load. It is the combination of primary/elastic and secondary settlements. The rate of settlement depends on thickness of soil layer, permeability of soil, number of drainage faces, and magnitude of applied load. The commonly accepted basis of design is that the total settlement of a footing should be restricted to about 25 mm [44-45] as by so doing the differential settlement between adjacent footings is confined within limits that can be tolerated by a structure. The settlement analysis for foundation width of 0.5 m at three depth levels of 1 m, 2 m and 3 m produces relative high values (greater than 25 mm) settlement values. But foundation width of 1.5 – 3 m produces settlement less than 25 mm (Table 4). Although according to Meyerhof [27], Schmertamnn [28] total settlement limits of 60 mm (clay) and 50 mm (granular soil) are still tolerable. Therefore foundation width of 0.5 m for different depth levels is still within tolerable limit. However settlement at foundation is low when the foundation width is increase to 2 m and 3 m. The calculation of bearing capacities for strip and square foundation is shown in Table 5. For strip foundation, the appropriate (recommended) ultimate bearing and allowable bearing capacity for depth levels of 0.6 m and 1.2 m are 1102 and 367 KN and 2254 and 751 KN respectively. The

ultimate bearing and allowable bearing capacity for square foundation at depth levels of 0.6 m and 1.2 m are 1590 - 2989 KN/m^2 and 530 - 996 KN/m^2 respectively.
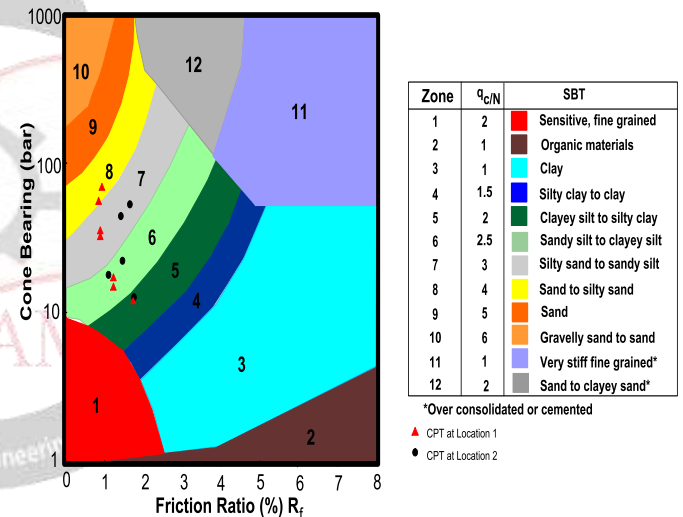

Figure 9: Robertson's (1990) Soil Classification Chart and Description of Different Zones for soil in the two localities, with predominant Zones 6 and 7

An attempt was made to design for deep foundation at depth levels of 5 m, 10 m, and 15 m for circular piles of 400 mm, 500 mm, and 600 mm while taking into consideration the groundwater level. The type of piles designed for in the area included driven/displacement piles and bored piles. The versatility and cost effectiveness are part of the governing factors considered. The bearing capacity of pile depends on pile diameter, founding depth, vertical stress, area of the pile, method of installation etc. The result shows that the larger the diameter of the pile, the better its bearing capacity. For driven pile at 5 m, 10 m, and 15 m, the adopted allowable capacity based on calculation is in between 64 – 115 KN, 206 – 347 KN, and 418 – 677 KN
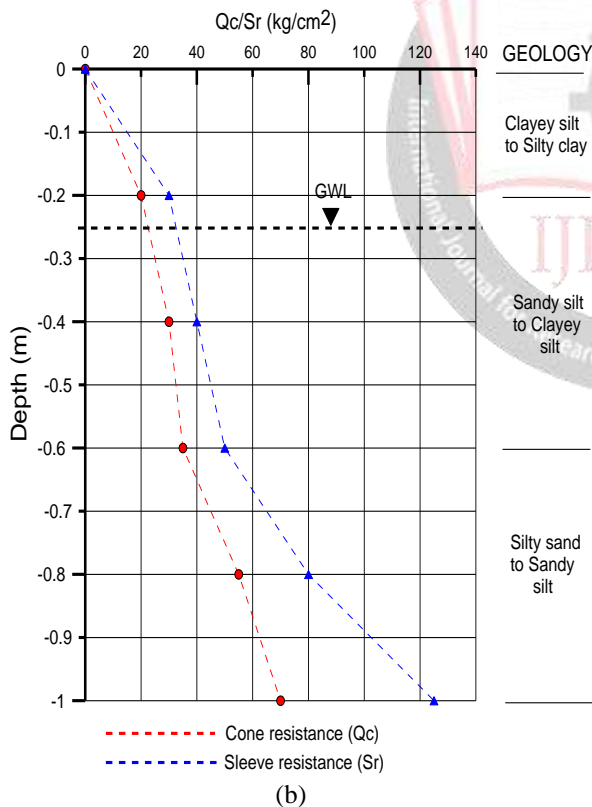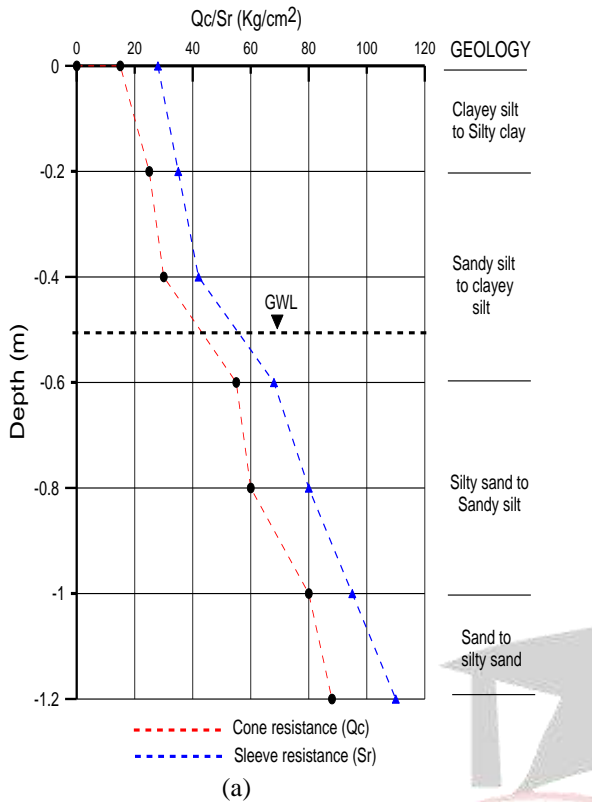
respectively.

Figure 10: Plots of Cone resistance and sleeve resistance against depth at location 1 and 2, corresponding to (a) Ojuala CPT 1 (b) Igbekebo CPT 2

The allowable bearing capacity for bored piles ranges from 34 – 69 KN, 85 – 165 KN, and 146 – 268 at depth levels of 5 m, 10 m, and 15 m respectively (Table 6). Although it is

recommended that pile load test be conducted before the design, construction and installation of pile [46] in the locality to ensure this proposed design alternatives be effectively and correctly used in the study area.

Table 3: Bearing Capacities estimated from the Cone resistance values for both sites (locations)

Depth (m)	CPT 1		CPT 2	
	$q_{allowable}$ (KN/m^2)	$q_{ultimate}$ (KN/m^2)	$q_{allowable}$ (KN/m^2)	$q_{ultimate}$ (KN/m^2)
0.2	37	110	49	147
0.4	61	184	74	221
0.6	74	221	86	257
0.8	135	404	135	404
1.0	147	441	172	515
1.2	196	588	-	-
1.4	216	647	-	-

Table 4: Settlement variation at Different Depths and Foundation Widths

Foundation width (m)	Settlement (mm) at Depth Level (m)		
	1 m	2 m	3 m
0.5	46.17	45.84	45.19
1.0	30.28	31.00	31.10
1.5	26.39	27.39	27.67
2.0	16.27	17.72	18.34
2.5	13.02	14.57	16.61
3.0	10.88	12.50	14.58

Table 5: Bearing Capacities for Strip and Square Shallow Foundations in the Study area

Depth (m)	Strip $q_{ultimate}$ (KN/m^2)	Square $q_{ultimate}$ (KN/m^2)	Strip $q_{allowable}$ (KN/m^2)	Square $q_{allowable}$ (KN/m^2)
0.6	1102	1590	367	530
1.2	2254	2989	751	996

IV. CONCLUSION

Subsoil evaluation and foundation design in Ilaje area of Ondo State for civil engineering structure using geoelectrical and geotechnical methods of investigation were carried out. The investigation was to provide information on the stratigraphy, nature, structural disposition and competence of the subsoil. Six (6) Vertical Electrical Sounding (VES) stations were occupied and this was complemented with geotechnical analysis using CPT and seven soil samples collected at the CPT locations, within 1.4 m depth. All the determined geotechnical parameters of the subsoil fall within the specification recommended for foundation material. According to Robertson's chart, the distribution of the CPT data obtained encompasses four geologic zones and composed of clayey silt to silty clay, sandy silt to silty clay, silty sand to sandy silt and sand to silty sand respectively. The VES shows a predominant HKQ curve types which account for 66.67%. The upper 5 m is characterized by moderate thickness and high resistivity values to sustain the structure. The topsoil (the upper 5 m) has high resistivity and thickness values that

can sustain structural foundation load. The average bearing capacity of 150 KN/m^2 could be adopted in the area for design of bases/footings for shallow foundation at a depth not less than 1.0 m. The total settlement is less than 50 mm tolerable limit for foundation width ranging from 0.5 to 3.0 m. The allowable capacity of the driven pile ranges 64 – 115 KN, 206 – 347 KN, and 418 – 677 KN at 5m, 10 m and 15 m respectively. The allowable bearing capacity for bored piles ranges from 34 – 69 KN, 85 – 165 KN, and 146 – 268 at depth levels of 5 m, 10 m, and 15 m respectively.

Table 6: Axial Pile Capacity for proposed Driven and Bored Piles

Depth (m)	Pile diameter (m)	End bearing capacity (KN)	Driven Pile			Bored Pile		
			Skin Friction (KN)	Ultimate pile capacity (KN)	Allowable pile capacity (KN)	Skin Friction (KN)	Ultimate pile capacity (KN)	Allowable pile capacity (KN)
	0.4	75	117	192	64	27	101	34
	0.5	117	147	264	88	33	150	50
5	0.6	168	176	344	115	40	208	69
	0.4	149	470	619	206	107	256	85
	0.5	234	587	821	274	133	367	122
10	0.6	336	705	1041	347	160	496	165
	0.4	198	1057	1255	418	240	438	146
	0.5	309	1321	1630	543	300	609	203
15	0.6	445	1585	2030	677	360	805	268

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