(RESEARCH ARTICLE)

In-situ foundation characterization and design in Ese Odo Local Government Area of Ondo State, Southwestern Nigeria

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Abstract

This study aims at assessing and characterizing the subsoil types/profile to determine the engineering characteristics of the underlying soils and provide appropriate foundation design alternatives, in Ese Odo area of Ondo State, Nigeria using geophysical and geotechnical methods. The geophysical method involved six vertical electrical soundings (VES) and complemented by two cone penetrometer tests and eight trial pits/boring where representative soil samples were obtained and analyzed in the laboratory, in accordance with relevant geotechnical engineering standards. The VES shows a predominant HK and KHK curve types which account for 33.3 % each. The upper 5 m is characterized by moderate thickness (3.3 – 9.6 m) which could dissipate the influence of structural load effectively and high resistivity values (380 – 2439 ohm-m) to sustain the structural load. The average bearing capacity of 100 KN/m² is recommended for design of bases/footings for shallow foundation at a depth not less than 2.8 m in the area with the total settlement less than 50 mm tolerable limit, for foundation width ranging from 0.5 to 3.0 m. The recommended range of allowable bearing capacity for strip foundation is 221 - 459 KN/m², while square is 352 - 641 KN/m². For driven pile at 5 m, 10 m, and 15 m, the adopted allowable capacity varies between 66 – 118 KN, 212 – 357 KN, and 440 – 717 KN respectively. The allowable bearing capacity for bored pile ranges from 35 – 71 KN, 88 – 170 KN, and 159 – 297 at depth levels of 5 m, 10 m, and 15 m respectively.

Keywords: Cone penetration test, Bearing capacities, Foundation design, Pile capacity, Ese Odo, VES

1. Introduction

In engineering, geotechnical information is important in foundation works and ensures that the effects of projects on the environment and natural resources are properly evaluated and mitigation measures are applied where necessary [1]. Foundation of any structure is very important because the safety and reliability of structure depends on it [2]. Load of structure are transmitted from the superstructure to the substructure by columns and walls. The selection of type of foundation for a given site depends on many factors. The most important factors are cost of the superstructure, subsurface condition, the functions of the structure and the load it must carry. Also the design load: dead, live, wind or earthquake forces, lateral pressures exerted by the foundation earths on the embedded structural elements; impact equivalents relating to moving and dynamic loads [3]. In this study, priority considerations in the design of structures is given to pre-construction investigation of the subsurface at Ese Odo Local Government of Ondo State, Southwestern Nigeria, in order to ascertain the competence of the subsurface material to host building structures. Such information is required for proper design and construction of foundation, for the assessment of ease of excavation.

Therefore realizing the importance of geophysical and geotechnical investigation for foundation design, an in-situ test using geophysical and cone penetrometer test were conducted in Ese Odo area of Ondo State, Nigeria with the objectives of measuring the thickness of the overburden or the depths to the bedrock; and determine the nature of the superficial deposit. Geophysical methods are applied in civil engineering construction related projects ranging from pre-

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construction feasibility studies to post construction integrity assessment [4 - 11]. Geophysical methods are used for the delineation of subsurface sequence, identification of geological structures and determination of physical parameters of rock formations. These information enhance rapid characterization of subsurface formations, identification of competent subsurface layers, determination of thickness and depth required for the design of civil engineering foundation. Resistivity survey is common geophysical method for subsoil exploration [12]. In this method, artificially generated electric (I) current injected into the ground via two current electrodes while the resulting potential difference (V) is measured by another pair of electrodes. The soil resistance given by Parasnis [13];

R = V / I

This needs to be standardized over a unit length. The equation of resistivity ρ (which measured in ohm-m) is;

ρ = 2πdR

where, d is spacing between the electrodes (in metre).

On the other hand cone penetration test (CPT) and standard penetration test (SPT) are the most commonly employed in-situ tests to delineate soil stratigraphy and determine the geotechnical engineering properties of subsurface soils [14]. In spite of continuous efforts to standardize the SPT procedure and equipment, problems associated with its repeatability and reliability still exists. The cone penetration test has been recognized as one of the most widely used in-situ tests [15 - 16]. The cone penetration test consists of advancing a cylindrical rod with a conical tip into the soil and measuring the forces required to push this rod. The friction cone penetrometer measures two forces during penetration. These forces are: the total tip resistance (q_c), which is the soil resistance to advance the cone tip and the sleeve friction (f_s), which is the sleeve friction developed between the soil and the sleeve of the cone penetrometer. The friction ratio (R_f) is defined as the ratio between the sleeve friction and tip resistance and is expressed in percent. The resistance parameters are used to classify soil strata and to estimate strength and deformation characteristics of soils. The cone penetration test data has been used to predict the ultimate axial pile load carrying capacity. Cone penetrometer testing (CPT) is a fast, effective, and relatively inexpensive system for collecting important soils parameters during a geotechnical site investigation. When used in conjunction with conventional drilling and sampling methods, it provides a more complete description of the subsurface conditions, thereby reducing uncertainty in design and construction. CPT methods can be divided into two basic groups: geophysical logging and stratigraphic profiling specific test methods [17]. Therefore an integration of geophysical with geotechnical methods proffer solutions discrete information usually got from conventional engineering soil characterization methods that lack complete imaging of the subsurface. Geophysical data interpretation can image the subsurface to the depths of competent layer and evaluate the real distribution of geological earth material. Information such as soil type, load bearing capacity of materials, zone of weakness, resistance to penetration, compressibility, shrinkage limit, etc. are usually revealed. This study is aimed at delineating the geologic features using 1-D probing techniques and geotechnical method using cone penetrometer test (CPT) for shallow foundation design.

2. Description of Study Area

The study area is situated in the southern part of Ondo State comprising towns like Igbotu, Igbekebo, Shabomi, Akotogbo etc. It is on the coastal plain between the western Nigeria highlands and the Atlantic Ocean. The study area lies within longitude $4^0 27'$ E to 5^0 10' E and Latitude $6^0 28'$N to 6^0 45' N (Fig. 1). The terrain of the study area is fairly plain. Deciduous continental marginal forest with dense vegetation including marshes, raffia, mangrove and related vegetation, characterize the study area.

The study area falls within the transition zone and the sedimentary portion of southwestern Nigeria on the eastern margin of the Dahomey basin. It is a coastal sedimentary basin filled with cretaceous and younger sediments unconformably overlying the block – faulted basement complex rocks [18]. The Dahomey basin extends from the Ghana – Ivory Coast boundary across Togo and Benin Republic to western Nigeria. It ends at the western margin of the Niger Delta Basin from which it is separated by the Okitipupa structural high and a major fault structure, the Benin Hinge line [19, 20 - 24]. The stratigraphy of the Dahomey basin has been discussed by several authors from surface and subsurface data as presented in Table 1. They also subdivided the basin sedimentary fill into three materials:

- Sand and sandstone at the base
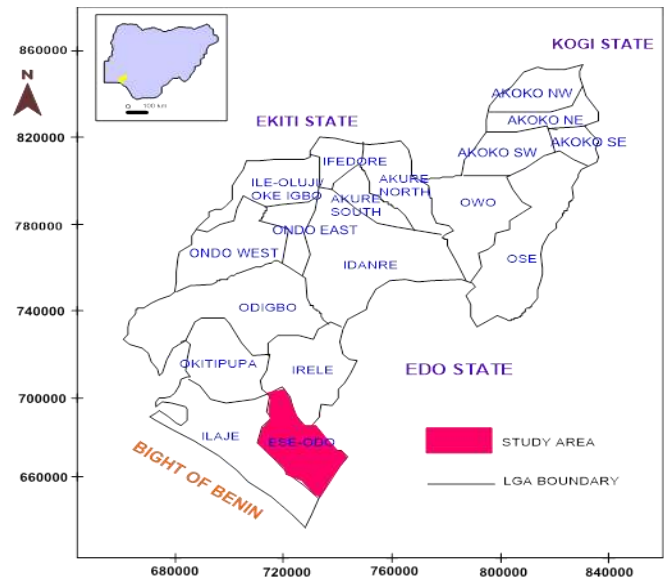- Alternating sands and shales and
- Upper shales

Figure 1 Location Map of The Study Area on the map of Ondo State and Nigeria

There are major and minor rivers in the area, the major rivers include rivers Oluwa and Siluko; others are Oloto, Salawa, Erifa, Iju-Osun, Otu rivers. The streams and rivers flow into the Atlantic Ocean which terminates the Ilaje end which shares border with the study area**.**

Table 1 Stratigraphic succession in the Dahomey Basin [22]

Age	Stratigraphic Unit
Recent	Alluvium
Pleistocene to Oligocene	Benin formation
Eocene	Ilaro formation/Oshoshun member
Paleocene	Imo group/Akinbo formation
	Ewekoro formation
Maestrichain to neocomain	Abeokuta group/
	Araromi formation
	Afowo formation
	Ise formation
Lower Paleozoic to Precambrian	Basement complex

3. Material and methods

The investigation entailed geologic mapping of the area to determine rock types; geophysical investigation to differentiate subsurface layers using vertical electrical sounding (VES); as well as in-situ geotechnical investigation to determine geotechnical properties of the earth materials (Fig. 2). The in-situ test involved CPT. The cone penetrometer test (CPT) was carried out at 2 locations on the two established traverse lines by forcing a hardened steel cone with a base area of 1000 mm^2 at an apex angle of 60° continuously into the ground and measuring its resistance to penetration [4, 15]. The Dutch cone 2.5 tonnes equipment is a manually operated unit furnished with a single cone that can measure the cone resistance, q_c only. The cone was advanced at regular intervals of 25 cm (0.25 m) and the corresponding pressure required to advance it is transmitted to a gauge which in turn records this pressure value. This procedure was repeated until the required depth is either reached or the total resistance to penetration of the tubes and cone reaches the capacity of the machine. Successive cone resistance readings were plotted against depth to form a resistance profile

which indicates the strata sequence penetrated. Disturbed soil samples were collected at different depth levels in a trial pits and used in carrying out laboratory/in-situ tests. Laboratory analyses included index properties, strength and compressibility tests adopting procedures prescribed by the British Standards [25]. The moisture content (BS 1377:4), liquid and plastic limits (BS 1377:2), triaxial test, and linear shrinkage tests were carried out on soil samples collected from different depths.

The materials and instruments used for vertical electrical sounding includes the following; location, topographic and geological maps of the study area, global positioning system (GPS), calibrated polyethylene twain, steel electrodes, reels of cables, hammers and resistivity meter. Four reels were used; two for current cables and the other two for potential cables. The wheels help to control the cables. They also provide points of connection between electrodes and the resistivity meter. Compass clinometer was used to establish geographic bearing in the study area. Global positioning system device (GPS) Garmin GPS-78-12-channels hand device was used to establish the coordinates and elevation of vertical electrical sounding (VES) locations across the study area. Two pairs of galvanized steel electrodes were used for the survey. A pair serves as current electrodes and another pair as the potential electrodes. Hammers were used for driving electrodes firmly into the ground in order to ensure good electrical contact, while PASI 16GL earth resistivity meter and its accessories were used in acquiring vertical electrical sounding data. The equipment has high sensitivity and can measure as small as 600 mV. The instrument is managed by multiprocessor and it is auto ranging; which is the ability to automatically adapt signal range to that of value to be measure. It also has the ability to stack data (continuous averaging of values acquired). Its internal memory can store up to 1800 readings and it allows direct downloading of reading into a computer via its data transfer port.

The schlumberger electrode configuration was adopted for the vertical electrical sounding (VES) techniques, which is a measure of vertical variations in the ground resistivity. This is achieved by expanding the inter-electrode spacing about a fixed center of array. A total of six (06) vertical electrical soundings were conducted across the survey area (Fig. 2). Each sounding involved stepwise expansion of current electrode separation (AB/2) from minimum of 1 m to maximum of 500 to 750 m depending on availability of space for spreading. The sounding data obtained were presented as sounding curves, which are plots of apparent resistivity (ℓa) values against electrode separation (AB/2) on bi-log graph. The interpretation of VES curves was done in two steps; the first step is manual curve matching [26].
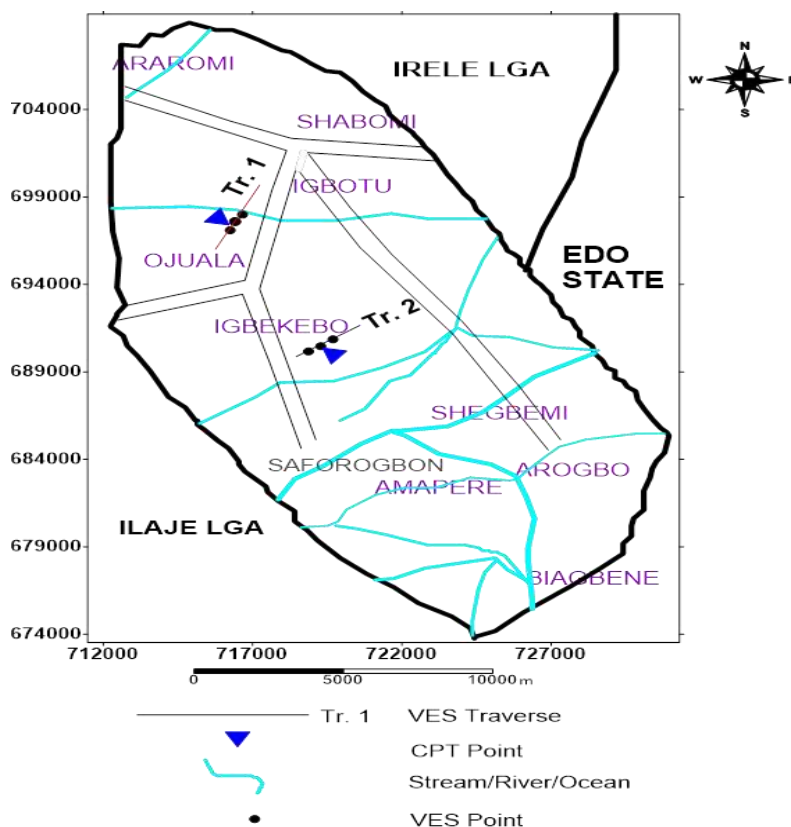

Figure 2 Data acquisition map showing VES and CPT Points

This involves segment-by-segment curve matching of the sounding curves with the theoretical curves with the assistance of auxiliary curves. This exercise yields geoelectric parameters, which is layer thickness and their resistivity values. The results were later enhanced by computer iteration. Computer software known as RESIST version 1.0 [27] was used to refine the geoelectric parameters obtained from the manual curve matching exercise. 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 [28 - 29] equations direct method for estimating ultimate bearing capacity (q_{ult}) from cone resistance for square and strip footings, as follows:

$$q_{ult} = q_c \left(\frac{B}{12.2}\right)\left(1 + \frac{D_f}{B}\right) \quad \text{kg/cm}^2$$

q_c = cone resistance value

D_f = Depth of footing

B = Width of foundation

Factor of safety at least 3 is recommended by [28] to obtain the allowable bearing pressure.

For cohensionless soils:

Strip $\quad q_{ult} = 28 - 0.0052 \, (300 - \, q_c)^{1.5}$

Square $\quad q_{ult} = 48 - 0.0052 \, (300 - \, q_c)^{1.5}$

For clay:

Strip $\quad q_{ult} = 2 + 0.28q_c$

Square $\quad q_{ult} = 5 + 0.34q_c$

All samples obtained in the field were carefully preserved and subjected to more detailed visual inspection and descriptions at the laboratory. Thereafter, representative samples were selected from each stratum for laboratory analysis in accordance with relevant geotechnical engineering standards [25]. 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. Total consolidation settlement (s) has been computed for foundation breadth (B) between 0.5 – 3.0 m, subjected to allowable bearing capacity of 100 KN/m². The induced vertical stress ($\Delta\sigma$) at the centre of the consolidating layer has been used in computing settlement. The consolidation settlement has been computed from the expression below [30].

$$s = \, m_v H \Delta\sigma'$$

m_v = coefficient of volume compressibility

H = thickness of compressible layer

$\Delta\sigma'$ = average increase in effective pressure

An m_v value of 0.125 m²/KN, which corresponds to the adopted net allowable bearing capacity was used in the settlement analysis and also corresponds to stiff clay in the range of (0.25 – 0.125 m²/KN). The settlement analysis of the granular soil is determined using [31] 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:

$$C_1 = 1 - 0.5\left[\frac{\sigma'_{zD}}{q'}\right]$$

$$C_2 = 1 + 0.2\log\left(t/0.1\right)$$

$$C_3 = 1.03 - 0.03 L/B \geq 0.73$$

In order to use Schmertmann [29, 31] methods, it is necessary to estimate the stiffness of the soil in terms of equivalent Young's modulus at various depths. The Dutch cone penetration resistance of the soils at the site. 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 [32 - 33]. The relationship suggested by [31] is;

$$E_s = 2q_c$$
where q_c = CPT bearing capacity

Using [28] 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. For this study [34] equation was used for modulus of elasticity determination. A modified version of the Terzaghi bearing capacity equation is widely used for pile design. The third term, or the density term, in the Terzaghi bearing capacity equation is negligible in piles and hence usually ignored [35 - 36]. The lateral earth pressure coefficient, K, is introduced to compute the skin friction of piles.

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

$$= \text{End bearing term + Skin friction term}$$

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, $A_p = (\pi d L)$

d - diameter

L = length of the pile

The API method [37] uses the following equation for end bearing capacity, which was adopted for this study, where

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

Q = end bearing capacity of the pile

σ'_t = 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 [38 - 39]. 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 [28] 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°

4. Results and discussion

4.1. Vertical Electrical Sounding (VES)

The interpreted results were presented in the form of geoelectric section along the two traverses in Figures 3 and 4. The VES results showed four to five geoelectric layers were delineated across the study area but are distinguished into three geologic zones. Layer resistivity varies from 601 - 2439, 380 - 1388, and 52 – 255 ohm-m in the topsoil, surficial aquifer unit, and intermediate aquifer unit geoelectric layers respectively. Four (04) different curve types were delineated across the study area: HAK, KHK, HKQ, and HK. Curve types HK and KHK dominates the study area with frequency of 2 each (33.3 %).

This geoelectric section along Traverse 1 (Ojuala) consist of sand dominated soil and reddish in colour due to presence of clay mineral, with resistivity ranging 612 - 1230 ohm-m with thickness of 3.3 – 6.9 m. The weathered layer (which is the surficial aquifer) underlain the red tropical sand topsoil with resistivity varying between 588 to 1017 ohm-m, with thickness of 58.4 – 73.2 m. The intermediate aquifer underlies the surficial aquifer with resistivity in between 52 and 98 ohm-m, and depth range of 81.5 – 83.4 m. Both the surficial and intermediate aquifer form the major aquifer system in the area. Consequently, the upper 5 m has moderate/high resistivity and thickness values that can sustain structural foundation load. The thickness is more than 5 m, which could help in evenly distribution of load to the subsoil. 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 from existing abandoned well and borehole, records values of 9.5 and 16.8 m. This depth is far below the targeted foundation depth of 5 m for shallow foundation design. This implies that the groundwater level may not or seriously affect the bases of the foundation footing in this area [3].
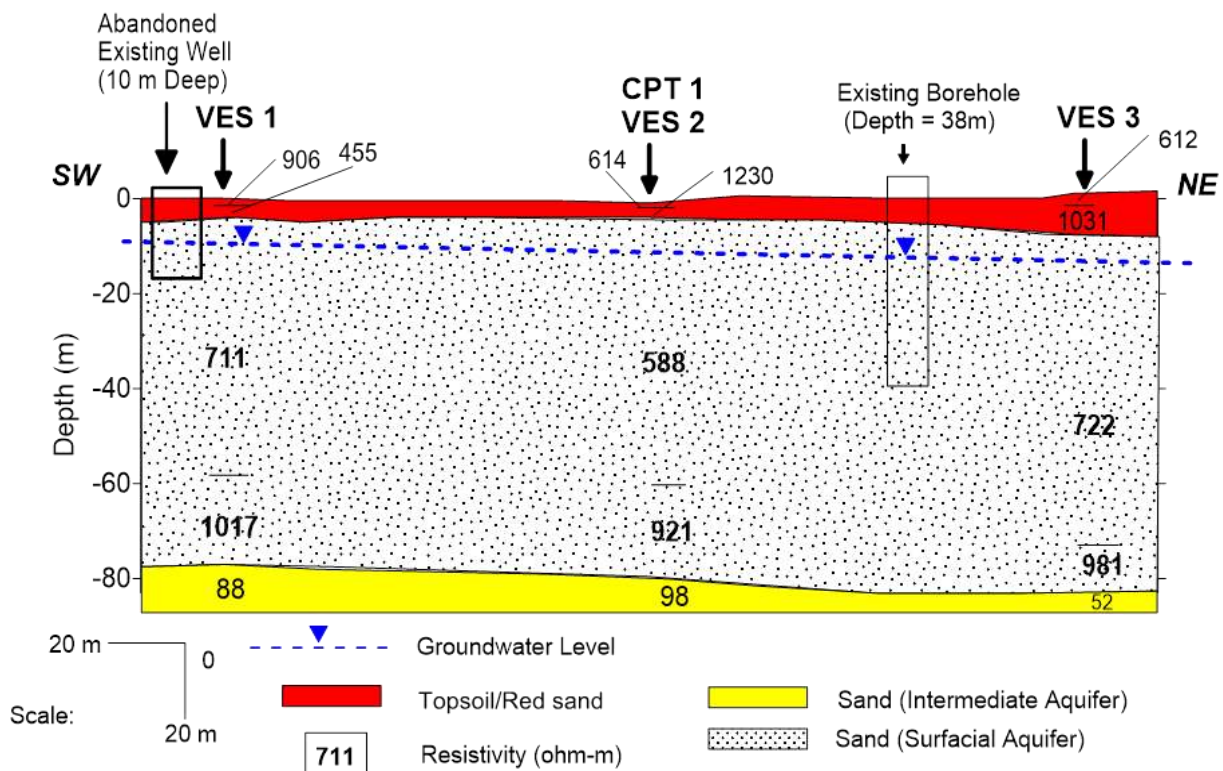

Figure 3 Geoelectric section along profile 1 in Ojuala

The geoelectric section along Traverse 2 (Igbekebo) also showed three distinct subsurface layering, comprising the topsoil, weathered layer (surficial aquifer) and intermediate aquifer unit. The topsoil is made tropical red sand/clayey sand with resistivity varying from 601 - 2439 ohm-m and thickness of 4.2 – 9.4 m. The surficial sand aquifer unit is the most prolific aquiferous zone, which is the depth of most boreholes in the area, the layer resistivity ranges from 380 - 1388 ohm-m and thickness of 72.4 – 68.4 m. The intermediate aquifer has resistivity ranging from 101 - 255 ohm-m. The depth to this aquifer unit is in between 92.6 and 100.4 m. The existing borehole in the area along this profile confirms groundwater level of 9.5 m, which is very similar to the one measured along profile 1 at VES 1. The

groundwater level is an indicative of type of soil and its permeability. Groundwater level affects many important phases in the designing and construction of foundation [40]. Consequently the water level along this traverse may not pose serious threat to foundation footing if the foundation base is founded within the topsoil.
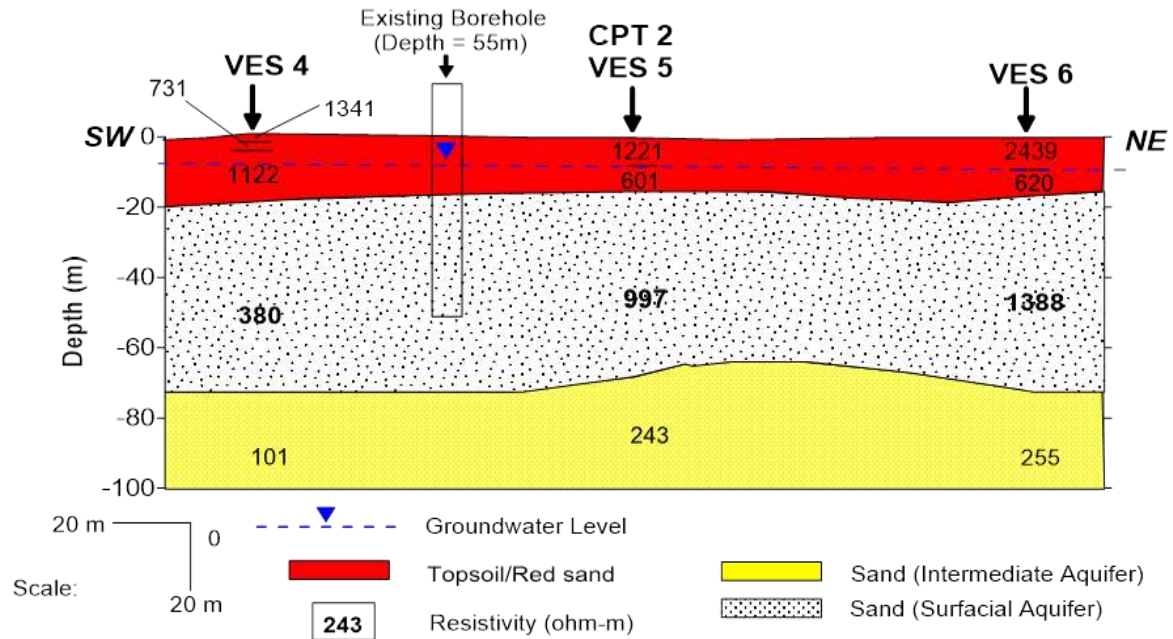

Figure 4 Geoelectric section along profile 2 in Igbekebo

4.2. Geotechnical Test

The results of the CPT and laboratory analysis of samples collected during trial test/soil examination is presented in Tables 2 and 3. The result shows increase in cone resistance and sleeve resistance with depth, ranging from 8 – 60 kg/cm^2 and 12 – 80 kg/cm^2 at CPT 1, and 10 - 50 kg/cm^2 and 12 - 55 kg/cm^2 at CPT 2 respectively. The friction ratio ranges from 1.07 – 1.50 (CPT 1) and 1.07 – 2.00 (CPT 2). The Robertson [41] soil chart classification shows a predominant zones 5 and 7 corresponding to clayey silt to silty clay and silty sand to sandy silt respectively (Fig. 5). The geologic section revealed sensitive fine grained clay material at 0.2 m depth. The plots of cone resistance and sleeve resistance against depth show high degree of heterogeneity in the upper 4 m, with alternation of sandy silt to clayey silt and silty sand to sandy silt. Consequently this result corroborates VES 2 which delineates high topsoil resistivity values. The geology (CPT 1) showed that, from 2.8 m depth the geology of the soil is a little bit consistent, and this would be the suitable/appropriate depth for foundation structure, in order to reduce differential settlement induced by soil inhomogeneity. At CPT 2, the geology is also alternating between sandy silt to clayey silt and silty sand to sandy silt. The upper 1 m is characterized by clayey silt to silty clay. Subsequently, placement of foundation basement below 1 m depth would be appropriate, with CPT value not less than 30 kg/cm^2.

The analyzed soil samples at both locations (Tables 2 and 3) shows liquid limits of 29.3 – 41.6 % (within recommended 50 %), plastic limits of 22.1 – 24.7% (within recommended 30 %), plasticity index of 8.2 – 17.53 % (within recommended 20 %), and shrinkage limits of 4.3 – 9.1 % (low/medium good soil quality). The natural moisture content ranges from 4.6 – 18.8 % which is low/moderate. The % gravel varies between 0.1 – 1.5, % sand is in between 22 – 56.6 %, silt varies from 19.0 – 43.5 %, clay ranges from 11.7 – 40.2 %. Generally the soil is composed of predominant sand and silt. The average clay content in the soil is less than 20 %. 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.

Table 2 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	8	12	1.50										
0.4	15	18	1.20										
0.6	18	20	1.11	33.3	22.2	11.12	9.1	11.3	0.1	36.9	28.9	34.1	2.69
0.8	18	20	1.11										
1.0	18	20	1.11										
1.2	20	30	1.50	36.6	22.1	14.50	7.9	11.8	0.2	44.6	43.5	11.7	2.64
1.4	28	32	1.14										
1.6	30	40	1.33										
1.8	32	40	1.25										
2.0	30	35	1.17	41.6	24.1	17.53	7.9	11.8	1.5	45.3	27.3	25.9	2.66
2.2	25	30	1.20										
2.4	30	35	1.17										
2.6	35	42	1.20										
2.8	42	45	1.07										
3.0	45	50	1.11	37.5	24.2	13.34	4.3	18.8	1.5	34.5	36.6	27.4	2.65
3.2	50	55	1.10										
3.4	50	60	1.20										
3.6	50	60	1.20										
3.8	58	65	1.12										
4.0	60	80	1.33										

Table 3 Geotechnical/Engineering Properties of Soil in Igbekebo (Location 2)

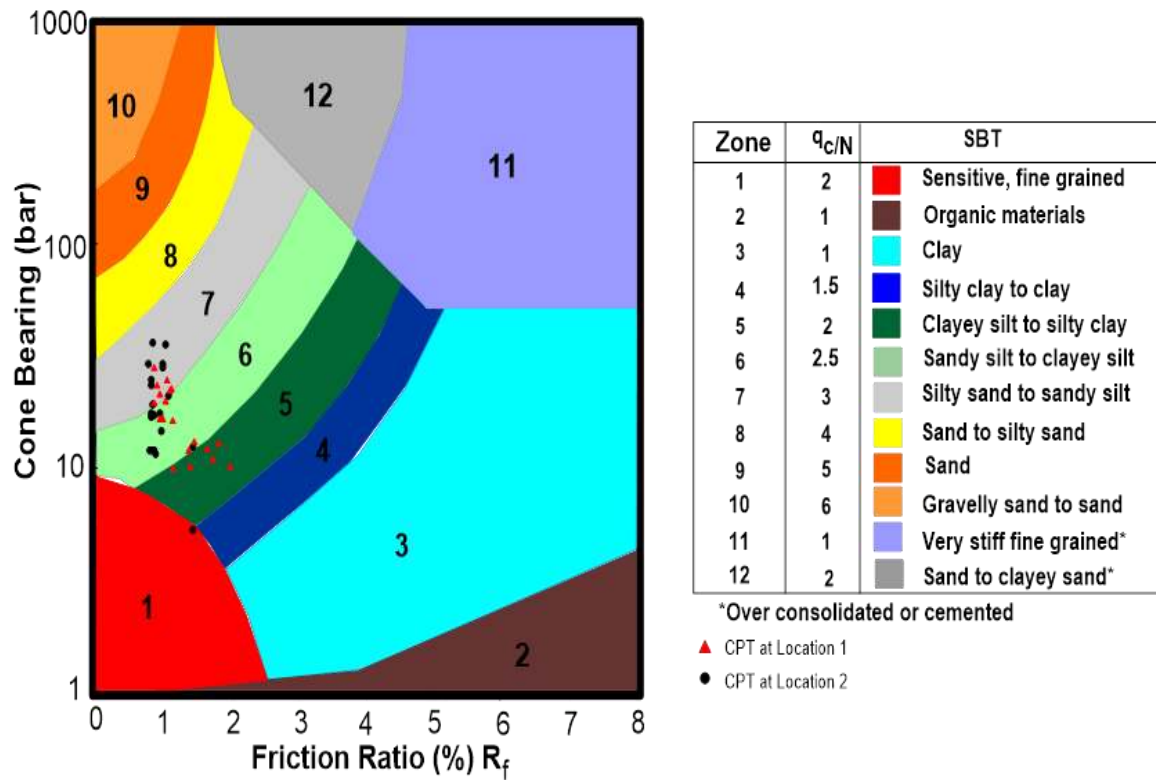| Depth (m)	Cone Resistance	Sleeve Resistance	Friction Ratio	L.L (%)	P.L (%)	P.I (%)	S.L (%)	M.C (%)	% Gravel	% Sand	% Silt	% Clay	S.G
0.2	10	12	1.20										
0.4	10	15	1.50										
0.6	10	20	2.00	41.2	24.4	16.80	8.6	11.1	1.4	56.6	19.0	23.0	2.64
0.8	12	22	1.83										
1.0	15	20	1.33										
1.2	18	25	1.39	36.3	19.3	17.00	7.9	6.20	1.2	41.9	19.5	37.4	2.66
1.4	20	28	1.40										
1.6	18	30	1.67										
1.8	20	35	1.75										
2.0	28	38	1.36	37.6	24.7	12.95	7.1	4.8	0.1	53	18.8	28.1	2.66
2.2	30	32	1.07										
2.4	30	35	1.17										
2.6	30	38	1.27										
2.8	32	35	1.09										
3.0	40	50	1.25	29.3	21.1	8.20	7.9	4.6	0.0	22	37.8	40.2	2.67
3.2	35	40	1.14										
3.4	38	45	1.18										
3.6	42	55	1.31										
3.8	40	55	1.38										
4.0	50	55	1.10										

Figure 5 Robertson's [41] Soil Classification Chart and Description of Different Zones for soil in the two localities

4.3. Bearing Capacity Determination

The ultimate and allowable bearing capacity estimated from the cone resistance using Meyerhof [28] equation as presented in Table 4. The calculated bearing capacities could be used in determining the foundation type for structures [40]. The allowable bearing of the soil varies between 20 to 147 KN/m² for CPT 1, and 74 – 368 KN/m² for CPT 2. Consequently an average allowable bearing capacity of 100 KN/m² (ultimate bearing capacity of 300 KN/m²) would be appropriate for design of shallow foundation in the area, at a depth not less than 2.8 m in location 1 and 3.6 m at location 2.

4.4. Settlement Analysis

The commonly accepted basis of design is that the total settlement of a footing should be restricted to about 25 mm [3, 42] 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 5). Although according to Meyerhof [28] and Schmertamnn [29] 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. The calculation of bearing capacities for strip and square foundation is shown in Table 6. For strip foundation, the appropriate (recommended) ultimate bearing and allowable bearing capacity for depth levels of 1 to 3 m vary from 662 – 1376 KN/m² and 221 - 459 KN/m² respectively, while square footing varies in between 1056 – 1923 KN/m² and 352 - 641 KN/m².
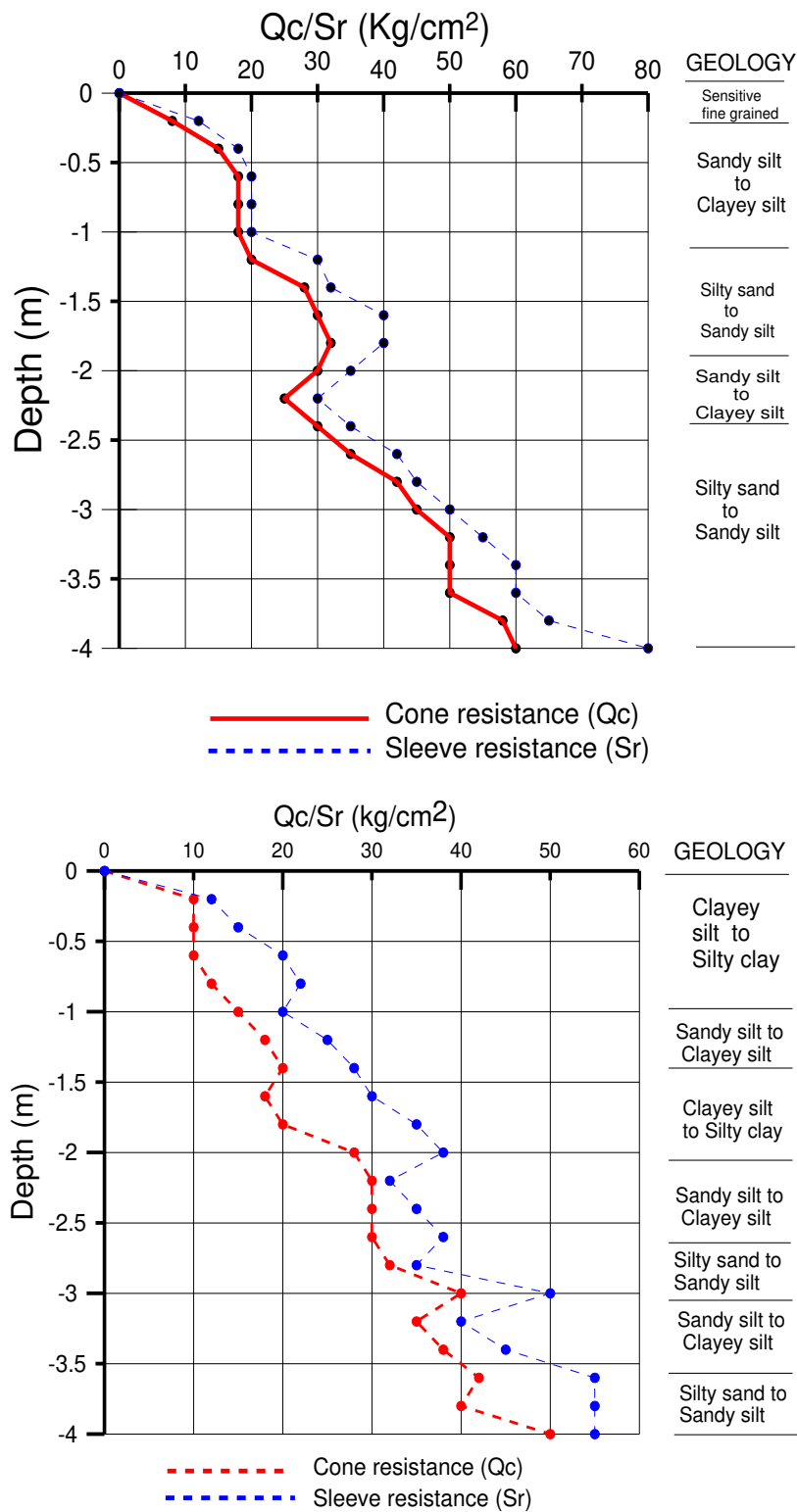
Figure 6 Plots of Cone resistance and sleeve resistance against depth at location 1 and 2, corresponding to (a) Ojuala CPT 1 (b) Igbekebo CPT 2

Table 4 Bearing Capacities estimated from the Cone resistance values for both sites (locations)

Depth (m)	CPT 1		CPT 2	
	$q_{allowable}$ (KN/m²)	$q_{ultimate}$ (KN/m²)	$q_{allowable}$ (KN/m²)	$q_{ultimate}$ (KN/m²)
0.2	20	59	25	74
0.4	37	110	25	74
0.6	44	132	25	74
0.8	44	132	29	88
1.0	44	132	37	110
1.2	49	147	44	132
1.4	69	206	49	147
1.6	74	221	44	132
1.8	78	235	49	147
2.0	74	221	69	206
2.2	61	184	74	221
2.4	74	221	74	221
2.6	86	257	74	221
2.8	103	309	78	235
3.0	110	331	98	294
3.2	123	368	86	257
3.4	123	368	93	279
3.6	123	368	103	309
3.8	142	426	98	294
4.0	147	441	123	368

Table 5 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	51.23	50.68	50.69
1.0	29.19	30.06	31.10
1.5	20.94	22.30	23.73
2.0	15.29	16.80	18.34
2.5	12.08	13.67	15.27
3.0	9.96	11.60	13.25

Table 6 Bearing Capacities for Strip and Square Shallow Foundations

Depth (m)	Strip	Square	Strip	Square
	$q_{ultimate}$ (KN/m^2)	$q_{ultimate}$ (KN/m^2)	$q_{allowable}$ (KN/m^2)	$q_{allowable}$ (KN/m^2)
1	662	1056	221	352
2	992	1456	331	485
3	1376	1923	459	641

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 66 – 118 KN, 212 – 357 KN, and 440 – 717 KN respectively. The allowable bearing capacity for bored piles ranges from 35 – 71 KN, 88 – 170 KN, and 159 – 297 at depth levels of 5 m, 10 m, and 15 m respectively (Table 7). Although it is recommended that pile load test be conducted before the design, construction and installation of pile [43] in the locality to ensure this proposed design alternatives be effectively and correctly used in the study area.

Table 7 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)
5	0.4	77	121	198	66	27	104	35
	0.5	120	151	271	90	34	155	52
	0.6	173	181	354	118	41	214	71
10	0.4	154	484	637	212	110	264	88
	0.5	240	604	845	282	137	378	126
	0.6	346	725	1072	357	165	511	170
15	0.4	231	1088	1319	440	247	478	159
	0.5	361	1360	1721	574	309	670	223
	0.6	519	1632	2151	717	371	890	297

5. Conclusion

The study integrated vertical electrical geophysical sounding and geotechnical investigation to characterize and assess the soil material in Ese Odo area of Ondo State for foundation design. According to Robertson's chart, the distribution of the CPT data obtained encompasses two zones 5 and 7 corresponding to clayey silt to silty clay and silty sand to sandy silt. The VES shows a predominant HK and KHK curve types which account for 33.3% each. These kind of curves imply stability at shallow depth. The upper 5 m is characterized by moderate thickness and high resistivity values to sustain the structure. The thickness (3.3 – 9.6 m) would dissipate the influence of structural load effectively. The average bearing capacity of 100 KN/m^2 could be adopted in the area for design of bases/footings for shallow foundation at a depth not less than 2.8 m. The total settlement is less than 50 mm tolerable limit for foundation width ranging from 0.5 to 3.0 m. The appropriate (recommended) allowable bearing capacity for strip foundation is 221 - 459 KN/m^2, while square footing varies from 352 - 641 KN/m^2. For driven pile at 5 m, 10 m, and 15 m, the adopted allowable capacity varies between 66 – 118 KN, 212 – 357 KN, and 440 – 717 KN respectively. The allowable bearing capacity for bored piles ranges from 35 – 71 KN, 88 – 170 KN, and 159 – 297 at depth levels of 5 m, 10 m, and 15 m respectively.

Compliance with ethical standards

Disclosure of conflict of interest

The authors declare that they have no conflict of interest.

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