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Assessment of Subsoil Suitability for Shallow Foundation Design at Part of Ibadan Area, Southwestern Nigeria, using Geophysical and Geotechnical Techniques

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Abstract

In the present study, geophysical and geotechnical techniques were applied to investigate the suitability of the subsoil for foundation design in part of Ibadan, Southwestern Nigeria. Vertical electrical resistivity and cone penetration tests were performed at seven points, and two samples each at 0.5 m (disturbed) and 1.5 m (undisturbed) were randomly collected at five (5) locations within the site. Three geo-electric layers exhibiting H-type curve patterns were observed for the VES sections: top soil (89.10-253.80 m, 1.38 m), weathered basement (24.00-50.10 m, 8.62 m), and fractured/fresh basement (190.80-585.00 m, depth-rock head = 9.84 m). Based on their average resistivities, these layers were classified as moderately competent, incompetent, and competent. CPT data shows allowable bearing capacity, qa (190.89-594.00 KN/m^2), allowable bearing pressure, ABP (63.63-198.00 KN/m^2), and ultimate bearing capacity, qu (212.10-660.00 KN/m^2) >100 KN/m^2 between 8.0-1.4 m where clayey-sand dominates would serve well as foundation bases. Gradation test (GT) analysis shows that >35% of particles pass sieve #200 at 0.5 m, while average Liquid limit, Plasticity limit, Plasticity Index, and Natural moisture content are 34.2%, 15.0%, 19.0%, and 9.6%, respectively, indicating low plasticity and compressibility soils classified as low liquid limit clay (CL) and A-7 (A-7-6 and A-7-5) soils according to the Unified Soil Classification System and Association of American State and Highway Transport Official. Additionally, samples taken at 1.5 m depth exhibited an average bulk density of 2.10 Mg/m^3, compatible with materials that have osmotic swelling capabilities. Cohesion (C) and angle of internal friction (ϕ) averaged 72.6 KN/m^2 and 18.8°, respectively, indicating good shear strengths. Finally, under increasing pressures (50-100, 100-200, and 200-300 KN/m^2), the coefficient of volume compressibility and coefficient of consolidation stay reasonably constant, indicating a material with low to moderate deformation on loading.

Keywords: *VES, CPT, Foundation, Shear strengths, Consistency Limits, Ibadan, Southwestern Nigeria.*

1. Introduction

Foundation failure is one of the underlying causes of building collapses, implying that the major cause of foundation failures is a lack of awareness of the subterranean condition (Abam, 2018). Civil building failure has been a recurring problem in today's world, particularly in coastal areas where fine-grained soil aggregates in contact with groundwater bodies dominate the supporting grounds (foundation materials). Uncertainties related to structural design and planning, on the other hand, play a part in such failures. Unknown soil properties are among the most important design uncertainties (Bremmer, 1999), but others, such as the non-linear behavior of soil under stress, the difficulty of estimating soil properties in undisturbed or in-situ conditions, and high spatial variation, all add to the problem. To perform optimally, a foundation must be safe from overall shear failure in the soil that supports it and not experience excessive settlement in comparison to the proposed structure's tolerance. This necessitates pre-foundation studies with suitable safety factors before foundation design to minimize structural lapses in terms of loss of life, litigation, and/or property destruction.

Structures in civil engineering interact with the ground, and some are made of earth-derived materials (Imeokparia and Falowo, 2019). Soil and rocks are still widely used in foundations, dams, and embankments in their natural state. For structures such as foundations, roadways, and tunnels, soil-structure interaction must be studied; for earth structures such as earth dams and slopes, good concepts must be developed on which to base studies (Atkinson, 1993; Imeokparia and Falowo, 2019). One of the most essential difficulties in foundation engineering, for example, is determining bearing capacity and behavior under stress. Although a country's building code stipulates the maximum allowable settlement for a certain structure, differential settlement is nonetheless possible.

The structural requirements, subsurface conditions, site characteristics, and economics are all analyzed and determined when selecting a foundation (Imeokparia and Falowo, 2019). A solely geotechnical investigation will not be able to generate a sufficient dataset to adequately define foundation soils. When the geology is complicated, this is frequently the case. To develop a reliable subterranean model for a proposed building site, a rigorous step-by-step strategy

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of subsurface exploration comprising the integration of geophysical and geotechnical techniques is necessary.

Direct measurements of soil parameters, either in situ or on soil samples in the laboratory, are used in geotechnical site evaluations. The most often utilized dynamic and static in-situ penetration tests in geo-engineering investigations (Baldi et al., 1995) are the standard penetration test (SPT) and cone penetration test (CPT), which are usually followed by the determination of soil physical parameters. Geophysical methods, on the other hand, are low-cost and quick to implement (Savvaidis et al., 1999; Luna and Jadi, 2000; Venkateswara, 2004; Olorunfemi et al., 2005; Soupios et al., 2005; Aizebeokhai et al., 2017; Oyeyemi, et al., 2015a; Imeokparia and Falowo, 2019; Oyeyemi et al., 2020).

Such integration of subsoil geotechnical and geophysical investigation exercises is required to have adequate knowledge of the engineering properties of subsoil materials that would have direct interaction with the proposed structure in the study area (Oke et al., 2009), especially in areas with complex geology that can result in inhomogeneity in foundation soils and rocks, which a purely geotechnical approach cannot provide (Oke et al., 2009). As a result, the goal of this work is to define the subsurface geological sequences/structures, as well as their resistivities, geotechnical characteristics, and overall site integrity for a building project. Findings from this study will aid in making informed decisions about the depth and type of foundation that should be used on the site to avoid possible failure, loss of life, and environmental damage.

2. Geology and Description of the Study Area

The study area is bounded between latitude 7° 26' 30.9'' N to 7° 27' 15'' N and longitude 3° 55' 08'' E to 3° 56' 10.1'' E located within the Basement Complex (BC) of southwestern Nigeria, composed of four main lithological units (Anifowose and Borode, 2007; Ayodele, 2015). These units (Fig.1) include; quartzite, quartz-schist of the meta-sedimentary series, the migmatites complex (banded gneiss, augen gneisses, and granite-gneiss), and variably migmatized biotite-hornblende gneiss with intruded pegmatites, quartz veins, aplites and dolerite dykes (Burke et al., 1976). Structural discontinuities run perpendicular and across the general rock foliation (NNE-SSW), characterizing the basement rocks. Some of these fractures are filled with dark-grey, unmetamorphosed amphibolitic dykes or quartzitic and quartzo-feldspathic intrusions. The subsurface succession of a typical weathered profile (topsoil, lateritic soil, saprolitic horizon, sap-rock/fractured basement, and fresh basement) in Ibadan agrees with a typical Basement Complex environment (Olayinka and Yaramanci, 1999; Tijani et al., 2009).

In basement complex terrains, the occurrence of groundwater depends on thick weathered overburden and deeply fractured zones. Such weathered overburden provides high storativity while the fractures account for their high permeability (Guiheneuf et al., 2014). However, hydraulic permeability is likely to be low when the regolith is derived from rocks rich in ferromagnesian minerals, notably biotites and feldspars which convert quickly to hydrobiotite and clays, producing low permeability regolith (Graham et al., 2010). Locally, the study site is underlain by banded gneiss (Fig.1) and can host the groundwater in an unconfined condition; otherwise, they are semi-confined to confined conditions (Olayinka and Yaramanci, 1999).

The area with its characteristic rainforest vegetation exhibits a typical tropical climate of averagely high temperature, high relative humidity, and generally two rainfall maxima regimes during the rainfall period of March to October. The dry season extends from November to February, while the rainy season extends from March to October (Oyinloye and Modebola-Fadimine, 2013). The mean temperature is highest at the end of the Harmattan (averaging 28°C), from the middle of January to the onset of the rains in the middle of March (Iloeje, 1981).

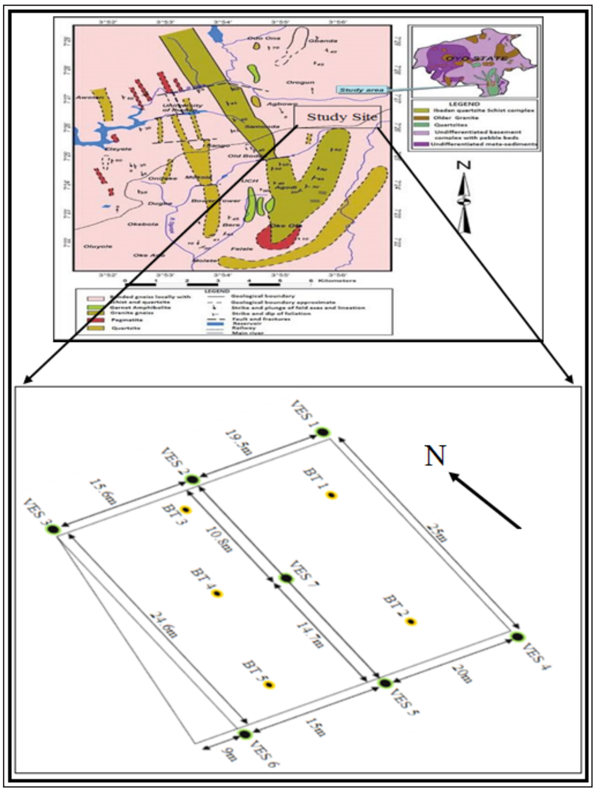


Figure 1. Geological Map *(Amanambu, 2015)* and Site Sketch showing VES/CPT and Sampling Points.

3. Data Acquisition and Analysis

3.1 Geophysical Survey

Earth materials' in-situ qualities and structural traits can be measured with surface geophysical methods (Lowrie, 2007). By reducing design uncertainty and lowering inquiry expenses, such strategies have demonstrated cost efficiencies (Willian, 2010).

The Allied Omega geophysical Terrameter was used to probe seven (7) vertical electrical sounding (VES) stations (Fig.1) in conjunction with the cone penetrometer test (CPT) stations. The Schlumberger array (Fig. 2) was adopted, with maximum AB/2 = 80 m and MN/2 = 5 m however, MN≤1/5AB was maintained as the geometric relationship between MN and AB. On bi-logarithmic graph sheets, the measured apparent resistivities, ρa, were plotted against AB/2 on ordinate and abscissa respectively. To obtain the layer parameters, ρa was first manually processed and quantitatively analyzed using a partial curve matching technique. To obtain the layered apparent resistivity and estimated thickness, the resultant curves were interpreted qualitatively through a visual examination and quantitatively through a partial curve matching technique using Win-RESIST software (Vander-Velpen, 2004).

Furthermore, interpretation was done keeping in mind the ideal depth of investigation equal to one-third (1/3) of the current electrode spacing at the inflection point (Tijani et al., 2021). Finally, geoelectric sections were constructed from the generated layer parameters for in-depth characterization of the subsurface environment and correlation.

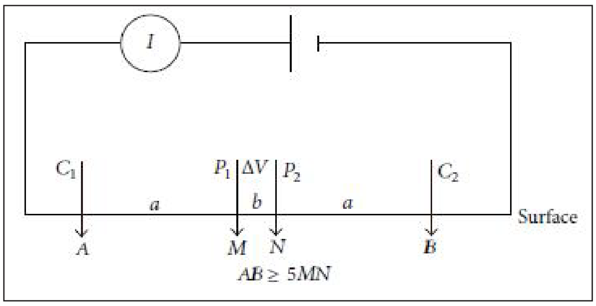


Figure 2. Schematic display of the Schlumberger electrode arrangement

3.2 Cone Penetration Test (CPT)

CPT is a quasi-static penetration test that uses a 60 % steel cone to determine the ground's penetration resistance at a specific point. A ten (10) ton capacity testing machine was deployed here. A cylindrical probe with a cross-sectional area of 1000 mm^2 and a conic head with a 60^o apex angle make up the equipment. The test was conducted by anchoring the winch frame to the earth, providing the necessary power to push the cone into the earth (Coerts, 1966). By exerting pressure on the outer sounding tube, the probe is pushed down into the earth at a continuous pace of around 2 cm/s in the closed position. At the same time, the penetration resistance is measured at predetermined intervals from the existing ground level down to refusal depths. Resistance to the penetration of the cone registered on the pressure gauge connected to the pressure capsule is recorded. The tube is then pushed down, and the procedure described above is repeated. Tests are usually terminated when dense sands or rock unit is encountered or when there is an excessive vertical misalignment, and the support anchors of the machine lift off the ground. Equation (1) which covers all foundations irrespective of the width according to Meyerhof (1974) was adopted for the estimation of allowable bearing capacity from the cone tip resistance value, qc:

$q_a = 2.7\, q_c$ (KN/m^2) …..……………………........………….. (1)

Where; q_a is the allowable bearing capacity;

And qc is the cone penetration resistance value.

The allowable bearing pressure (ABP) was calculated by multiplying the factor of safety (equal 3) on the allowable bearing capacity, while the ultimate bearing capacity (qu) was calculated by multiplying the allowable bearing capacity by three (3) (Skempton and MacDonald, 1956). Using Microsoft Office Excel 2010, a resistance profile was created by plotting successive cone resistance readings against depth. The inflection points of the obtained penetrometer curves were interpreted as the boundary between the distinct lithologies, whereas layer sequences were interpreted from variations in cone tip resistance with depth.

The results of the geophysical surveys were used to choose the CPT points/locations. VES and CPT were followed by a random collection of two samples per pit at 0.5 m (disturbed) and 1.5 m (undisturbed) depths for five (5) sampling points to ensure a thorough exploration of the study site (Fig.1). Manual sampling was carried out with a pick axe and shovel, as well as a sealable polyethylene bag (for disturbed samples) and PVC pipe measuring 76 mm x 38 mm (undisturbed samples).

The water level was not monitored because no groundwater was intercepted. The sampling process and specification outlined by British Standard Institute (BSI 1377, 1990) for geotechnical soil sampling were properly followed. All sampling took place between January and February, during the dry season.

3.3 Geotechnical Laboratory Tests (GLT)

The soil samples were subjected to the following laboratory tests: consistency limits (liquid limit, LL; plastic limit, PL; and plasticity index, PI), grain size distribution (dry sieving) analysis, unit weight determination (bulk density, ɣB), moisture content, the undrained triaxial test was performed to compute shear strength parameters (angle of internal friction, ϕ; and cohesion, C) of the soil samples were obtained from the relationship between the principal stresses at failure. At 50-100 KN/m^2, 100-200 KN/m^2, and 200-300 KN/m^2 stress ranges, an Oedometer consolidation test was performed to measure the coefficient of volume compressibility, Mv; and coefficient of consolidation, Cv for settlement characteristics. These tests were carried out adhering to the British Standard Institution 1377 (1990) for testing material used for all laboratory tests.

4. Results and Discussion

Table 1 summarizes the findings of an interpreted geophysical research, whereas tables 4 and 5 indicate theoretically estimated levels of bearing capacity based on cone resistance, qc values, and geotechnical data, respectively.

4.1 Geophysical Studies

Table 1 depicts the subsurface lithology it penetrates (Sattar *et al*., 2004), with small differences expected in a normal geophysical result (Tijani et al., 2021). Three subsurface layers (top soils, weathered basements, and fractured/fresh basements) with distinct H-curve patterns were shown by VES curves (Fig.3). Top soil revealed relatively high resistivity materials (89.10-253.80 Ωm), indicating reworked/artificially compacted top soils, whereas the weathered basement revealed low resistivities (24.00-50.10 Ωm), indicating a very saturated medium composed of clay materials (Arora, 2008), potentially linked to poor drainage conditions (Giza and Igwe, 2018). Resistivity values in the fractured/fresh basement ranged from 190.80

to 585.00 Ωm. The first layers (top soil) are slightly thicker than 1.0 m for VES 2 and 4 (Table 1), and primarily consist of clay, sandy clay, and lateritic lithologies whose constituent minerals (silicates, feldspar, micas, iron, and aluminum) may not readily favour foundation founding due to expansion (seasonal volume fluctuations). Except where an appreciable thickness of lateritic materials is encountered, basic ground improvement by ripping off and backfilling with a more admixture of granulated and cohesive materials to aid drainage and increase shear strength is required to erect structures on/within the first layers. Laterites are rich in iron and aluminum, and thus are firm and physically resistant (Hill et al., 2000; Agada et al., 2017). Furthermore, the weathered basement or saprolitic horizon, which has an average thickness of 8.62 m, may not serve well because it may exacerbate excessive pore pressure development caused by poor drainage, resulting in significant effective stress decreases and foundation instability in the study site. A typical depth-rock head in the range of 6.7-13.9 m was discovered for the fractured/fresh basement (ave. 9.84 m). Therefore, based on their average resistivities, top soils (163.68 Ωm) are fairly competent, weathered units are incompetent (35.49 Ωm), and fresh/fractured basements are competent (378.00 Ωm) as per Sherrif (1991) classification for foundation materials (Table 2).

The topsoil is relatively competent but very thin (Fig.4a and b), which is consistent with a previous study in Akure Metropolis by Ojo et al., (2015), which found topsoil thickness ranging from 0.3 to 5.2 m and resistivity of 15.0-7,133 Ωm. Although the thickness of the soil layer, among other things, affects bearing capacity (Mosallanezhad and Moayedi, 2017), the measured mean thickness (8.6 m) of the overburdened/weathered soil on the basement will nevertheless help to distribute the foundation load evenly.

4.1.1 Evaluation of Soil Corrosivity

Electrical current-carrying materials used in civil engineering projects, whether at the beginning or end, are prone to deterioration, necessitating proper soil assessments to minimize corrosion. Soil corrosivity in the research site range from "moderately" to "slightly" to "practically non-corrosive" (Oladapo et al., 2004; Mosura et al., 2017), (Table 3). Table 3 shows that the second geoelectric layers of VES 1-7 have a moderately corrosive potential, whereas VES 4-7 has a slightly corrosive rating associated with its first geoelectric layer, and a non-corrosivity potential is attributed to the third geoelectric layers linked to VES 1-7, except for VES 1 and 2, which have non-corrosivity potentials associated with the first (1) and second (2) geoelectric layers, respectively.

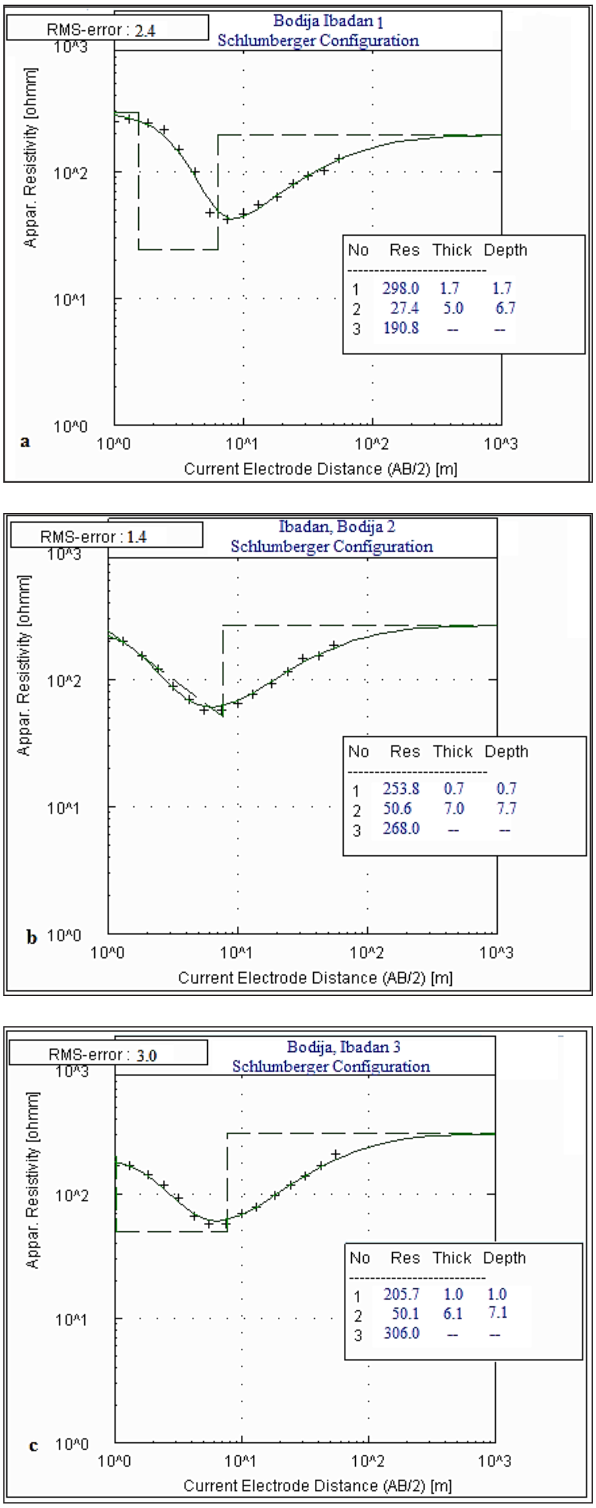
Table 1. Geoelectrical layer properties and inferred lithological units

VES No.	Resistivity (ohm-m)	Layer Parameters		Layers	Curve Type	Inferred Lithology
		Thickness (m)	Depth (m)			
1	298.00	1.70	1.70	1	H	Top soil
	27.40	5.00	6.70	2		Weathered basement
	190.80	-	-	3		Fractured/Fresh basement
2	253.80	0.70	0.70	1	H	Top soil
	50.00	7.00	7.70	2		Weathered basement
	268.00	-	-	3		Fresh/fractured basement
3	205.70	1.00	1.00	1	H	Top soil
	50.10	6.10	7.10	2		Weathered basement
	306.00	-	-	3		Fractured/Fresh basement
4	149.70	0.60	0.60	1	H	Top soil
	24.00	8.70	9.30	2		Weathered basement
	466.90	-	-	3		Fractured/Fresh basement
5	117.80	1.50	1.50	1	H	Top soil
	34.60	9.90	11.90	2		Weathered basement
	343.90	-	-	3		Fractured/Fresh basement
6	89.10	1.80	1.80	1	H	Top soil
	30.30	12.10	13.90	2		Weathered basement
	487.30	-	-	3		Fractured/Fresh basement
7	166.00	1.70	1.70	1	H	Top soil
	32.00	11.60	12.30	2		Weathered basement
	585.00	-	-	3		Fractured/Fresh basement

The corrosivity potentials of the soils in the site differ among the various geoelectric layers, making metallic pipes buried inside the slightly (second layers) to moderately (first layers) corrosive layers more sensitive to corrosion and eventual failure. As a result, underground metal storage tanks galvanized pipes, and steel pipes can be buried at the third layer (>180 Ω m) without the risk of possible chemical corrosion, as metal or steel structures within this layer are mostly unaffected by corrosion.

Clay materials have low electrical resistivity and great electrical conductivity, with resistivity values ranging from

1 to 100 Ωm. At 1-6 m depth below the surface within which the electrical materials could be earthed have resistivity values ranging from 27.40-50.10 Ωm (Table 1). These clayey soils are a good medium for earthen depth to absorb any excess charge.



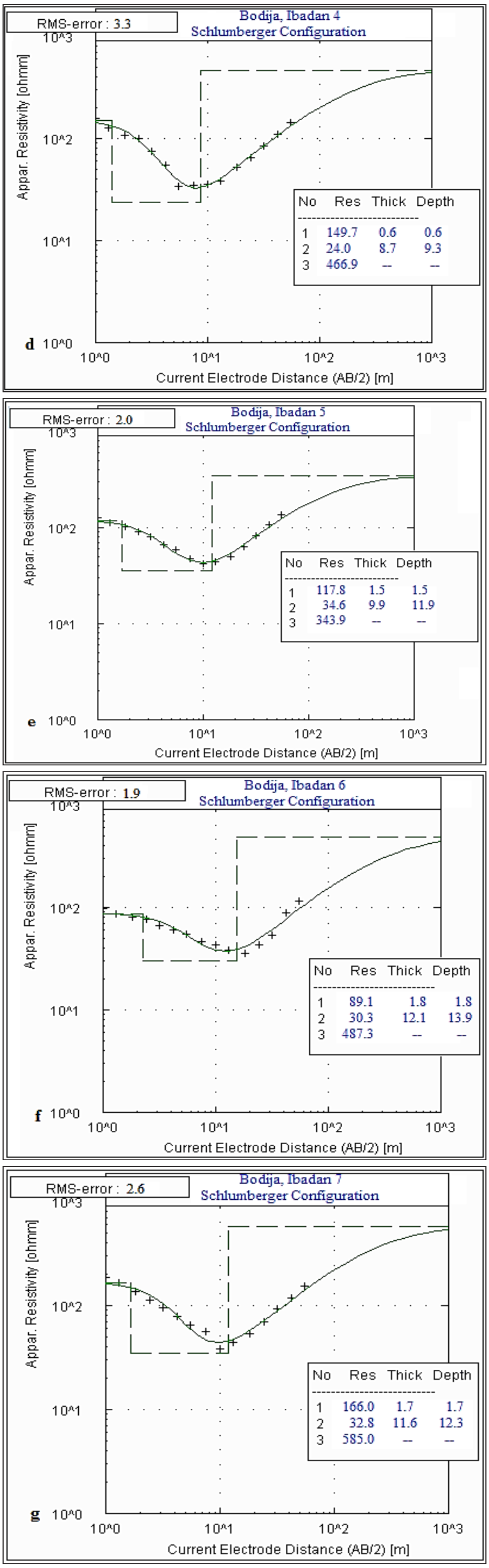


Figure 3. a-g. Inverted VES models associated with their RMS error for the Investigated points

4.2 Cone Penetration Test (CPT) and Geotechnical Studies

4.2.1 Cone Penetrometer Test (CPT)

The cone penetrometer test is a simple, accurate, and quick way of measuring various degrees of bearing capacity, stratigraphic correlation, and soil deformation characteristics. A further inspection of the displayed CPT data revealed outlines/curves that corresponded to three to four geologic layers (Fig.5a-g).

Table 1. Lithologic competence rating in terms of apparent values (Sherrif, 1991)

ρ_a rating (Ωm)	Lithology	Competence rating
<100	Clay	Incompetent
100- 350	Sandy clay	Moderately competent
350-750	Clayey sand	Competent
>750	Sand/laterite/bedrock	Highly competent

Here, Fig.5a depicts a subsurface environment with clay materials at the surface (0.0-0.2 m), clay/silty clay at 0.2-0.4 m, sandy clay with a thickness of 0.18 m (0.4-0.58 m), and clayey sand material at 0.6 m down to the refusal depth (1.0 m).

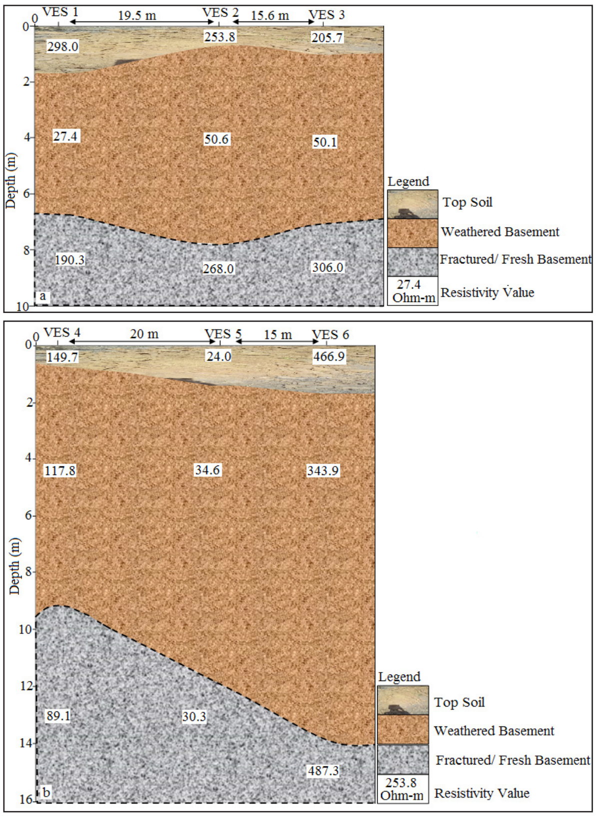


Figure 4. Constructed Geoelectric Sections of the Study Site

Table 3. Soil Corrosivity Ratings According to Oladapo et al., (2004) and Mosura et al., (2017)

S/N	Soil Resistivity Range (Ωm)	Soil Corrosivity	VES Points	Geoelectric Layers
1	<10	Very Strongly	Nil	-
2	10-60	Moderate	1, 2.3, 4, 5, 6, and 7	2, 2, 2, 2, 2, 2 and 2
3	60-180	Slightly	4, 5, 6 and 7	1, 1, 1, 1
4	>180above	Practically noncorrosive	1, 2, 3, 4, 5, 6 and 7	1 and 2, 3, 3, 3, 3, 3

Table 4. Theoretically Estimated levels of Bearing Capacities from Cone Tip Resistance, qc

S/N	Depth (m)	Average qc (kg/cm^2)	Average q_a (KN/m^2)	Average ABP (KN/m^2)	Average q_u (KN/m^2)
1	0.00	0.00	0.00	0.00	0.00
2	-0.20	26.40	71.28	23.76	79.20
3	-0.40	33.50	90.45	30.15	100.5
4	-0.60	59.30	160.11	53.37	177.90
5	-0.80	70.70	190.89	63.63	212.10
6	-1.00	157.10	424.17	141.39	471.30
7	-1.20	148.30	400.00	133.47	444.90
8	-1.40	220.00	594.00	198.00	660.00

q_c = cone tip resistance, q_a=allowable bearing capacity, ABP= allowable bearing pressure, q_u=ultimate bearing capacity, and factor of safety = 3

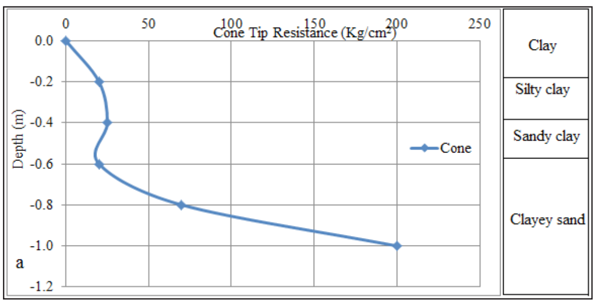
At 1 m, clayey sand lithology is competent and would be suitable for laying a shallow foundation. Similarly, CPT 2 revealed clay/silt clay material at 0-0.2 m, clay-silt with 0.6 m thickness at 0.2-0.4 m, sandy clay at 0.4-1.0 m, and clayey sand material with a thickness of 0.4 m at 0.4 to maximum depth (1.4 m), all of which could support shallow foundations within the site. The CPT curve in Fig.5c indicated three geologic layers: clay/clay silt at 0-0.4 m, clayey silt at 0.4-0.6 m, and sandy clay and clayey sand at 0.6-0.8 m and 0.8-1.2 m, respectively, which support the results of other CPTs. The fourth (4th) CPT point (Fig.5d) revealed a 0.2 m thick clay top soil, followed by silty clay in the range of 0.2-0.8 m, sandy clay at 0.8-1.0 m, and clayey sand at 1.0-1.4 m below the surface. Similarly, CPT 5 exhibits a four-layer zonation, with clay/silty clay at 0-0.4 m, clay silt at 0.4-0.6 m, sandy clay at 0.6-1.0 m, and the last horizon (clayey sand material) with a thickness of 0.4 m lying at 1.0-1.2 m deep (Fig.5e). Further analysis found that CPT 6 has a subsurface soil profile defined by silt/silty clay top soil, sandy clay, and sand/clayey sand lithologies below the surface in the range of 0-0.2 m, 0.2-0.4 m, and 0.4-1.4 m, respectively (Fig.5f). Figure 5g (CPT 7) showed a subsurface soil profile with clay at a depth of 0-0.2 m, silty clay at 0.2-0.8 m, sandy clay at 0.8-1.0 m, and clayey sand at 1.0-1.4 m.

Although, except CPT 3, which was characterized by three sequences, the CPT results were associated with low to high allowable bearing capacities (71.3-594.0 KN/m^2), allowable bearing pressures (23.8-198.0 KN/m^2), and ultimate bearing capacities (79.2-660 KN/m^2) according to Bell (2007), revealing appropriate founding depths and supporting bases/ media for shallow foundations in the study site (Table 4). The allowable bearing capacities, allowable bearing pressures, and ultimate bearing capacities estimated at various depths (Table 4) corresponded to material strengths at such depths, such that ground penetration resistance decreases (<100 kg/cm^2) near the surface (0-0.8 m), but increases significantly (100 kg/cm^2-200 kg/cm^2) beyond 0.8 m where targeted CPT values exist. The allowable bearing capacities, allowable bearing pressures, and ultimate bearing capacities estimated at various depths (Table 4) corresponded with material strengths at such depths, such that ground penetration resistance decreases (100 kg/cm2) near the surface between 0-0.8 m, but increases significantly (100 kg/cm^2-200 kg/cm^2) beyond 0.8 m where targeted CPT values exist. The consequence is that where competent materials are available, foundations in the research site can be securely built beyond 0.8 m. Because there is no near-surface groundwater table, this is advantageous. As a result, the depth of footings subject to 25 mm total settlement as a frequently accepted basis for designs (Bell, 2007) should be at least 0.8 m below the surface.

4.2.2 Geotechnical Studies

The determination of a soil's physical property aids in the identification and classification of soils. Because particle size and distribution of pores within a soil matrix considerably influence soil stability (Bidyashwari et al., 2017) the particle size distribution of soil is an important predictor of its geotechnical features (Falowo, 2018). Table 5 shows that the proportions of particles passing sieves No. 4 (4.76 mm), 6 (3.36 mm), and 200 (0.075 mm) vary as 40.0-47.0, 13.0-25.0, 71.0-84.0, with average values of 44.0 %, 20.0 %, and 80.0 %, indicating high clayey material greater than 35 % recommended by British Standard (1990) as foundation support.

The soil's consistency limits in terms of Liquid limit, LL, Plastic limit, PL, and Plasticity index, PI, vary from 30.0-37.0, 12.0-17.0 %, and 17.0-21.0 %, with respective means of 34.2, 15.0 % and 19.0 % (Table 5). For natural soils, LL is a useful predictor of the shrink-swell potential (Sherrif, 1991). LL, PL, and PI all fall within the Federal Ministry of Works and Housing's foundation material restrictions (LL= 50 %, PL= 30 %, and PI= 20 %).



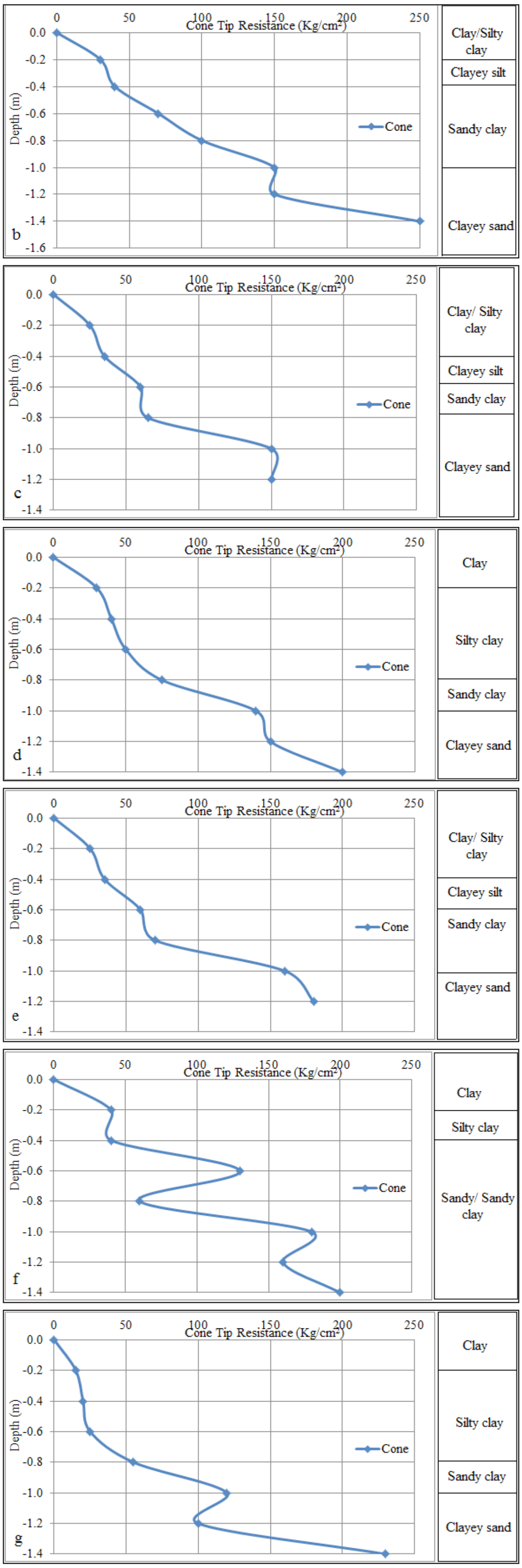


Figure 5. Depth (m) against Cone tip Resistance (kg/cm2)

Table 5. A Summary of Geotechnical Results

Disturbed Sample (0.5 m)										
Sample	% passing Sieves			Atterberg Limits (%)			NMC	AASHTO	UCS	Rating
	No.4	No.6	No.200	LL	PL	PI				
BT1	42.0	13.0	84.0	30.0	12.0	18.0	17.0	A-7-5	CL	Poor
BT2	47.0	21.0	82.0	37.0	17.0	20.0	11.0	A-7-6	CL	Poor
BT3	46.0	18.0	84.0	36.0	16.0	20.0	6.0	A-7-6	CL	Poor
BT4	40.0	23.0	71.0	31.0	14.0	17.0	11.0	A-7-6	CL	Poor
BT5	45.0	25.0	80.0	37.0	16.0	21.0	13.0	A-7-6	CL	Poor
Ave.	44.0	20.0	80.2	34.2	15.0	19.2	15.4			

Undisturbed Sample (1.5 m)						
Sample	Bulk Density (Mg/m^3)	Shear strength Parameters		Consolidation parameters		
		C (KN/m^2)	Φ (°)	Stress range	Mv (m^2/MN)	Cv (m^2/yr)
BT1	2.2	72.0	21.0	50-100	0.137	3.90
				100-200	0.125	3.90
				200-300	0.107	3.90
BT2	2.1	73.0	19.0	50-100	0.135	3.60
				100-200	0.122	3.70
				200-300	0.104	3.60
BT3	2.1	73.0	18.0	50-100	0.134	3.50
				100-200	0.120	3.50
				200-300	0.102	3.50
BT4	2.1	72.0	18.0	50-100	0.135	3.60
				100-200	0.121	3.60
				200-300	0.102	3.70
BT5	2.1	73.0	18.0	50-100	0.131	3.60
				100-200	0.119	3.60
Ave.	2.10	72.60	18.80	200-300	0.102	3.60

Sowers and Sowers (1970) characterized soils with a PI greater than 31% as extremely plastic, highly compressible, with low permeability, and low hydraulic conductivity. As a result, all samples analyzed from the study site fit into this category, and they would be ideal as supporting materials for shallow foundations, according to PI. Many attributes of clay and silt can be matched with the consistency limit using the plasticity chart (Fig.6), used in classifying fine-grain materials. The plasticity chart revealed a CL soil class above the A-line (Fig.6), indicating that the soils are primarily composed of inorganic materials with intermediate plasticity and compressibility and would have medium expansive potential (Chen, 1975; Peck et al., 1974) as a result of associated clay mineral content (Kalinski, 2011). This is in sync with Imeokparia and Falowo, (2019) studies carried out in a similar basement complex area of Owo, southwestern Nigeria. This is consistent with the findings of Imeokparia and Falowo (2019) in a comparable basement complex area in Owo, southwestern Nigeria. Inorganic clay elements of low-medium plasticity are also generally found in these soils, according to the Unified Soil Classification System (USCS). Moreso, according to the AASHTO (1982) classification system, these soils were similarly categorized as poor (A-7; A-7-5 to A-7-6) foundation materials with shrink-swell potential with moisture changes. Moisture content (NMC) depicts the clay content and type of soil material, by measuring the water-holding capability of soils (Sowers and Sowers, 1970). The soil's NMC soil range from 6.0-13.0 %, with an average of 9.6 %, indicative of a moderately plastic material (Underwood, 1967), also accords with the soil's plasticity (intermediate).

Similarly, the physical qualities of disturbed soil (bulk density, and shear strengths) have a significant impact on its stability (Kitutu, 2009). These soil properties were employed by Bidyashwari et al., (2017) to characterize the nature and behavior of soils. Table 5 shows that the bulk density (γB) ranges from 2.1 to 2.2 Mg/m^3, with an average value of 2.1 Mg/m^3. The γB values obtained here are consistent with (Seedman, 1986) observations that osmotic swelling of clay-composed materials occurs in samples with a bulk density of less than 2.45 Mg/m^3.

A material's shear strength refers to its capacity to withstand shearing deformational pressures (Sowers, 1963). C range from 72 KN/m^2-73 KN/m^2, while ϕ range from 18.0°-21.0°, averaging 72.6 KN/m^2 and 18.8° respectively (Table 5). The high values of C and ϕ are owing to an excellent clay-sand mixture, in which the clays supply the requisite cohesiveness (C) and the sands provide good frictional contact between the particles-thus, higher frictional strength is achievable by combining both C and ϕ (Idris and Igwe, 2018).

Consolidation has long been thought to be a fundamental phenomenon that effectively explains soil behavior in foundation issues (Adebisi and Adeyemi, 2012). Table 5 shows the results of the consolation test under various stress levels. Mv and Cv, which are the consolidation parameters, remain fairly constant with applied pressure such that at stress ranges of 50-100, 100-200, and 200-300 KN/m^2, Mv range from 1.31x10^{-1}-1.37x10^{-1} m/MN, 1.20x10^{-1}-2.51x10^{-1} m/MN and 1.02-1.07x10^{-1} m/MN, indicating very low compressibility (Mckinlay, 1996), with no likelihood for differential settlement of the structure while Cv also remains fairly constant (3.50-3.90 m^2/yr) at all pressure range. The implication is that when the site is loaded, the soil will have a modest consolidation settlement.

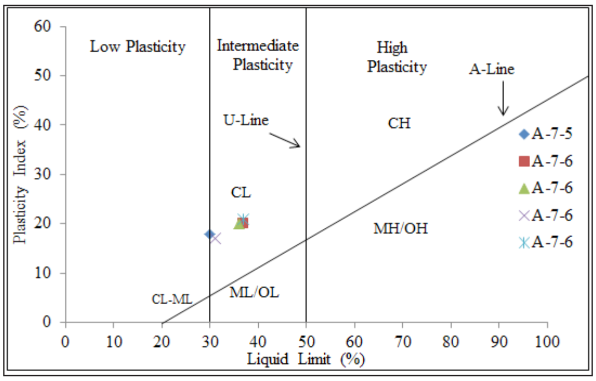


Figure 6. Plasticity chart showing the study soil samples.

Table 6. Soil Expansion Relative to Liquid and Plasticity Index.

S/N	Chen, (1975)		Peck *et al.*,(1974)		Expansion	Samples
	Liquid limit	Plasticity Index	Plasticity Index	Swelling Potential		
1	<30	0-15	0-15	Low	Low	Nil
2	30-40	10-35	10-35	Medium	Medium	1, 2, 3, 4 and 5
3	40-60	20-55	20-35	High	High	Nil
4	>60	>35	Above 35	Very High	Very high	Nil

Furthermore, the compressibility index (Eqn. 2), Cc expression given by (Terzaghi and Peck, 1969) for naturally cemented clays of low to moderate sensitivity modified after (Skempton and Northey, 1952), was used to hypothetically predict the susceptibility of these soils to settlement.

$$Cc = 0.009\ (W_L-10) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Where Cc = Compressibility index

WL= Liquid limit in percentage

Here in this study, Cc has been found to range from low to moderate (Table 7) however, some of the soils have a moderate LL, which may cause a moderate amount of foundation settling. Compaction can improve the soil's suitability as a foundation because compaction minimizes void spaces, pore pressure, and its implications in construction projects, boosting its applicability, notably as fills (Aghamelu *et al.*, 2011).

Table 7. Soil Compressibility Analyses According to Terzaghi and Peck (1969).

S/N	Liquid Limit (%)	Compressibility Index; Cc	Class
1	30.0	0.18	Low
2	37.0	0.24	Moderate
3	36.0	0.23	Moderate
4	31.0	0.19	Low
5	37.0	0.24	Moderate

5. Conclusion and Recommendation

To define the sub-soil environment suitability for a proposed building foundation design, geophysical (vertical electrical sounding; VES) and geotechnical techniques (Cone penetration test; CPT, consistency test, gradation, shear strength, and consolidation analysis) were used. The VES data revealed three (3) geoelectric layers with distinctive H-curve patterns: topsoil (89.10-253.80 Ωm, 1.38 m), weathered basement (24.00-50.10 Ωm, 8.62m), and fractured/fresh basement (190.80-585.00 Ωm, depth-rock head = 9.84 m). However, the top soils (163.68 Ωm), weathered basement (35.49 Ωm), and fractured/fresh basement units (378 Ωm) were classed as somewhat competent, incompetent, and competent, respectively, based on mean resistivities. Although the topsoil is moderately competent but has a thin thickness (only 2 m), the weathered materials in the basement have a massive thickness (9.8 m) that may aid in the even distribution of foundation loads.

The CPT curves reflected a subsurface environment with clay, silty clay, sandy clay, and clayey sand lithologies of varying thicknesses and penetration resistances, as well as different ranges of allowable bearing capacities, qa allowable bearing pressures, ABC and ultimate bearing capacities, such as 79.2-660 KN/m^2, 71.3-594.0 KN/m^2, and 23.8-198.0 KN/m^2. From 0.8 m depth, clayey-sand materials with reasonable bearing capacity values that would optimally function as foundation materials were discovered, with qc >100 kg/cm^2. Grain size distribution indicates clay-dominated materials with >35% particles passing the No.200 sieve) recommendation for materials for use as foundation support by the Federal Ministry of Works and Housing (1972). The typical results for LL, PL, PI, and NMC were 34.2 percent, 15.0 percent, 19.0 percent, and 9.6 percent, indicating a low plastic and high metal content, respectively.

Materials formed of osmotic swelling potentials with B 2.45 Mg/m^3 have a bulk density of 2.1 Mg/m^3. C and ϕ denote a large proportion of clay-sand admixture, resulting in increased shear strength. Finally, the consolidation parameter; Mv remains fairly constant with changes in applied pressure (50-100, 100-200, and 200-300 KN/m^2) as 1.31x10^{-1}-1.37x10^{-1} m/MN, 1.20x10^{-1}-2.51x10^{-1} m/MN and 1.02-1.07x10^{-1} m/MN respectively indicating low compressible soils. Also, at all pressures, Cv is pretty stable (3.50-3.90 m2/yr). The implication is that when the soils are loaded, they will be prone to low to moderate deformation (low compressibility and consolidation).

As a result, where earth materials with suitable permitted such as qa, ABP and qu exist, a shallow foundation depth with a base of 0.8 m or above is advised.

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