Engineering Subsoil Exploration for Foundation Design and Planning of a proposed Filling Station in Ore Area of Ondo State

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Available online at: www.isroset.org

Received: 25/Apr/2020, Accepted: 19/May/2020, Online: 30/June/2020

Abstract—The study presents the usefulness of preconstruction engineering site investigation in the vicinity of proposed site for petroleum filling station in Ore, southwestern Nigeria using geoelectrical and geotechnical methods. The electrical resistivity utilized schlumberger configuration, and was complemented with in-situ geotechnical test and laboratory analysis of eleven soil samples at depths not exceeding 3.0 m. The results show that the topsoil resistivity (avg. 351ohm-m) is moderately competent/corrosive to sustain the structural load with appreciable thickness of 3.1 – 15.5 m for shallow foundation. The thickness of the weathered layer is characterized by thickness greater than 10 m in all places within the site and is mildly/slightly corrosive, hence, favours burial of storage fuel tanks. The settlement analysis for foundation width of 0.6 – 3.0 m at three depth levels of 1 m, 2 m and 3 m produced values between 9.46 – 47.98 mm. But foundation width above 0.6 m is recommended so as to have settlement value less than 25 mm. The calculation of bearing capacities for strip and square foundation within 0.6 – 1.4 m depth gives an allowable bearing capacity varying from 523 – 806 kPa and 719 - 1063 kPa respectively. An average allowable bearing capacity of 150 kPa is recommended and would be appropriate for design of shallow foundation in the site, at a depth not less than 1.0 m. Therefore the proposed architectural plan for the station is professionally acceptable.

Keywords—Bearing capacity, Foundation structure, Settlement analysis, Electrical resistivity, Square and strip footing

I. INTRODUCTION

Civil Engineering preconstruction investigation for structures such as tunnels, bridges, roads, dams, buildings has become an important process in civil engineering design and construction (1-2). It assist in generating subsurface models, it furnishes information on groundwater condition, will help in knowing the thickness and nature of earth material below the foundation of a proposed structure [3-4]. In addition, it forms an important aspect of civil engineering construction [5]. Many failures of structures reported in Nigeria Dailies have been attributed to lack of comprehensive preconstruction subsoil investigation. Many engineers take this aspect of preconstruction work lightly, some would have come up with the foundation design without any recourse to subsoil information of the proposed site in question. Information such as allowable bearing pressure, shear strength, and soil profile of such site are not usually determined (from the onset of the construction) which are useful information that the foundation design for such structures should be based. Many site engineers, for reasons known to them usually incorporate some assumptions in their structural design, and sometimes fail to include pre-construction investigations in their site work schedules. However studies have shown that preconstruction site investigation is therefore germane in revealing possible subsurface problems/hazards and proffer appropriate solutions before the erection of buildings [6-10].

In view of the importance of engineering investigation or subsoil exploration, the government of Ondo State has mandated the preconstruction information/report for any intended structure in the State through the offices of the urban and regional planning and ministry of works and housing. This important pronouncement has cautioned and prevented people from building structures without permission from the authority. Subsequently, a clients, after collection of approval from the government called the researchers for engineering site investigation of a proposed filling station in Ore, south-western Nigeria (Fig. 1), with principal aim of establishing the soil profile underlain the site, the bearing capacity, and shear strength parameters, for assessment of the competence of soil on the site to host the proposed structure drawn by the architect. Also information on the soil corrosivity for the underground storage tanks was also required [11-12].

The paper is organized as follows, Section I contains the introduction of the study with full discussion on the importance of the study, study background, aim and objectives of the study. Section II contain the related work especially literatures/published articles in relation to engineering site investigation and foundation design.
Section III contain the description of the study area and its environmental. Section IV contain the methodology of study, with analytical description of the methods adopted, data acquisition and processing. Section V describes results and discussion. Section VI contain the conclusion and recommendation of future work.

II. RELATED WORK

Geophysical methods and geotechnical tests are essential for engineering characterization of the subsoil. In recent times the use of geophysical and geotechnical methods to engineering construction has become an established valid fact, especially when combined geotechnical method [13-16]. Geotechnical technique using cone penetrometer test (CPT) measures values of cone resistance and sleeve friction resistance. It is cost efficiency, expeditious, chastity, authenticity and the capacity to give perpetual clues on the soil deposits with depth. The Penetrometer resistance parameters were used for lithological classification and estimation of the soil strength [17]. The electrical resistivity method on the other hand is commonly used for the delineation of horizontal and vertical discontinuities in the electrical properties of the subsurface and also for the detection of three-dimensional bodies of anomalous electrical conductivity [18-21]. The corrosivity of soils is inversely related to the soil resistivity, with low resistivity indicating high probability of corrosion [12].

III. STUDY AREA

The study area is located in Ore and within Odigbo Local Government of Ondo State. The area can be accessed through Akure – Ore Lagos – Benin highways and is located within latitude 715000 – 755000mN and longitude 664750 – 725000mE (Fig. 2). Major part of the study area is devoted to agricultural and commercial activities. The study area tropical rainforest climate. The average temperature is 25°C. Relative humidity of the area differs within 70% to 80%. The average annual rainfall of the area is about 1500 mm and 2500mm [22]. The topographic variation of the site is within 155 – 158 m with respect to sea level. The geology plays a significant role for assessing groundwater potential and quality of the region. The northern area is underlain by migmatite-gneiss complex, comprising older granites, charnockites, quartzite and minor intrusive lithologies [23-25].

The local geology consists of charnockite, fine grained biotite granite and gneiss in the north. Field observation shows that biotite granites in the area occur as large igneous bodies, and largely coarse grained. However the southern parts of the study area is underlain by Coastal plain sands of Benin formation; Ewekoro & Akinbo and Aboekuta formations (Fig. 2). The Aboekuta Formation in surface outcrops comprises mainly sand with sandstone, siltstone, silt, clay, mudstone and shale interbeds [26]. The sands are coarse grained, clayey, micaceous and poorly sorted [27], and indicative of short distances of transportation or short duration of weathering and possible derivation from the granitic rocks located to the north. The area is well drained by rivers and streams that flow in the same direction as the rock strike. These streams take their source from relatively high elevation about 200 m above the mean sea level and flow downhill along the strike into valleys.

IV. METHODOLOGY

After a thorough reconnaissance and desk study, electrical resistivity geophysical method was utilized, employing schlumberger array because it requires less man power and less sensitive to the effects of near surface lateral inhomogeneities than the werner arrangement [28-29]. The current electrodes spread (AB/2) or spacing range from 1 m to 65 m. The AB/2 was measured in both directions from 1.0 m to 65 m using the base station as the midpoint [30-31]. Similarly the potential electrodes spread (MN/2) was also measured on both sides with values varying from 0.25m to 5m. The Ohmega resistivity meter was used to measure the electrical resistance.

The apparent resistivity was calculated using the product of geometric factor and measured resistance. This was subsequently applied on all the point data obtained for each VES station to give the set of apparent resistivity values. Three (03) depth soundings were conducted along two established traverses in a southwest – northeast direction (Fig. 3). The field curves were manually interpreted [32], using master curves [33] and auxiliary point charts [28]. Geoelectric parameters such as resistivity, thickness, and depth obtained from manual interpretation were used as a starting model for computer-assisted interpretation [34-35] through an iterative process.

The cone penetration test used for this study was the Dutch cone penetrometer with base area of 1000mm² and apex angle of 60°. It also consists of a steel frame carrying driving head which houses a hydraulic pressure capsule mounted on a movable frame. The driving head is raised or lowered by motor driven hydraulic ram. Dutch cone penetration test (DCPT) involve pressing a hardened steel cone continuously into the ground using a system of outer sounding tubes, a signal cable connecting the cone and a mounted data logger to measure the resistance to penetration. Tests are normally carried out in accordance with the principle outlined in British Standard BS 5930 [36]. A 2.5 tons Dutch Cone Penetrometer (DCP) was utilized to determine the penetration resistance necessary for the determination of bearing capacity of the soils from layer to layer. This strength was measured at every 0.25 m depth interval. The measurement was terminated at refusal, which is when the maximum capacity (150 Kg/cm²) of the machine was attained.
Figure 1. The Plot of land proposed for the filling station showing deposits of granite and sand materials

Figure 2. Geological map of Southern part of Ondo State
A total number of five points were occupied in the site (Fig. 4). Successive cone resistance readings were plotted against depth to form a resistance profile which indicates the strata sequence penetrated. Disturbed/undisturbed 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 [36]. The moisture content (BS 1377:4), liquid and plastic limits (BS 1377:2), triaxial test, and linear shrinkage tests were carried out on eleven soil samples collected at different depths.

The allowable bearing pressure of the soil layers on each location was calculated using [37] and [38] equations for direct estimation of ultimate bearing capacity \( q_{ult} \) from cone resistance for square and strip footings, as follows:

\[
q_{ult} = q_c \left( \frac{B}{D_f} \right) \left( 1 + \frac{D_f}{B} \right) \tag{1}
\]

\( q_c = \) cone resistance value
\( D_f = \) Depth of footing
\( B = \) Width of foundation

Factor of safety at least 3 is recommended by [37] to obtain the allowable bearing pressure.

For cohesionless soils:
- **Strip**:
  \[
  q_{ult} = 28 - 0.0052 (300 - q_c)^{1.5} \text{ kg/cm}^2 \tag{2}
  \]
- **Square**:
  \[
  q_{ult} = 48 - 0.0052 (300 - q_c)^{1.5} \text{ kg/cm}^2 \tag{3}
  \]

For clay:
- **Strip**:
  \[
  q_{ult} = 2 + 0.28q_c \text{ kg/cm}^2 \tag{4}
  \]
- **Square**:
  \[
  q_{ult} = 5 + 0.34q_c \text{ kg/cm}^2 \tag{5}
  \]

Using [37] equation, the allowable and ultimate bearing capacity was calculated using this equation:

\[
q_a = 2.7q_c \left( \frac{KN}{m^2} \right) \tag{6}
\]

\[
q_a = \frac{q_c}{40} \text{ (kg/cm}^2) \tag{7}
\]

Subsequently a factor of safety of 3 was applied on the allowable bearing capacity to get the ultimate bearing capacity [17, 37].

Total consolidation settlement \( s \) was computed for foundation breadth \( B \) between 0.5 – 3.0 m, subjected to a allowable bearing capacity of 100kN/m². The induced vertical stress (\( \Delta \sigma \)) at the centre of the consolidating layer has been used in computing \( s \). The consolidation settlement [39] has been computed from the expression below.

\[
s = m_v H \Delta \sigma \tag{8}
\]

\( m_v = \) 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 stiff clay/silt/sand mixed soil and 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)

### V. RESULTS AND DISCUSSION

The VES curves identified in the site are KH and QH, with four geoelectric layer combinations. The QH curve type predominates, constituting 66.67%, while KH curve type constitutes 33.33%.

![Figure 3. Schematic Layout map for the Data acquisition](image-url)
The VES resistivity values were interpreted using competent rating of [40] as shown in Table 1, while the soil corrosivity for storage pipes installation is evaluated using [12] and [41] as shown in Table 2.

Table 1. Rating Of Subsoil Competence Using Resistivity Values [40]

<table>
<thead>
<tr>
<th>Apparent resistivity range (ohm-m)</th>
<th>Lithology</th>
<th>Competence rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 100</td>
<td>Clay</td>
<td>Incompetent</td>
</tr>
<tr>
<td>100 – 350</td>
<td>Sandy clay</td>
<td>Moderately competent</td>
</tr>
<tr>
<td>350 – 750</td>
<td>Clayey sand</td>
<td>Competent</td>
</tr>
<tr>
<td>&gt; 750</td>
<td>Sand/ Laterite/ Crystalline Rock</td>
<td>Highly competent</td>
</tr>
</tbody>
</table>

Table 2. Soil Corrosivity Rating ([12, 41])

<table>
<thead>
<tr>
<th>Soil resistivity range (ohm-m)</th>
<th>Corrosivity Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 100</td>
<td>Essentially/Practically non-corrosive</td>
</tr>
<tr>
<td>100 – 350</td>
<td>Mildly/Slightly corrosive</td>
</tr>
<tr>
<td>350 – 750</td>
<td>Moderately corrosive</td>
</tr>
<tr>
<td>&gt; 750</td>
<td>Very Strongly corrosive</td>
</tr>
</tbody>
</table>

The geoelectric section identified maximum of four geoelectric/geologic subsurface layers comprising the topsoil, weathered layer, partly weathered/fractured basement/fresh basement (Fig. 5). The topsoil varies in composition from clay, clayey sand and laterite with resistivity values ranging from 30 to 1031 ohm-m and thickness varies from 3.1 – 15.5 m, with an average value resistivity value of 351 ohm-m whose geoelectric characteristic is typical of clayey sand. The weathered layer resistivities are generally within the range of 94 and 615 ohm-m, typical of clay, sandy clay, and clayey sand. Resistivity in the range of 40 – 200 ohm-m is the most dominant, signifying a clayey weathered layer. The thickness is moderately thick with values varying from 13.8 m to 20.3 m. The partly weathered/fractured basement/fresh basement has layer resistivity values vary from 705 – 1910 ohm-m. The depth to (overburden thickness) to this layer is in between 18 – 35.8 m. Information gathered from existing hand dug well and boreholes were drilled to use during construction, shows that the groundwater level varies from 3.5 to 7.2 m. Therefore caution must be taken to incorporate this water level values during design consideration. Therefore judging from the later resistivities, the topsoil (avg. 351ohm-m) is moderately competent to harbor and sustain the structural load. And at the same time, it’s moderately corrosive with moderate thickness. The thickness of the weathered layer could favour burial of storage fuel tanks since it’s characterized with thickness greater than 10 m in all places within the site. The average resistivity of this layer is 262 ohm-m which can be categorized as mildly/slightly corrosive soil. Therefore its geoelectric resistivity may require re-worked or stabilized prior construction to improve the geotechnical properties and its corrosivity.
The results of cone resistance and sleeve friction against depth, show increase in cone resistance and sleeve resistance/friction with depth (Tables 3–7). The cone resistance and sleeve resistance vary from 25 – 78 kg/cm² and 35 – 130 kg/cm² at CPT 1, 35 - 100 kg/cm² and 72 - 140 kg/cm² at CPT 2, 10 - 82 kg/cm² and 25 - 110 kg/cm² at CPT 3, 25 - 80 kg/cm² and 40 - 135 kg/cm² at CPT 4; 15 - 60 kg/cm² and 30 - 60 kg/cm² at CPT 5 respectively. The friction ratio ranges from 1.17 – 2.04 (CPT 1), 1.60 – 2.0 (CPT 2), 1.6 – 2.0 (CPT 3), 1.34 – 2.50 (CPT 4), and 1.13 – 2.17 (CPT 5). The soil classification chart [45] shows three dominant zones of 5, 6 and 7 corresponding to clayey silt to silty clay, sandy silt to clayey silt and silty sand to sandy silt respectively (Fig. 8). The plots of cone resistance and sleeve resistance against depth (Fig. 9) showed four geological succession of sandy silt to clayey silt (0 - 0.2 m), silty sand to sandy silt (0.2 – 0.8 m) at CPT 1, two geologic sequence in CPT 2, namely sandy silt to clayey silt (0 – 0.5 m), and silty sand to sandy silt (0.5 – 0.8 m), two geologic units in CPT 3, silty clay to clay (0 – 0.25 m) and sandy silt to clayey silt (0.25 – 1.0 m). CPT is characterized by sandy silt to clayey silt (0 – 0.2 m), silty sand to sandy silt (0.2 – 1.0 m), while five geological sequence observed under CPT 5, namely, clayey silt to silty clay (0 – 0.3 m), sandy silt to clayey silt (0.3 – 0.7 m), silty sand to sandy silt (0.7 – 1.0 m), clayey silt to silty clay (1.0 – 1.3 m), and sandy silt to clayey silt (1.3 – 1.6 m). Consequently, at least depth of 1.0 m would be appropriate as founding depth for design and construction of shallow foundation. This depth is generally of silty sandy to sandy silt.

The ultimate and allowable bearing capacity estimated from the cone resistance using [37] equation are presented in Table 9. These bearing capacities could be used in determining the type of foundation appropriate for structures. The allowable bearing of the soil (within upper 1.2 m) varies between 25 (CPT 3) to 245 kPa (CPT 2) and ultimate bearing capacity of 75 to 735 kPa. Consequently an average allowable bearing capacity of 150 kPa (ultimate bearing capacity of 450 kPa) is recommended and would be appropriate for design of shallow foundation in the area, at a depth not less than 1.0 m. with high shear strength parameters [44]. Also these range of values make the soil samples competent to support civil engineering foundation structures, especially shallow foundation at least 1 m depth.
Table 3. Summary of the CPT, Atterberg Limits, and Grain Size Analysis at Different Depth Levels at Point 1

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Cone Resistance</th>
<th>Sleeve Resistance</th>
<th>Friction Ratio</th>
<th>L.L. (%)</th>
<th>P.I. (%)</th>
<th>P.I. (%)</th>
<th>S.I. (%)</th>
<th>M.C. (%)</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
<th>S.G</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
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<td>2.04</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>1.57</td>
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<td>25.1</td>
<td>19.05</td>
<td>8.6</td>
<td>22.3</td>
<td>-</td>
<td>44.0</td>
<td>18.8</td>
<td>37.3</td>
<td>2.68</td>
</tr>
<tr>
<td>0.75</td>
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<td>65</td>
<td>1.30</td>
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<td></td>
<td></td>
<td></td>
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<td>-</td>
<td>44.0</td>
<td>18.8</td>
<td>37.3</td>
<td>2.68</td>
</tr>
<tr>
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<td>35</td>
<td>1.17</td>
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<td>22.4</td>
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Table 4. Summary of the CPT, Atterberg Limits, and Grain Size Analysis at Different Depth Levels at Point 2

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Cone Resistance</th>
<th>Sleeve Resistance</th>
<th>Friction Ratio</th>
<th>L.L. (%)</th>
<th>P.I. (%)</th>
<th>P.I. (%)</th>
<th>S.I. (%)</th>
<th>M.C. (%)</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
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<tr>
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<td></td>
<td></td>
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<td>14.3</td>
<td>45.6</td>
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<td>14.3</td>
<td>45.6</td>
<td>2.69</td>
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Table 5. Summary of the CPT, Atterberg Limits, and Grain Size Analysis at Different Depth Levels at Point 3

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Cone Resistance</th>
<th>Sleeve Resistance</th>
<th>Friction Ratio</th>
<th>L.L. (%)</th>
<th>P.I. (%)</th>
<th>P.I. (%)</th>
<th>S.I. (%)</th>
<th>M.C. (%)</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
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<td></td>
<td></td>
<td></td>
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<td>36.8</td>
<td>14.7</td>
<td>48.5</td>
<td>2.68</td>
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<td>2.68</td>
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<td>36.8</td>
<td>14.7</td>
<td>48.5</td>
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</table>

Table 6. Summary of the CPT, Atterberg Limits, and Grain Size Analysis at Different Depth Levels at Point 4

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Cone Resistance</th>
<th>Sleeve Resistance</th>
<th>Friction Ratio</th>
<th>L.L. (%)</th>
<th>P.I. (%)</th>
<th>P.I. (%)</th>
<th>S.I. (%)</th>
<th>M.C. (%)</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
<th>S.G</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>25</td>
<td>40</td>
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<td></td>
<td></td>
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<td>51.7</td>
<td>10.1</td>
<td>38.2</td>
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</tr>
<tr>
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<td>68</td>
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<td>10.1</td>
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<td>13.3</td>
<td>9.5</td>
<td>19.5</td>
<td>-</td>
<td>53.8</td>
<td>9.8</td>
<td>36.4</td>
<td>2.66</td>
</tr>
</tbody>
</table>

Table 7. Summary of the CPT, Atterberg Limits, and Grain Size Analysis at Different Depth Levels at Point 5

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Cone Resistance</th>
<th>Sleeve Resistance</th>
<th>Friction Ratio</th>
<th>L.L. (%)</th>
<th>P.I. (%)</th>
<th>P.I. (%)</th>
<th>S.I. (%)</th>
<th>M.C. (%)</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
<th>S.G</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>20</td>
<td>35</td>
<td>1.75</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td>55.3</td>
<td>10.5</td>
<td>34.2</td>
<td>2.67</td>
</tr>
<tr>
<td>0.50</td>
<td>35</td>
<td>50</td>
<td>1.43</td>
<td>50.2</td>
<td>20.6</td>
<td>29.6</td>
<td>8.9</td>
<td>13.8</td>
<td>-</td>
<td>55.3</td>
<td>10.5</td>
<td>34.2</td>
<td>2.67</td>
</tr>
<tr>
<td>0.75</td>
<td>45</td>
<td>70</td>
<td>1.56</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td>55.3</td>
<td>10.5</td>
<td>34.2</td>
<td>2.67</td>
</tr>
<tr>
<td>1.0</td>
<td>40</td>
<td>45</td>
<td>1.13</td>
<td>44.8</td>
<td>22.9</td>
<td>21.9</td>
<td>9.5</td>
<td>15.2</td>
<td>-</td>
<td>54.7</td>
<td>12.9</td>
<td>32.4</td>
<td>2.65</td>
</tr>
<tr>
<td>1.25</td>
<td>15</td>
<td>30</td>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td>54.7</td>
<td>12.9</td>
<td>32.4</td>
<td>2.65</td>
</tr>
<tr>
<td>1.50</td>
<td>60</td>
<td>130</td>
<td>2.17</td>
<td>42.5</td>
<td>29.1</td>
<td>13.4</td>
<td>9.8</td>
<td>17.8</td>
<td>-</td>
<td>60.0</td>
<td>9.2</td>
<td>30.8</td>
<td>2.65</td>
</tr>
</tbody>
</table>

Settlement and bearing capacity are the major factors that govern foundation design. The accepted standard of design is that the total settlement of a footing should be restricted to about 25 mm [46] 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 0.6 – 3.0 m at three depth levels of 1m, 2m and 3m produces values between 9.46 – 44.26 mm at 1 m depth; 11.82 – 45.08 mm at 2 m, and 15.99 – 47.98 mm at 3 m depth (Table 10). But foundation width above 0.6 m produces settlement less than 25 mm (Table 5) recommended by Bell [46] as it ranges between 9.03 – 21.33 mm. The settlement analysis for foundation width of 0.6 – 3.0 m at three depth levels of 1m, 2m and 3m produces values between 9.46 – 44.26 mm at 1 m depth; 11.82 – 45.08 mm at 2 m, and 15.99 – 47.98 mm at 3 m depth (Table 10).
But foundation width above 0.6 m produces settlement less than 25 mm (Table 5) recommended by Bell [46] as it ranges between 9.03 – 21.33 mm. Although according to [17, 37], total settlement limits of 60 mm (clay) and 50 mm (granular soil) are still tolerable. Therefore foundation width not less than 0.6 m for depth not less than 1 m is still okay and appropriate. The calculation of bearing capacities for strip and square foundation is shown in Table V within 0.6 – 1.4 m depth. For strip foundation, the appropriate (recommended) ultimate bearing and allowable bearing capacity vary from 1568 – 2419 kPa and 523 – 806 kPa, while square footing varies in between 2156 – 3189 kPa and 719 - 1063 kPa respectively (Table XI).

VI. CONCLUSION AND FUTURE WORK

Therefore judging from the layer resistivity, the topsoil (avg. 351 ohm-m) is moderately competent and corrosive to harbor and sustain the structural load with appreciable thickness of 3.1 – 15.5 m. The thickness of the weathered layer favours burial of storage fuel tanks since it’s characterized with thickness greater than 10 m in all places within the site. The average resistivity of this layer is 262 ohm-m which can be categorized as midly/slightly corrosive soil. Therefore it may require re-worked or stabilized prior construction to improve the geotechnical properties and its corrosivity. The plasticity index of the soil samples are generally greater than 20% recommended signifying poor soil material. This corroborates the geoelectric characteristics of the clayey-rich topsoil. Plasticity char indicate that the soils have from low (CL) - intermediate (CI). All the samples plot above the A-line indicating an inorganic clay. An average allowable bearing capacity of 150 kPa (ultimate bearing capacity of 450 kPa) is recommended and would be appropriate for design of shallow foundation in the area, at a depth not less than 1.0 m.

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth</th>
<th>(σ) Deviator stress at Different Cell Pressures (KPa)</th>
<th>Cohesion (c) Kpa</th>
<th>(δ) Angle of Friction</th>
<th>Undrained Compressive Strength (KPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>50</td>
<td>100</td>
<td>150</td>
<td>@ Min. Cell Pressure</td>
</tr>
<tr>
<td>CPT 1</td>
<td>0.5</td>
<td>395</td>
<td>470</td>
<td>545</td>
<td>102.6</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>395</td>
<td>468</td>
<td>542</td>
<td>101.9</td>
</tr>
<tr>
<td>CPT 5</td>
<td>0.5</td>
<td>440</td>
<td>532</td>
<td>625</td>
<td>56.0</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>440</td>
<td>528</td>
<td>616</td>
<td>62.2</td>
</tr>
</tbody>
</table>

Figure 7. Mohr Circle of Unconfined Compressive strength for (a) Sample at CPT 5 taken at 0.5 m depth (b) Sample at CPT 1 taken at 1.0 m depth

Figure 8. Robertson Chart for the soil classification in the site using cone resistance and friction ratio values
Figure 9. Plots of Cone resistance and sleeve resistance against depth at: (a) CPT 1 (b) CPT 2 (c) CPT 3 (d) CPT 4 (e) CPT 5
The settlement analysis for foundation width of 0.6 – 3.0 m at three depth levels of 1m, 2m and 3m produces values between 9.46 – 44.26 mm at 1 m depth; 11.82 – 45.08 mm at 2 m, and 15.99 – 47.98 mm at 3 m depth. But foundation width above 0.6 m is recommended so as to have settlement value less than 25 mm. The calculation of bearing capacities for strip and square foundation within 0.6 – 1.4 m depth give an ultimate bearing and allowable bearing capacity varying from 1568 – 2419 kPa and 523 - 806 kPa while square footing varies in between 2156 – 3189 kPa and 719 – 1063 kPa respectively. It is therefore recommended that more sophisticated geophysical tools/methods and modern boring techniques be employed to proof the geology of the site further.

ACKNOWLEDGMENT

Special thanks to Mr. Samuel Alabi of the Geology Department of Federal University of Technology Akure, Ondo State, Nigeria.

REFERENCES

World Academics Journal of Engineering Sciences

Vol.7, Issue 2, June 2020


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