Evaluation of near-surface conditions for engineering site characterization using geophysical and geotechnical methods in Lagos, Southwestern Nigeria

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ABSTRACT
The geological condition of the site in Lagos was investigated using integrated methods. This was necessary following evidence of structural defects prevalent in the area. Against this backdrop, measurements were carried out using in-situ geotechnical and geophysical techniques. The techniques were the Standard Penetration Test, Cone Penetration Test, Electrical Resistivity and seismic refraction surface waves. The various data were collected along eight (8) traverses established strategically in the study area in order to obtain the most desirable results. The results obtained generally show that a low resistive and highly compressive soft soils of organic peats/clays are prevalent in the study area. These geomaterials are characterised with low SPT-N values, low penetrative resistance, low bearing capacities and low electrical resistivity values. The shear (weak) zones have also been identified particularly in the inverted resistivity models. These geomaterials are unfriendly, inimical and severe enough to cause instability in ground conditions. The peat/clay and the weak zones could possibly be responsible for the various degrees of structural defects observed in the study area and should be avoided as foundation placements.

1. Introduction
Building collapses are becoming recurring events in most major cities around the globe, posing great threat to urban development and sustainability. Issues arising from climate changes have suggested the possibility of higher disaster risk as more structural failures could take place in areas prone to the meteorological hazards (Boateng and Wright 2018). The disturbing issues of structural collapse have necessitated the need for urgent attention to be taken particularly in the area of subsurface investigations towards unravelling and addressing the underlying causes.

The ability of near-surface geophysical methods in complementing geotechnical studies in foundation engineering studies cannot be over-emphasised (Chu et al. 2018). Poor planning of construction and the inability to carry out pre-development geophysical or geotechnical surveys have led to a lot of structural problems. Many cases have resulted in poor building conditions noticed as cracks on walls (differential settlement), sinking of buildings (either dipping or uniform settlement) and total collapse of the building.

In engineering or foundation studies, geophysics plays essential roles in the investigation of the subsurface materials and structural identification. Geophysical methods such as the electrical resistivity and seismic refraction among others offer high temporal and spatial capabilities, fast, economically viable, less tedious and cost-effective ways of evaluating the competence of soil types for the design of building foundations because larger area can be covered in a relatively short time (Jung-Ho et al. 2007; Soupios et al., 2007; Suhda et al., 2009; Chalikakis et al. 2011; Ayolabi et al. 2012; Ishola et al. 2014; Kowalczyk et al. 2015; Adeoti et al. 2018; Chu et al. 2018; Prudhomme et al. 2019).

Several geophysical methods have been used for site characterisation either individually or in an integrated manner with a view to providing detailed information about the subsurface being investigated. Among these methods, only the electrical resistivity techniques (Johansson and Dahlin 1996; Soupios et al. 2006; Jones et al. 2014; Prudhomme et al. 2019) and seismic methods (Azimam et al. 2016; Goren and Gelishi 2017) are the main focus in this study.

Besides the use of geophysical methods, indirect methods such as aerial photography (Pope et al. 1996), topographic map and soil surveys have also been utilised (Soupios et al., 2007). The use of direct methods such as boring tests from which representative soil samples are collected and analysed have also been emphasised. The Standard Penetration Test (SPT) and Cone Penetration Test (CPT) are examples of direct geotechnical techniques which are used to
determine sub-soils or rocks resistance to penetration (Sully and Campanella 1991; Lunne et al. 1997; Sudha et al. 2009; De-Silva et al., 2014).

In the literature, concerted efforts have been made to characterise subsurface lithological units for detailed knowledge of the local geology through integrated approach (Das 1994; Braga et al. 1999; Seshunaryana 2002; Giao et al. 2003; Cosenza et al. 2006; Gay et al. 2006 cited in Sudha et al. 2009; Oyedele and Olorode 2010; Trupt et al. 2012; Mahajan et al. 2015; Gupta et al. 2019; Hussain et al., 2020). The application of both geotechnical and geophysical techniques is particularly useful for soil characterisation and site assessment. In this regard, the present study was embarked using both techniques with a view to achieving the following objectives:

(i) delineate the geological units and identify weak zones in the electrical and seismic models
(ii) estimate the physical parameters of the soil strata
(iii) establish empirical relationships for the measured physical parameters

2. Location and geology setting

The study area lies between Latitudes 6° 33’ 5.8” N to 6° 33’ 48” N and Longitudes 3° 22’ 34.7”E to 3° 23’ 5.2” E and belongs to the Dahomey Basin, as shown in Figure 1. The surface geology of the basin shows that is made up of the Benin Formation (Miocene to Recent) and the Recent Littoral alluvial deposits (Adegoke 1969; Kogbe 1975; Olabode and Adekoya 2008), as depicted in Figure 2. The Benin Formation consists of thick bodies of ferruginous and white sands (Jones and Hockey 1964). The coastal plain sands are transported along the coast and reworked alluvial sands originally deposited by flowing rivers (Durotuye 1975). The recent littoral deposits are the water bearing aquifers which consist of sands, gravel or a mixture of the sands and gravel. Additionally, the study area belongs to the coastal creeks and lagoons that are entirely surrounded by the Lagos lagoon systems developed by barrier breaches associated with sand deposits (Kaki et al. 2012). Among the prominent Formations that made up the basin is the Ise Formation which consists of basal conglomerates overlain by coarse to medium-grained loose sands, sandstones and grits containing kaolinitic clays (Omatsubo and Adegoke 1981); the Albian Formation predominantly sandstone with shaley and dolomitic layers. Others are the Abeokuta Formation consists of sandstones and conglomerates with thin layers of silty sand or shales and the Awgu Formation that is younger than the Abeokuta Formation. It consists of sequence of anaerobic marine dark grey calcareous shales with calcareous siltstone and more rarely fine-grained sandstones.

3. Methodology

In this study, measurements were carried out along eight traverses established for both the geophysical and geotechnical techniques. The 2D electrical resistivity surveys were carried out along traverses 1–4, while the seismic using the multichannel analysis of surface waves technique was acquired along traverses 5–8. To allow for good correlation and constraint the interpretation of the geophysical measurements, two boreholes were drilled closer to the geophysical survey lines.

3.1. Geophysical surveys

The two geophysical methods used were the Electrical Resistivity Method (ERM) and Multichannel Analysis of Surface Waves (MASW). For the ERM measurements, two-dimensional (2-D) electrical imaging using the Wenner array and Schlumberger Vertical Electrical Sounding (VES) techniques were carried out with a view to gaining knowledge of the lateral and depth variations in electrical resistivity, respectively, in the study area.

3.1.1. Electrical resistivity measurements

In order to produce the 2D electrical resistivity models for the study area, four metal stake electrodes were used for the apparent resistivity measurements. Two of these electrodes serve as current electrodes (AB) and the other two as potential electrodes (MN). These electrodes were driven into the ground at fixed equal electrode spacing. A direct current (DC) of 20 mA through a 12 V battery was injected into the ground through the AB, and the resulting electric potential was measured between the MN with a fixed minimum inter-electrode spacing of 5 m using a PASI 16GL Earth resistivity metre. The choice of 5 m minimum electrode separation was to ensure a good delineation of the study area. This was followed by electrode separation sequences of 10, 15, 20, 25 and 30 m along all the traverses. The first sequence of measurement was obtained from four electrodes 1, 2, 3 and 4 at corresponding positions A, M, N and B, respectively. Then, the next readings were made by shifting the electrode position to the right while maintaining the current and potential positions leading to the next measurements acquired at 2, 3, 4 and 5 with A, M, N and B, respectively, in a plane. This pattern continued until measurements involving the last four
electrodes were performed. To further have an overview of the variation of resistivity with depths, we performed eighteen (18) Schlumberger VES measurements along the 2D resistivity profiles.

The sets of apparent resistivity data acquired were inverted to obtain true resistivity distribution in the subsurface using DIPRO™ software. This inversion programme automatically created the 2D resistivity models by dividing the subsurface into rectangular blocks and estimated the apparent resistivity using finite difference optimisation technique (Adepelumi et al. 2008).

For the VES surveys, the sets of apparent resistivity data at each station were plotted against the half electrode spacing (AB/2) on a log–log graph. The VES data were interpreted by partial curve matching and computer-assisted 1-D forward modelling. The digitised field (i.e. observations) data of the various curves generated were inverted and interpreted using the inversion WINRESIST software developed to give the model parameters for an n-layered model. From the interpreted inverted VES, four geoelectric sections along the directions (N-W and S-E) of the traverses were produced.

3.1.2. Multichannel analysis of surface waves (MASW) measurements
A 24 channel ABEM Terraloc MK6 seismograph from Geometrics, a 12 V battery for power supply to the seismograph, a set of twenty-four, 4.5 Hz vertical geophones and two 100 m spread cable reels deployed...
in a linear array were used for the MASW measurements. The twenty-four (24) geophones were connected to the two cable reels. The two ends of the cables were in turns plunged into the two ports of the seismograph. Also, a trigger geophone placed close to the metal plate was connected to the seismograph through a trigger cable. Measurements were carried out only along four traverses with receiver array length that spans between 69 and 92 m. A 9.5 kg impulsive sledgehammer that has the ability to probe to an average depth of 35 m struck against a base metal plate was used as seismic energy source. To ensure a good signal to noise ratio, two to four stacks were used for the acquired data. The standard common mid-point (CMP) technique which involved an assembly of traces that have the same mid-point was considered. The other survey parameters used to ensure an optimum coverage and good characterisation of the earth materials of the study area are summarised in Table 1.

### 3.1.3. Data processing of MASW data

The 24-trace shot gathers acquired during MASW measurements were analysed using SeisImager software, a proprietary software package of Geometrics to obtain the 2D shear-wave velocity images of the study area. As a pre-processing step, the survey geometry was edited. The dispersion curve was generated using the common mid-point cross-correlation (CMPCC) technique. To assemble CMPCC gathers, a cross-correction of each pair of shot traces was carried out. Then, the correlated traces with identical common mid-point were collected and stacked in the time domain mode (Hayashi and Suzuki 2004). Next, the dispersion curves were displayed as phase velocity–frequency plots from the CMPCC gather by transformation from time domain gathers to frequency domain. Also, for each CMPCC gather, a one-dimensional (1-D) shear-wave velocity profile was reconstructed from the inversion of the phase velocity dispersion curves using a linearised iterative least-square technique in the WaveEq module (Xia et al. 1999; Socco and Strobia 2004). Then, a 2D contour plot of the shear-wave velocity model was generated by

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**Table 1. Acquisition data recording parameters for MASW survey.**

| Survey Parameter       | Acquisition Parameter |
|------------------------|-----------------------|
| Sampling Time (ms)     | 1                     |
| No of samples          | 512                   |
| No of stacks           | 2–4                   |
| Rec. time (ms)         | 12.8                  |
| Record length          | 1 or 25               |
| Receiver spacing d_r(m)| 1                     |
| Receiver spread length| X_r (m)               |
| Sampling rate (ms)     | 23                    |
| Acquisition interval, dS (m) | 1024          |
| Offset distance, X_1 (m) | 3                   |

*Figure 2. A modified cross section of the Dahomey Basin (Brownfield and Charpentier 2007; Elvsborg and Dalode 1989 culled from; Gbadamosi 2009).*
3.2. Geotechnical assessment

The SPT and CPT measurements were conducted to estimate the geotechnical properties of the materials at some selected points. The choice of these points where the geotechnical tests were carried out was informed by the suspected low and high anomalies zones detected during the preliminary 2D electrical resistivity surveys. Also, the first borehole (BH1) was drilled close to the existing structures with some structures being visibly defective. The second borehole (BH2) was drilled at the site reserved for future development. On this backdrop, the information on the SPT measurements from the two boreholes provided better understanding of the subsurface conditions in the study area. These tests were carried out to estimate the resistance of the soil to penetration and engineering properties of the soils under dynamic or static loading. A percussion machine with a rotary drilling bit was used for the boreholes (BH1 and BH2) drilling to a depth of about 30 m. Also, the groundwater levels were monitored during and after removal of the drilling equipment. The in-situ SPT measurement was carried out on the non-cohesive soils using a split barrel sampling unit (split spoon) hammered into the undisturbed soil at a distance of about 460 mm (18 inches). The addition of hammer strikes in the final two intervals gave the penetration resistance, and hence, the number of blows or counts i.e. the N-values. The N-values of the soils recorded provided information on the compactness of the soil. The classification scheme of Bowles (1984) presented in Table 2 was used to assign lithology units to the different soil types characterised. The CPT was another in-situ measurements carried out to assess the soil’s stratigraphic units because the CPT has ability to penetrate underlying layers and continuously measure the cone penetration resistance ($q_c$) of the soils (Mayne et al. 2002; Sudha et al. 2009). The CPT measurements were conducted at four points in the study area using a 2.5 Tonnes capacity machine. The procedures followed in measuring the cone tip resistance to penetration into soils are the standards available in the literature (Adeoti et al. 2018). The classification scheme of Robert (1996) presented in Table 3 guided in the assignment of lithology units to the soil types.

3.3. Estimation of soil’s dynamic parameters

It is known that shear-wave velocities ($V_s$) are associated with porosity, fracture, and the mechanical properties of the soils (Stokoe et al. 1994). The bearing capacity of any foundation is used to describe the load-carrying capacity of the foundation soil or rock in terms of average pressure (i.e. critical load per unit area) that enables it to bear and transmit loads to the structure at either the ground surface or at a certain depth from the ground surface. The ultimate bearing capacity of a load (e.g. foundation) is estimated on the assumption that the soil is relatively uniform within the zones of shear deformation beneath the foundation (Meyerhof and Hanna 1978). The evaluation of bearing capacity is just a rough approximation to give an idea of the low resistance of soil types. For the determination of allowable bearing capacity, the use of geophysical methods, especially seismic surface wave technique in which there is no disturbance of natural site conditions, could give relatively more reliable results than those of the geotechnical methods, which are based mainly on borehole log data and laboratory testing of the undisturbed soil samples.

In order to know the strength–deformation characteristics of the subsurface soils at the investigated site, we estimated the ultimate bearing capacity ($Q_{ult}$), allowable bearing capacity ($Q_a$), total unit weight ($\gamma$) and the coefficient of subgrade reaction of the soils ($k_s$). The appropriate equations used were in Eqs. 1–4.

$$Q_{ult}(kPa) = 0.1yV_s$$  (Tezcan and Ozdemir 2011)  \hspace{1cm} (1)

$$Q_a(kPa) = \frac{0.1yV_s}{n}$$  \hspace{1cm} (2)

$$\gamma = 4.3V_s^{0.25}$$  \hspace{1cm} (3)

The unit weight ($\gamma$) provided in Table 2 was used as the average weight for the soil layer above the foundation, while the in-situ measurements of the shear-wave velocity ($V_s$) for the soil layer were used in the estimation of the ultimate and allowable bearing capacities.

In geotechnical studies, an interaction between the soils and foundation type is very significant in understanding the behaviour of soils when subject to external loads. In this regard, the determination of the

| N – Value | State of Soil | Unit Weight, $\gamma$ (pcf) | Unit Weight, $\gamma$ (kN/m$^2$) |
|-----------|---------------|-----------------------------|-----------------------------|
| 0–10      | Loose         | 75–100                      | 12–16                       |
| 11–30     | Medium Dense  | 90–115                      | 14–18                       |
| 31–50     | Dense         | 100–125                     | 16–20                       |
| >50       | Very Dense    | 115–145                     | 18–23                       |

Table 2. Correlations between SPT-N Values and cohesionless soil properties (Bowles 1984).

| Cone Resistance (kgf/cm$^2$) | Relative Density |
|-----------------------------|------------------|
| 0–40                        | Very loose to loose |
| 40–120                      | Medium dense     |
| 120–200                     | Dense            |
| Above 200                   | Very dense       |

Table 3. Classification of Cone Penetration Test for Granular Soils (Robert 1996).
coefficient of subgrade reaction of the soil \( (k_s) \) is essential. It is the ratio of the contact pressure at any given point on the surface to the settlement resulting from the loading at the point where it was applied. In a more explicit way, it can be estimated as the vertical pressure \( (Q_v) \) necessary to produce a unit vertical displacement/settlement \( (\gamma) \) on the soil (Terzaghi 1955; Moayed and Bolandi 2012).

\[
k_s = \frac{Q_v}{\gamma}
\]

(4)

According to Terzaghi and Peck (1976), the total vertical displacement \( (\gamma) \) for foundations is 0.025 m. On substitution of this value for the displacement in Eq. 4, the coefficient of subgrade reaction is given in Eq. 5 (Emujakporue 2011; Kaptan 2012):

\[
k_s = 40Q_v
\]

(5)

where:

\( Q_v \) = Ultimate bearing capacity
\( V_s \) = Shear-wave velocity
\( \gamma \) = Unit weight a soil in unit of kN/m\(^3\)
\( n \) = Factor of Safety (FS)
\( k_s \) = Coefficient of subgrade reaction of soil
\( Q_s \) = Allowable bearing capacity

The stability of a foundation on the soils is very vital for a good foundation design. As a result, the stability can be evaluated using the factor of safety (FS) (Li et al. 2017). The FS is evaluated on the reliability of the subsoil information. This is because it is the limit of design (minimum) required to guarantee satisfaction in structural design and construction. Thus, FS is the ratio of available shear strength to strength required to maintain stability (i.e. the load-carrying capacity of the soil).

Depending on soils and rock types, the FS denoted by “n” ranges between 1.4 < n < 4.0 with variations in Vs from 750 ≤ \( V_s \) ≤ 4000m/s (Tezcan et al. 2009).

For soils, \( n = 4.0 \), and 750 ≥ \( V_s \)m/s; for hard rocks, \( n = 1.4 \) and \( V_s \) ≥ 4000m/s, while for soft weak rocks, \( n = 4.6–0.0008 \) Vs with 750 ≤ \( V_s \) ≤ 4000m/s (Tezcan et al. 2009).

In site characterisation, it is important to determine the shear-wave velocity directly using empirical equations especially in situations where it is difficult to carry out test at all locations. To this end, several empirical relationships have been established between shear-wave velocity \( (V_s) \) and the soil’s resistance to penetration \( (N-values) \) for different soils. For this present study, statistical correlation between the shear-wave velocity and uncorrected SPT-N values was established for only non-cohesive soil using simple regression analysis. In order to examine the predictive capability, the regression model obtained was compared with the previously established models in Table 4.

4. Results and discussion

4.1. Results of geophysical investigations

5. 4.1.1 2-D Electrical resistivity models results

The 2-D resistivity data after inversion are presented as inverted resistivity models (IRM). The inverted models show the distribution of resistivity beneath the area investigated together with the number of iterations and the model misfit values (i.e. RMS errors). The inverted resistivity models produced after the fourth and fifth iterations are shown in Figure. 3(a-d). The RMS error range between 0.047 and 0.069. Generally, all the IRMs show that three zones are clearly mapped with resistivity distribution in the range of 5–1298 Ωm. Inferred lithologic units are assigned to the demarcated zones using the range of resistivity values available in the literature (Loke 1999) and the borehole log data. The low resistivity zone (LRZ) in dark blue colour has resistivity values between 5–16 Ωm. The intermediate resistivity zone (IRZ) in greenish-yellow colour with resistivity values is in the range of 18–101 Ωm. Also, the relatively high resistivity zone (HRZ) in reddish purple is noticed to dominate the deeper portion of the section characterised with resistivity values between 157 and 1298

| S/N | Author(s) | Sand | Clay | All soils |
|-----|-----------|------|------|----------|
| 1   | Shibata (1970) | \( V_s = 31.7N^{0.54} \) | – | – |
| 2   | Ohta et al. (1972) | \( V_s = 87.2N^{0.36} \) | – | – |
| 3   | Imai and Yoshimura (1975) | \( V_s = 31.7N^{0.54} \) | – | – |
| 4   | Imai (1977) | \( V_s = 80.6N^{0.33} \) | \( V_s = 80.2N^{0.59} \) | \( V_s = 91.0N^{0.37} \) |
| 5   | Ohta and Goto (1978) | \( V_s = 85.3N^{0.15} \) | – | – |
| 6   | Seed and Idris (1981) | – | \( V_s = 10.0N^{0.29} \) | – | – |
| 7   | Sykora and Stokoe (1983) | \( V_s = 100.5N^{0.29} \) | – | – |
| 8   | Okamoto et al. (1989) | \( V_s = 125.0N^{0.30} \) | – | – |
| 9   | Pitilakis et al. (1992) | \( V_s = 162.0N^{0.17} \) | \( V_s = 165.7N^{0.19} \) | – | – |
| 10  | Athanasiopoulos (1995) | – | \( V_s = 76.5N^{0.45} \) | \( V_s = 107.6N^{0.36} \) | – | – |
| 11  | Raptakis et al. (1995) | \( V_s = 123.4N^{0.29} \) | \( V_s = 184.2N^{0.17} \) | \( V_s = 19.0N^{0.6} \) |
| 12  | Kanai (1966) | – | – | \( V_s = 27.0N^{0.27} \) |
| 13  | Jafari et al. (2002) | – | – | \( V_s = 90.0N^{0.31} \) |
| 14  | Hasanci and Ulusay (2006) | \( V_s = 90.82N^{0.32} \) | \( V_s = 97.0N^{0.27} \) | \( V_s = 90.0N^{0.31} \) |
| 15  | Esfahanzadeh et al. (2015) | \( V_s = 107.2N^{0.34} \) | – | – |
The inferred lithologic units are the peat/lateritic clay, sandy clay and sand. The first layer (topsoil), which is an LRZ, is characterised with resistivity values in the range of $5 < \rho < 16 \, \Omega\cdot\text{m}$ and thicknesses between $3 < t < 22 \, \text{m}$. It is composed of fillings/organic clay/peat (Ayolabi et al. 2012; Oyedele et al. 2015). The second geoelectric layer (IRZ) is characterised with resistivity values ranging from $28 < \rho < 101 \, \Omega\cdot\text{m}$ and thickness that spans $5 < t < 25 \, \text{m}$. It is interpreted as clay/sandy clay. Also, on traverses 1 and 3, pockets of peat layers are noticed unevenly in the section at different lateral (horizontal) positions, while on traverse 2, the peat layer is seen to extend laterally throughout the IRM with thicknesses between $10 < t < 25 \, \text{m}$ from the ground surface. The third layer with resistivity in the range of $102 < \rho < 1298 \, \Omega\cdot\text{m}$ is identified as sand layer. The low resistivity zone is associated with high compressibility and plasticity of these layers with relatively low resistivity values observed in all the IRMs. Subsequently, these materials have low strength (i.e. bearing capacity) and cannot support massive engineering buildings (Akintorinwa et al., 2009; Ayolabi et al. 2012; Oyedele et al. 2014). The lateral extents of the peat/clay layer are

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**Figure 3.** 2D Inverted electrical resistivity images along (a) traverse one in NW-SE, (b) traverse two in NW-SE, (c) traverse three in NW-SE and (d) traverse four in NW-SE.
well defined in the inverted resistivity sections, due to the good resistivity contrast. The progressive increase in resistivity with depths along the IRM inverted sections is because of changes in the lithological units and degree of saturation (i.e. water content) of the soils as evidenced from the borehole log data. All the boundaries between the three geological units are of sedimentary origin showing lateral and vertical changes in composition. Also, the 2D resistivity method is able to map/identify shear zone, a geological problem that should be given attention prior to selection of a site for structural developments. An increase in saturation could aid movement of peat/clay materials down to the underlying layers, making the peat/clay layers incompetent for supporting massive engineering structures.

5.1. Geoelectrical VES Results
Qualitative interpretation of the resistivity sounding data shows that the subsurface is characterised with two groups of geological models. They are a three-layer model corresponding to the H curve type, as shown in Figure 4, and four-layer models corresponding to HA, QH, HK and AH, as shown in Figure 5. In all the inverted curves, it is observed that the resistivity decreases initially and then rises with an increasing depth. The implication is that the injected current flows from a conductive layer (clay/peat) to a more resistive layer (sand). The four geoelectric sections generated from the results of the inverted model parameters (i.e. the resistivity and depth values) are shown in Figure 6. The geoelectric sections reveal that three to four substrata are delineated. Figure 6a shows that the geoelectric section along A–A’ consists of VES 1–5. The first geoelectric layer represents the topsoil with electrical resistivity and thickness values that range from 45 to 115 Ωm and 0.7 to 0.8 m, respectively. This first geoelectric layer is composed of fillings and borrowed material (lateritic soil). The second geoelectric layer represents peat with resistivity and thickness values that span from 9 to 48 Ωm and 0.8 to 22 m. The third geoelectric layer represents sand with resistivity and thickness values between 88 and 378 Ωm and depth from 5 to 52 m. The depth to the third layer is most competent for engineering purpose.

Figure 6b shows the geoelectric section along BB’ and is composed of VES 6–10. Along this profile, the first geoelectric layer represents the topsoil with electrical resistivity and thickness values that range from 58 to 111 Ωm and 0.7 to 0.9 m, respectively. The second geoelectric layer represents peat with resistivity and thickness values that range from 11 to 29 Ωm and 3.2 to 34 m, respectively. The third geoelectric layer also represents clayey sand with resistivity and thickness values that range from 57 to 134 Ωm.

Figure 4. Typical inverted 1D VES curve type for a three-layer model.
Figure 5. Typical inverted 1D VES curve types for a four-layer model.

Figure 6. Geoelectric sections trending (a) NW-SE along A-A’, (b) NW-SE along B-B’, (c) NE-SW along C-C’ and (d) NW-SE along D-D’.
and 3.2 to 46 m, respectively. The fourth stratum represents sand with resistivity values that range from 152 to 2714 Ωm with unknown value of thickness due to the termination of current injected within this depth. This layer is again most suitable for engineering purpose. The very high resistivity (2714 Ωm) delineated beneath VES 10 could not be well understood as at the time of the study. However, this has opened room for future study to be carried out using other subsurface probing techniques. The geoelectric section along CC containing VES 11–15 is shown in Figure 6c. The first geoelectric layer represents the topsoil with electrical resistivity and thickness values that range from 38 to 160 Ωm and 0.6 to 1.2 m. The second geoelectric layer represents peat with resistivity and thickness values that range from 6 to 16 Ωm and 5.9 to 22 m. The third geoelectric layer represents clay with resistivity and thickness values that range from 32 to 44 Ωm with depth in the range of 6–25 m. The fourth layer has resistivity values between 133 and 245 Ωm and is representative of sand layer a competent layer for suitable for structural developments. Figure 6d shows the geoelectric section along DD with VES 16–18. Along this profile, the first geoelectric layer has electrical resistivity and thickness values that range from 67 to 144 Ωm and 1.0 to 1.5 m. The second geoelectric layer represents clay with resistivity and thickness values that range from 23 to 39 Ωm and thickness ranges from 1.9 to 10 m. The third geoelectric layer represents peat with resistivity and thickness values that range from 10 to 14 Ωm and with thicknesses between 6.7 and 22 m. The fourth layer with resistivity values of 69–298 Ωm is composed of sand. Additionally, a comparison of the 2D ERT and VES models is achieved by superposing the column VES sections on the 2D ERT. Visual inspection, however, shows that the ERT models mapped three resistivity zones, while the VES models delineated three to four resistivity zones. The fourth layer that the 2D ERT could not map underscores the limitations of the 2D ERT in terms of depth of investigation. Although it provides wider coverage of the study area, its depth of investigation is approximately 25 m; while owing to the expanding current electrodes used in the VES survey, its depth of investigation is more than 25 m. This allows more subsurface information though at a specific point in the study area to be obtained with the VES technique. Generally, both resistivity techniques delineated the area investigated as low, intermediate and high resistivity layers.

5.0.2. Shear-wave velocity results

Following the shear-wave data acquisition, the representative shot gathers/traces of 24 vertical geophones are shown in Figure 7. These traces contain information on Rayleigh waves and P-waves. The Rayleigh wave dispersion curves are obtained by picking the maximum amplitudes on phase velocity–frequency plot by applying the frequency-wave number (F-K) technique. The F-K is one of the several techniques usually used in seismic data processing to attenuate coherent noise while maintaining the primary signals and their amplitudes. Representative of the phase velocity–frequency plots corresponding to a particular frequency content is shown in Figure 8. The maximum wavelength corresponds to the slope of the upper blue line and is equal to 65 m and the minimum wavelength determined from the slope of the lower blue line is 2.5 m. From the dispersion curves, the depth of investigation is approximately 28 m. Also, the 2-D MASW velocity models generated reveal three substrata, as shown in Figure 9. The first layer has Vs between 80 and 120 m/s and thickness between 3 and 17 m. This layer is made of loose fillings/decomposed plant materials/peat. The second and third layers are characterised with Vs in the range of 120–300 m/s and thickness that spans between 5 and 20 m. These are suggestive of clay/clayey sand layer. The fourth layer delineated has Vs that lies between 300 and 600 m/s and thickness between 5 and 16 m. This zone is identified as sand layer (Emujakporue 2011; Adegbola 2014; Adegbola et al. 2016).

5.1. Results of geotechnical investigations

5.1.1. Boring with SPT-N values

The geotechnical parameters for foundation design are estimated from SPT-N values. The boreholes data log with the SPT-N values is shown in Figure 10. The groundwater levels remain at depths of 0.25 m and 0.60 m during the drilling exercise. Visual assessment of the soil profiles show that four major lithologic layers from ground surface to a depth of about 30 m made up the subsurface strata. These are the topsoil (organic clay/sand/cobbles/filling), peat, clay and sandy deposits. These deposits are typical of alluvial sediments of swamp and creek environment (Adeyemi 1972; Longe et al. 1987; Adepelumi et al. 2008). The soft reddish lateritic clay, greyish sandy clay, loose medium- to coarse-grained sand (N4) to a depth of about 14 m. Beyond this depth, the other compositions are soft-firm yellowish clay (as undisturbed samples), medium dense yellowish silty sand (N22) and fine- to medium-grained sand (N26 – N29) to depth of 30 m for BH-1. The soil profile for BH-2 consists of loose fine-grained sand with cobbles, peat, soft greyish sandy clay and firm brownish grey clay to an approximate depth of 14 m. These geomaterials due to their nature have no SPT-N value. The other compositions include: grey clayey medium-grained sand (N12) and fine- to medium-grained sand with cobbles (N13 – N20). In Table 5, continuous sand sediments are observed in both BH-1 and BH-2.
from a depth of 21.75 m. The presence of silty to coarse-grained sand deposits reflects soil with increasing strength (medium density) with N-values between 13 and 29 (Bowles 1984). Also, the fibrous peat/organic clay deposit of low strength due to its soft nature making the collection of its undisturbed sample nearly impossible. Generally, the N-values are noted to increase linearly with depth in both boreholes due to the soil’s strength parameters such as porosity, degree of saturation and cementation of the soil matrix (Sudha et al. 2009).

5.1.2. CPT Soundings
The four CPTs results are shown in Figure 11. The soil types are identified using the classification charts since the CPT data does not provide soil sample. A low tip resistance between 2 and 40 kPa to a depth of approximately 8 m from the ground surface is encountered, suggesting the presence of very loose soil of peat/clay/sandy clay layers. Beyond the depth of 8 m, the cone resistance increases to about 65 kPa in line with the borehole log, pointing at fine-/medium-grained sand as corroborated in the results of (Oyedele et al. 2015). At depth between 9 and 16.2 m, the tip resistance increases from 75 to 106 kPa, signifying the occurrence of medium dense grained sand. This zone is expected to be the load-bearing layer.

5.1.3. Variation of SPT-N values and Vs with depth
Plots of SPT-N values (dotted curve) and Vs (solid curve) with depth for BH-1 and BH-2 are shown in Figure 12. There is resemblance in the pattern of the
plots that suggest a good relationship between the two parameters. This means that the N-values and Vs increase with depth. This increase reflects the dependence of these parameters on the soil's strength properties such as grain size, degree of saturation and cementation of soil matrix (Sudha et al. 2009). The plots show that loose unconsolidated soils are encountered at shallow depth, while medium to dense soils are prevalent at greater depth corroborating the lithologic units in BHs1 and 2. As regards the soil strength, it could be suggested that there is an increase in the degree of compactness of the soils from loose or unconsolidated to consolidated soils. This indicates that the competence or integrity of the soil is high at greater depth. The low SPT-N values and corresponding low Vs are associated with peat/clay layers at a shallow depth with low strength and bearing capacities.

5.1.4. Empirical Relation for Vs and SPT-N Values

Figure 13 shows the in-situ Vs correlated with the SPT-N values for the two boreholes. The generalised form of a regression equation for sandy soil is given by:

$$V_s = p \times N^q$$  \hspace{1cm} (10)

where p and q are regression coefficients that depend on soil types and locations (Sudha et al. 2009; Kirar et al. 2016). In Table 4, it is seen that most of the
Figure 9. 2D inverted S-wave velocity models along traverses 5–8
authors proposed values for p and q in the range of 31–125 and 0.3–0.54 for sandy soils, respectively. For this study, however, the values for p and q are in the range of 96–144 and 0.17–0.42, respectively. This agrees very well with several such empirical correlations already established. Also, to know the degree of the relationship between the Vs and N-values, a statistical correlation was estimated based on the least squared technique. This gives the correction coefficient (r) that ranges between 0.64 (good) and 0.97 (high). The good to high correlation coefficients produced suggest that the SPT-N value has a major effect in the prediction of Vs. The regression models developed along with the regression coefficients and correlation coefficient are given in Table 6. On comparison with some of the previous works in Table 4, especially in the works of Sykora and Stokie (1983); Okamoto et al. (1989); Pitilakis et al. (1992); Raptakis et al. (1995) and Esfahanizadeh et al. (2015) shows good agreement, thus adequate for the prediction of the Vs for soils.

5.1.5. Generalised Subsoil Sequence/Competence Evaluation

The results of the geotechnical and geophysical surveys reveal that subsurface conditions in the study area have four strata to a depth of about 30 m from the ground surface. The first two layers are made of loose peat/silty sand and organic clay/sandy clay deposits delineated at depth of about 20 m. The 2-D IRM shows pockets of peat/clay layers dispersed unevenly within the subsurface reflecting the instability of the ground conditions. This shear or weak zone shows low resistivity of 9.6–160 Ωm, very low shear-wave velocity range from 118 to 292 m/s, low peneration resistance of 2–40 kPa and low bearing capacity in the range of 49–154 kPa. The high compressibility with the tendency to shrinkage when dry, high porosity and low shear strength characteristics of peat and clay soils make them unsuitable for and sometimes inimical to hosting the foundation of most construction works, particularly when medium to superstructures are of interest. These soils, even when they are not under a long-term process of compression and stiffening, are more susceptible to subsidence (Soupois et al. 2007). The mechanical strength of the subsoil increases with depth from the third layer made of clayey sand deposits to the fourth layer with composition of sand. These layers exhibit relatively high resistivity ranging from 120 to 2714 Ωm, high shear-wave velocity between 367 and 550 m/s, high peneration resistance of 56–106 kPa and high bearing capacity of 158–254 kPa, as presented in Table 7. The fourth layer in the boreholes 1 and 2 has sand sediments with cumulative N-values of 50 and 77 at depths of 21–30 m, respectively. This is suggestive of dense sand material, and this layer

Figure 10. Sampled boreholes data at depth 30 m showing the SPT-N values and soil strata.
### Table 5. Summary of depth, SPT-N value and soil-type characteristics.

| Depth (m) | BOREHOLE 1 | BOREHOLE 2 |
|-----------|------------|------------|
|           | SPT-N Value | Relative Density | Depth (m) | SPT-N Values | Relative Density |
| 18–21.75  | N 1,2,2(4)  | Loose sand material | 13–14.5   | N 1,2,2(4)  | Loose sand material |
|           | N 2,3,5(8)  |                   | 21.75–30  | N 1,2,2(4)  | Loose sand material |
| 21.75–30  | N 1,2,2(4)  | Tight sand material | 21.75–30  | N 1,3,4(8)  | Tight sand material |
|           | N 3,5,7(13) | Dense sand material | 21.75–30  | N 4,6,8(22) | Dense sand material |
|           | N 6,7,9(17) |                   | 21.75–30  | N 7,8,9(26) | Very Dense sand material |
|           | N 8,9,11(20)|                   | 21.75–30  | N 9,10,12(29)| Very Dense sand material |

### Figure 11. CPT sounding curves of the cone tip resistance versus depth within BH 1 and BH 2 with a column of the soil type.
represents a competent layer for the foundation of any engineering structures in the study area. This is also in consonance with boreholes log data, BHs-1, 2 and the different lithological units identified.

Structural failure could take place when foundation movements occur in structures standing on peat/clay soils particularly when shrinkage takes place. In this study, our findings show that most of the foundation-
related problems in the area are due to the presence of the peat/clay soils which change in volume because of continuous flow of water from a nearby canal and the soil becomes expansive. Consequently, these soil types tend to display high plasticity. This is influenced by the very small sizes of clay particles, and in the presence of water, the individual clay ion’s crystalline structure grows in size. When all the clay ions/molecules underlying a foundation footing absorb a large quantity of water, the soil in turn grows in size. This could lead to the formation of large cracks in the ground surface and/or fissure in the foundation (i.e. differential settlements) caused by the shrinkage of the clay soil.

Table 7 Details of collected database: Number of profile layer, depth and range of soil properties (Vs, Y, Qult, Qa, k0) obtained for the study area.

5.2. Conclusions

The use of multi-parameters approach has been carried out to ascertain the subsurface conditions in the investigated site arising from the visible structural defects such as the tilting or cracking/collapse of engineering structures with a case study of Gbagada in Lagos, southwestern Nigeria, leading to the deployment of geophysical and followed by geotechnical techniques. The following conclusions can be drawn from the different techniques used and the results obtained:

(i) The inverted images/models from the geophysical techniques revealed that the area is composed of three and four geological layers with variations in resistivities and velocities. The geomaterials are lateritic clay/fine sand/peat, clay, clayey sand and sand with peat/clay as the dominant lithological layers. This is corroborated by the borehole logs obtained. The boreholes indicated the presence of four main lithologic layers which are lateritic clay/fine sand/filling, peat, clay and sand deposits. The peat and clay layers are incompetent as foundation soils for heavy engineering structures.

(ii) The peat/clay materials are characterised with low resistivity values (5–35) Ωm, low shear-wave velocities (80–300) m/s and low penetration resistance (0–20) kPa.

(iii) The CPT soundings attained refusal resistances between 64 and 106 kPa at depth of 13–16 m indicating that dense course sand material was encountered. This could be considered suitable for hosting the foundation footings of massive engineering structures.

(iv) Our findings show that the existence of conductive materials (peat and organic clays), which adversely affect the subsurface conditions due to their expansive nature.

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Table 6. Regression models for Vs and SPT-N values.

| Soil type | Traverse No | BH-1 | BH-2 |
|-----------|-------------|------|------|
| Sand      | 5           | Vs = 96.708 N°.5, r = 0.64 | Vs = 109.27 N°.48, r = 0.69 |
| Sand      | 6           | Vs = 132.02 N°.35, r = 0.86 | Vs = 114.68 N°.40, r = 0.96 |
| Sand      | 7           | Vs = 117.45 N°.30, r = 0.94 | Vs = 134.33 N°.41, r = 0.97 |
| Sand      | 8           | Vs = 144.94 N°.31, r = 0.75 | Vs = 135.74 N°.31, r = 0.82 |

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Table 7. Details of collected database: Number of profile layer, depth and range of soil properties (Vs, Y, Qult, Qa, k0) obtained for the study area.

| Survey / Traverse No. | Layer | Depth(m) | Shear-wave velocity Vs (m/s) | Total Unit Weight (y)/kN/m² | Ultimate Bearing Capacity Que (kPa) | Allowable Bearing Capacity Qa (kPa) | Coefficient of Subgrade Reaction k0 (kN/m²) | Lithology |
|-----------------------|-------|----------|-----------------------------|------------------------------|----------------------------------|-----------------------------------|---------------------------------------------|----------|
| 5                     | 1     | 0–3      | 176                         | 15.7                         | 275.7                            | 68.9                              | 275.6                         | Fillings/Borrowed Materials/Peat |
|                       | 2     | 3–13.5   | 134–275                     | 14.6 – 17.5                  | 196.0–481.5                      | 49.0–120.4                       | 1960.4–4815.4                  | Clay     |
|                       | 3     | 13.5–21.0| 281–343                     | 17.6 – 18.5                  | 494.7–634.7                      | 123.7–158.7                      | 4947.1–6347.3                  | Clayey Sand |
| 6                     | 1     | 0–7.1    | 152–183                     | 15.1 – 15.8                  | 229.5–289.4                      | 57.4–72.4                        | 2294.9–2894.2                  | Peat     |
|                       | 2     | 0.2–21.4 | 276–335                     | 17.5 – 8.4                   | 483.7–616.3                      | 120.9–154.1                      | 4837.3–6162.7                  | Clay/Sand clay |
|                       | 3     | 7–48     | 423–497                     | 19.5 – 20.3                  | 824.9–1009.1                     | 206.2–252.3                      | 8248.9–1090.5                  | Sand     |
| 7                     | 1     | 0–7      | 118                         | 14.2                         | 167.2                            | 41.8                              | 1672.3                        | Fillings/Borrowed Materials/Peat |
|                       | 2     | 0.2–10   | 176–199                     | 15.7 – 16.2                  | 275.7–321.4                      | 68.9–80.3                        | 2756.5–3213.9                  | Clay     |
|                       | 3     | 8–23     | 285–368                     | 17.7 – 18.8                  | 503.5–693.1                      | 125.9–173.3                      | 5035.3–6930.7                  | Clayey sand |
|                       | 4     | 23–48    | 370–501                     | 18.9 – 20.3                  | 697.8–1019.2                     | 174.4–254.8                      | 6977.8–10192.1                | Sand     |
| 8                     | 1     | 0–15    | 164–182                     | 15.4 – 15.8                  | 252.4–287.4                      | 63.1–71.9                        | 2523.6–2874.2                 | Peat     |
|                       | 2     | 0.2–21   | 241–437                     | 16.9–19.7                    | 408.3–859.2                      | 102.1–214.8                      | 4083.1–8591.5                 | Clay/Sandy Clay/ Clayey sand |
|                       | 3     | 21–48   | 444–515                     | 19.7–20.5                    | 876.4–1054.9                     | 219.1–263.7                      | 8763.9–10549.4                | Sand     |
have been identified as the possible cause of structural mishaps been witnessed in the study area.

(v) Soil stabilisation techniques can be introduced to considerably improve the dynamic properties of the expansive soils. Among the materials that can be used as additives to stabilise the soils are lime, cement, resins or fly ash.

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No potential conflict of interest was reported by the author(s).

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