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Subsurface investigation for pre-foundation study utilizing integrated geophysical and geotechnical methods, UNILORIN campus

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Abstract
This study involves subsurface investigation of a proposed site within senior staff quarters at the University of Ilorin campus utilizing the electrical resistivity technique, magnetic method and geotechnical analysis for the pre-foundation study. The land magnetic survey was acquired at 20 m spacing along seven traverses with the use of Geometrics G857 Proton Precession Magnetometer revealing the presence of concealed basement contact separating the biotite schist in the north from granite gneiss in the south. The Schlumberger configuration was used for the 1-D vertical electrical sounding (VES) technique, with the current electrode spacing (AB/2) varied between 1 and 100 m, which revealed three to four geo-electric layers which include lateritic/topsoil (93-3053Ωm, 0.3–2.5 m), clayey-sand (127-639Ωm, 1–13.2 m), weathered basement/sandy-clay (55-767Ωm, 3.6–33.3 m), and fresh basement (above 1000Ωm). WINRESIST and Surfer were used for the processing of resistivity data while Microsoft Excel was adopted for ground magnetic processing. However, the geotechnical analysis conducted on undisturbed samples picked at different depths from eight pits dug to a depth of 2.3 m revealed the following: bulk density (1.80 g/cm^3-1.99 g/cm^3), dry density (1.67 g/cm^3-1.86 g/cm^3) liquid limits (22.3%-34.1%), plastic limits (12.6%-19.8%), plasticity index (9.7%-14.3%) and cone penetration test (89.8 kN/m^2-2110.3 kN/m^2 at a depth of 0.1 m to 0.3 m), in which all these geotechnical analysis results classified the soil to be competent in variations and assess soil mechanical behaviour to support structural loads within study area. The integration of geophysical and geotechnical results revealed high topsoil resistivity correlates with high bearing capacity and is interpreted as thick and compacted lateritic materials. Highlights:

i. Laying of foundation across the contact zone should be avoided and attention must be taken when selecting the foundation's materials.
ii. Excavation of the soil to a depth where it is sufficiently consolidated and competent to support any type of construction.
iii. The use of a shallow foundation type, such as a raft foundation in areas of lower competence and strip footing in competent areas with large bearing capacities.

Keywords Pre-foundation Investigation · Electrical Resistivity Method · Magnetic Method · Geotechnical Analysis · Construction Design

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1 Introduction

The use of geophysics in the domains of civil and environmental engineering has gained popularity throughout the last ten years. The growing number of structural failures in Nigeria, involving the collapse of buildings, bridges, and roads, has become a cause for concern for both Federal and State Governments. Consequently, these governing bodies have directed their town planning departments to develop regulatory codes for buildings taller than one storey [1]. A thorough understanding of the subsurface through the integration of geophysical methods and soil analysis is necessary to successfully design and construct foundations, especially for large structures like buildings, dams, highways, and bridges. Important details on the bedrock's topography, the soil's stability, the subsurface layers' arrangement, and its structural properties are needed. This information is necessary to set up structures that complement the bedrock's properties. Utilization of integrated geophysical methods and geotechnical soil analysis provides data that are used for a comprehensive assessment of the subsurface conditions which helps to minimise risks associated with construction failures [2]. Based on this data, civil engineers evaluate the soil's ability to support loads and identify the most appropriate base for engineering initiatives [3]. One of the most crucial stages in the construction of any building is the foundation design. It transfers the building's weight to the ground and keeps the structure together. Ground conditions should have a big influence on foundation design in Nigeria, thus it's important to consider them when creating a building's foundation. The soil's strength, kind, and moisture content are the primary factors influencing foundation design. Soils can range from soft clays, silts and sands to more solid rock formations. Soft soils have a higher propensity to settle, therefore building foundation design in these areas needs to be carefully considered. Soil strength can affect foundation design since stronger soils can support larger loads than weaker soils. The moisture content of the soil is an essential consideration when building a foundation since it may have a significant influence on the soil's stability and capacity to expand or contract.

The issue of building foundation condition is often addressed at the very late phases of engineering geophysics when earthquake damage is either expected or observed [4, 5]. When building a structure, geophysics can be utilized for exploration to yield valuable data about the early identification of potentially hazardous subsurface conditions [6, 7]. Understanding a region's geological basement is also important [8]. In order to determine the depth of competent subsurface geologic layers that are stable and appropriate for the development of engineering foundations, geoelectric methods are therefore frequently used in engineering foundation investigations. These methods typically demonstrate a clear contrast in the geoelectric characteristics of subsurface geologic materials [9]. In addition, this approach offers a thorough way to estimate the subsurface parameters such as the thickness, depth, and apparent resistivity to establish conductive zones. This is quicker and more affordable than some other geophysical methods. Any engineering structure that is planned must consider soil strength indices. These indices are essential for the stability and competence determination of the materials within the Earth that will support the structure. This evaluation sheds light on the soil's ability to resist strain and stress from a variety of causes, such as cracking, swelling, overburden, pore pressure, and other conduct triggered by humans [10]. Numerous variables have been connected to foundation failures, such as poor foundation design, the use of inferior building materials, and insufficient knowledge about the soil and subsurface geological elements [11]. These elements, due to stressed soils' nonlinear behaviour and the challenge of accurately calculating soil parameters in situ, can result in significant losses and structural damage. Thus, for appropriate engineering foundation planning and management, these properties must be estimated and characterized. In most cases, civil engineers use the evaluation of soil competence for foundation studies to help them design the foundations for buildings and other engineering projects.

Geophysical approaches have been employed in foundation investigations [12–14]. This is dependent on the physical characteristics of the several strata that comprise the Earth's subsurface having a discernible and substantial contrast. These characteristics provide information on the kind and composition of the materials that make up the Earth, including density, electrical resistivity/conductivity, velocity, dielectric permittivity, and acoustic qualities. The strength and compressibility of soils are mostly determined by the presence of clay minerals, according to geotechnical research [15, 16]. The soil's clay concentration has a direct bearing on soil competence since it is correlated with soil cohesion, friction angle, and plasticity index [16]. The integration of geophysical and geotechnical methods offers significant advantages for pre-foundation studies, where geophysical methods provide a broad overview of the subsurface condition and geotechnical soil investigation offers detailed soil physical and chemical properties of the subsurface. By combining these methods, the approach helps to identify variations in soil strength and composition, locate potential hazards such as sinkholes or soft zones, and ensure that the foundation is made to the specific soil conditions of the site [17–19]. Direct measurement of soil properties like density, shear strength, and permeability is made possible by combining geophysical methods

with geotechnical soil analysis. These measurements are important for comprehending the mechanical behaviours of the soil under load, especially concerning its capacity to support building foundations [20, 21]. To ensure structural integrity, longevity, and safety in any construction project, geotechnical soil analysis and geophysical methods such as electrical resistivity and ground magnetic method ought to be integrated into pre-foundation studies [21–24]. Before starting construction, the properties of the subsurface are assessed using geophysical methods and geotechnical soil analysis. The success of this type of work primarily depends on how well the underlying rock layers and soil can support the imposed loads without experiencing excessive settlement, deformation, or instability [21, 23]. The results of this investigation will be valuable in providing a detailed understanding to guide future construction activities, ensuring the safety and durability of infrastructure development projects in Nigeria.

Preceding this research, numerous building collapse accidents in Nigeria have been documented. The Council for the Regulation of Engineering in Nigeria (COREN), which oversees the regulation of the engineering profession in the country, reported twenty-two (22) building collapses in Nigeria resulted in thirty-three (33) fatalities from January to July 2024 [25]. Twenty-one (21) of the eighty-four (84) collapse occurrences that were reported between 2011 and 2019 occurred outside of Lagos. Remarkably, of these events, 41% involved already-existing structures, and 59% involved buildings that were still under construction. Over 42 people lost their lives when a 21-story building at 44 Gerrard Road, Ikoyi, that was still under construction collapsed on November 1, 2021. According to the Building Collapse Prevention Guild, there were at least 62 mishaps in 2022 that resulted in 113 injuries and 84 fatalities. Twenty cases were recorded from Lagos, five cases each from Anambra and Kano, and four cases each from Jigawa and Delta [26, 27]. The need for housing arises from the substantial urban population, potentially prompting some property developers to compromise standards in their efforts to meet the high demand. Building professionals argue that structural failures occur either due to weaknesses in the materials employed or instability in the building's design. However, instances of substandard material usage, inadequate or insufficient soil testing, deficient supervision, corruption within government agencies, and a shortage of building professionals on-site have also been identified as contributing factors to building failures. The menace of building collapse in Nigeria needs urgent attention, hence this study focuses on the integration of surface geophysics and geotechnical analysis at the proposed senior staff quarters UNILORIN, this could serve as a model to be used to avoid potential foundation failure. It is known that the geotechnical analysis usually best understands the subsurface, which normally yields highly localized, point-specific data (1D). These data points are obtained from boreholes, Standard Penetration Tests (SPT), or Cone Penetration Tests (CPT), and they give precise, direct information on the conditions at particular sites. However, the number of these tests is often restricted because of their high cost and time requirements, which leaves gaps in the subsurface data between test locations. Conversely, geophysical techniques such as magnetic surveys and electrical resistivity offer continuous data across wider regions, effectively bridging the gaps between geotechnical data points. This research aims to carry out a comprehensive subsurface investigation on the site on the Unilorin campus using both geophysical and geotechnical methods with the objectives of mapping the subsurface geology, assessing soil properties, and identifying any potential risks that may affect the stability of future construction project within the study area and also offer valuable knowledge for future construction projects with similar geology and across Nigeria and beyond.

2 Physiography and geology of the study area

The research site is situated in the Ilorin main campus in Kwara State, Nigeria as shown in Fig. 1. It is bounded to the south by the open natural vegetation, and in the east and the north by the Unilorin Teak plantation, to the west by the Unilorin pure water factory, a fisheries department facility that incorporates the ponds sited by a stream running in NNE-SSW direction with the elevation of about 306 m, and farmlands. The site covers an area of 360,000 square meters, it was designated for phase II residential development catering to the senior staff of the institution. The elevation within the site ranges between 321—342 m. The university campus occupies an expansive area totalling 37,000 acres of land. Positioned approximately 16 kms east of the Ilorin Township, the school is geographically located between longitude 4° 35E to 4° 42E and latitude 8° 27N to 8° 32N. Ilorin's climate is distinguished by distinct wet and dry seasons. Typically, the rainy season begins in late April and lasts until October, whereas the dry season runs from November to April. From November to January, Ilorin has temperature fluctuations of 33 °C (91.4^{0}F) to 35 °C (95^{0}F), and from February to April, it ranges between 34 °C (93.2^{0}F) and 37 °C (98.6^{0}F). It is very hot during the dry season. The range of the area's yearly rainfall is 990.3 mm to 1318 mm. Ilorin City experiences twice as much rainfall, with more temporal and geographical

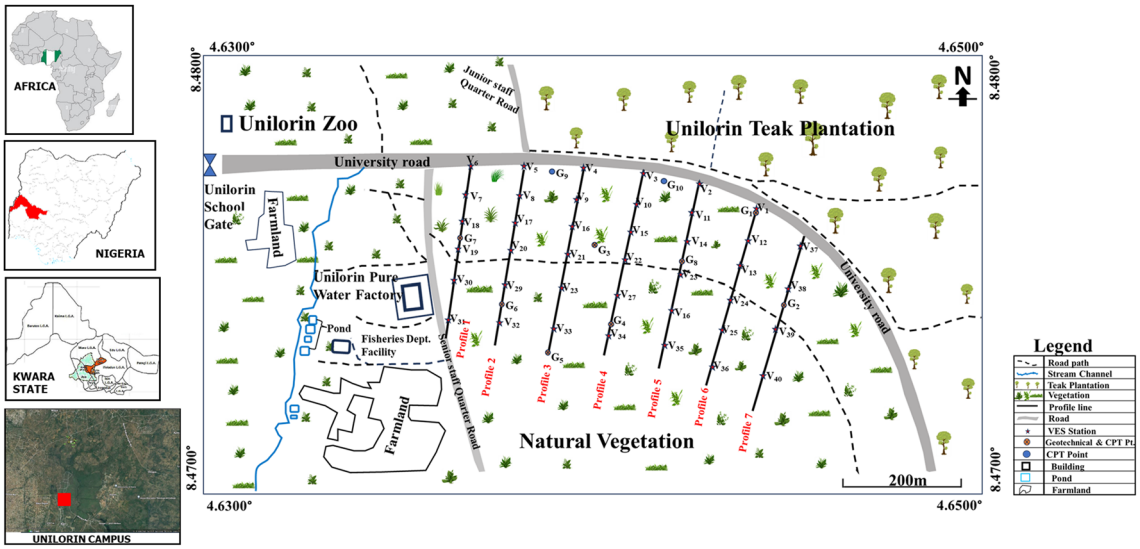


Fig. 1 The basemap of the study area within the University of Ilorin

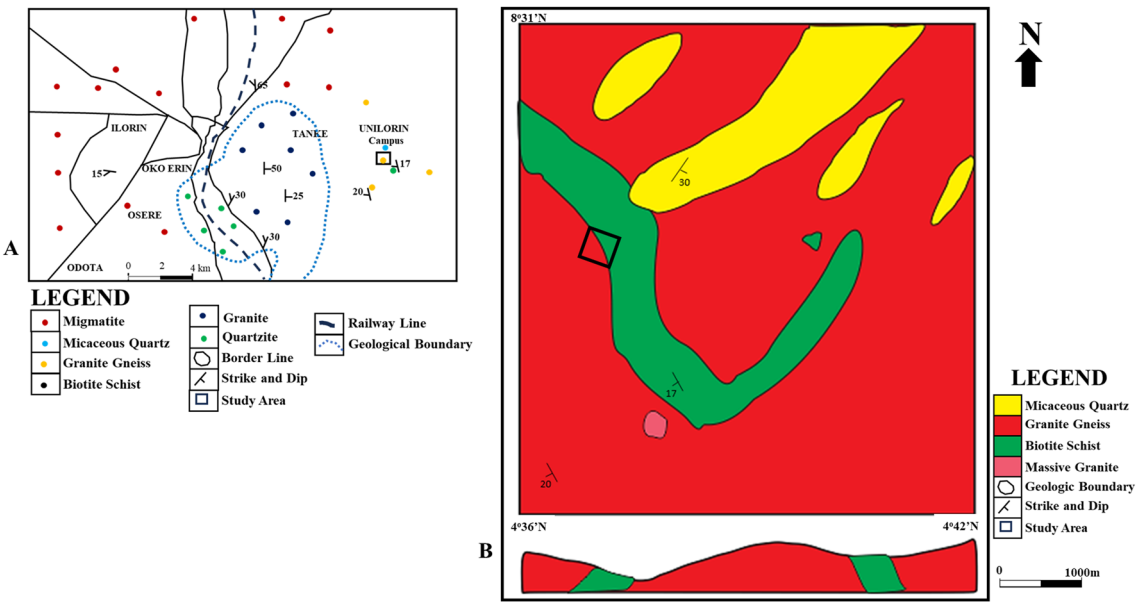


Fig. 2 Geological Maps: **a** Ilorin **b** Part of the University of Ilorin Permanent Site (modified after Annor and Olasehinde [30])

variability. Relative humidity in Ilorin City ranges from 35 to 80% in the dry season and from 75 to 88% in the May–October period [28–30].

2.1 Geological setting

According to Annor and Olasehinde [30], Ilorin is located in Nigeria's basement complex terrain, which is distinguished by a mixture of medium-grade and low-grade metamorphic rock (Fig. 2a). Evidence of occasional granitic suites is found in the region. Several groups of outcrops identified within the University's main campus are gneiss complex, granite suites, metasediments, amphibolites, and veins (Fig. 2b). Annor and Olasehinde's findings show that an NNE-SSW foliation is developing which is responsible for the production of important geological features like the synform and the antiform [30]. The Eburnean and Pan-African orogenies appear to have had an impact on the crystalline basement rock found in both Kwara State and the University of Ilorin campus. Granitic materials were introduced by the Pan-African orogeny,

but the magnetization and metamorphism of earlier metasediments were facilitated by the Eburnean orogeny. As seen in Fig. 2b, the research area is underlain by both biotite schist and granite gneiss.

3 Materials and methods of study

This research involved the integration of Electrical resistivity and Ground Magnetic method along with geotechnical analysis, through the use of primary equipment such as Herojat Ultra-Smart Resistivity meter for measuring the subsurface resistance with other supporting materials like electrodes, hammer, cable wires, GPS reader, tape meter and field note which are essential for carrying the electrical resistivity survey of vertical electrical sounding (VES) of the research site. Then, the calculated subsurface resistivity values were processed using Winresist 1.0 Version and Surfer Software. For the ground magnetic survey, the Geometrics G857 Proton Precession Magnetometer was used to measure the magnetic field's total field, alongside the GPS reader to take coordinates. The acquired magnetic fields were plotted and processed using Microsoft Excel software. Geotechnical analysis equipment and materials were used, such as the dynamic cone penetration, auger, shovel, tape meter, GPS reader, and sample bags, for the collection of soil samples. The soil samples were then analyzed in the University of Ilorin Geology Laboratory for geotechnical studies.

3.1 Electrical resistivity method

The Schlumberger configuration was used for the 1-D vertical electrical sounding (VES) technique, which operates on the principle that subsurface materials have different resistivity values depending on the moisture content, dissolved electrolytes, conductivity of minerals and soil composition. A total of forty (40) VES stations were systematically placed on seven profile lines with the current electrode spacing (AB/2) varied between 1 and 100 m within the study area (Fig. 3a). After the measurement of the resistance, the resistivity value was calculated by multiplying the measured resistance with the geometric factor of the configuration, and then both qualitative and quantitative data analysis were done, using partial curve matching techniques and plotting apparent resistivity versus electrode spread (AB/2), with computer-aided 1D forward modelling facilitated by Winresist 1.0 Version software.

3.2 Magnetic method

With a resolution of 0.1 nT, the Geometrics G857 Proton Precession Magnetometer was used to gather 2.72 line kilometres of the earth's magnetic field's total field component along seven (7) profiles spaced 100 m apart from one another (Fig. 1 and Fig. 3b). The device is a portable magnetometer independent of location, temperature, or sensor orientation. Additionally, it is equipped with an integrated global positioning system (GPS) that logs each measuring station's coordinates (Fig. 3b). To counteract the diurnal impact, the base station was positioned around a kilometer from the site in a magnetically calm area. Along the profile, measurements of the magnetic intensity were recorded every 20 m. Upon completion of the fieldwork, all collected field data were loaded into a computer for further analysis. The obtained ground magnetic data was adjusted for diurnal variation due to the time variability of the earth's magnetic field by calculating the variance in the base station's magnetic measurements over time. The difference is applied to the stations' measurements obtained during that specific time frame i.e., added if the difference is negative or deducted if the difference is positive. The calculated results were plotted using the Excel spreadsheet.

3.3 Geotechnical study

3.3.1 Site preparation

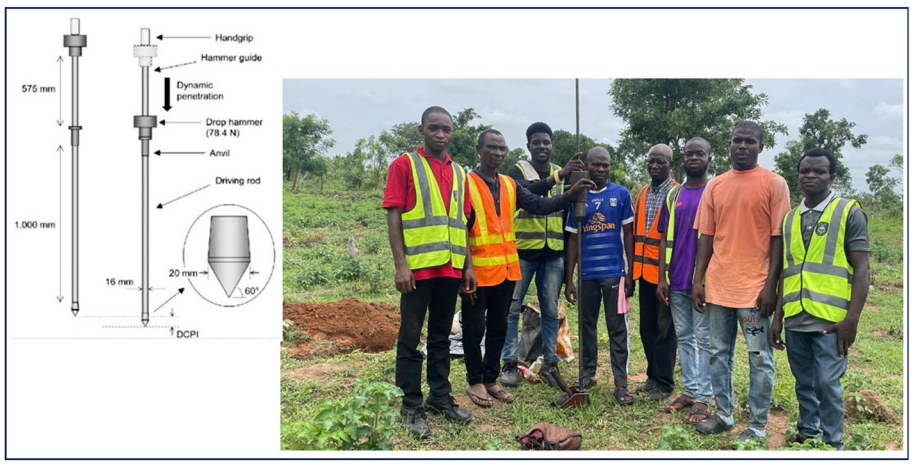
This started with the identification of the exact location where the sample will be taken, surface debris (vegetation and loose soil) was removed using a shovel, this was done to minimise contamination of the sample. An Auger was used to drill to the required depth where the undisturbed sample will be collected. The rotation speed was lowered to prevent excessive disruption of soil structure. Cleaning of the auger bit was frequently done to avoid over-compaction or disturbing deeper layers. Once the desired depth was reached, the auger was carefully removed to avoid borehole wall collapse. Cleaning of the sides of the borehole with minimal disturbance to remove any loose or smeared soil created during the augering process. This is important for ensuring that the sample collected is as undisturbed as possible. A



a: Data acquisition with Herojat Ultra-Smart Resistivity meter



b: Data acquisition with the Geometrics G857 Magnetometer



c: Team of researchers in the study area with the dynamic Cone Penetration equipment

Fig. 3 **a** Data acquisition with Herojat Ultra-Smart Resistivity meter. **b.** Data acquisition with the Geometrics G857 Magnetometer. **c.** Team of researchers in the study area with the dynamic Cone Penetration equipment

thin-walled tube, like a Shelby tube, into the hole created by the auger. A thin-wall tube was driven gently into the soil manually to avoid excessive compression and minimise disturbance. The tube was carefully removed, and both ends of the tube were sealed to prevent moisture loss, and contamination, and to also preserve the sample's natural state. The samples were properly labelled with depth, location, and date, stored in a protective container and later transported to the laboratory for analysis. After sampling, the borehole was backfilled with native material.

3.3.2 Sample collection

Eight soil samples were collected with fair distribution within the research area (G1-G8) as defined by the interpretation results of both magnetic and VES data for laboratory analysis. A total of ten points (G1-G10) were investigated for the dynamic cone penetrometer test. Soil samples were collected from test pits excavated from locations G1 to G8 at depths of 0.6, 0.8, 1.2, 1.5, 1.6, 1.9, 2.1, and 2.3 m, to ensure an appropriate representative of the subsoil under the research area (Fig. 1). A GPS was used to document the coordinates and height of every sampling location. After being appropriately labelled and packed in a polythene bag, each soil sample was taken to the lab for examination. The water content, specific gravity, bulk and dry densities, particle size distribution, Atterberg consistency limits and compaction were measured based on British Standards BS 1377 [31]. The only in-situ geotechnical test carried out in the study area (G1 – G10) is the dynamic cone penetrometer test (DCPT). DCPT is a commonly used method for field research and subsurface soil layer condition assessment because is notable for its exceptional capacity to yield consistent outcomes at a reasonable cost [32, 33]. There are numerous ways to use DCP testing to describe the properties of soil. However, cone penetration's most notable benefit could be its ability to continuously record relative soil strength with depth [33]. A 20 mm diameter, 60-degree cone, and a 10 kg hammer were used to measure the number of blows (n) needed to achieve a maximum depth penetration of 1000 mm (1 m). The readings and soil load-bearing capacity were shown to be correlated using the equation presented by Ampadu [34] below:

$$q_{allow} = 44.9n$$

where q_{allow} is defined as "allowable load" or "allowable stress." It represents the maximum load or stress that a material, structure, or soil can safely withstand under specific conditions without failing or undergoing unacceptable deformation. Figure 3c is the picture of our team members shortly after sampling with the dynamic Cone Penetration equipment.

3.3.3 Sample preparation and laboratory procedure

An auger was used for the collection of samples of the undisturbed soils in the research area. To avoid water loss, the soil samples for the water content study were carefully stored in polythene bags before being transported to the laboratory. The remaining soil samples for grain size analysis, Atterberg consistency limits, and compaction were air-dried prior to these analyses being carried out in the laboratory. The soils intended for bulk and dry densities were extracted from the in-situ soils using moulds of known weight and volume, respectively. Based on British standard BS 1377 (1990), all fundamental geotechnical characteristics of the studied soil samples were undertaken and are determined as follows:

3.3.4 Water content determination

The BS 1377 (1990): Part 2.3.2 provides specific instructions for the moisture content test. Using the weighing balance, a non-corrodible can weighed as (w_1). After each wet soil sample was placed into the can and weighed as (w_2), after oven-dried for 24 h at a temperature between 1050 and 1100 degrees Celsius, and weighed (w_3). The water content of the sample was then calculated using the mathematical expression below:

$$Water\ Content = \frac{w_2 - w_3}{w_2 - w_1} * 100$$

3.3.5 Bulk and dry densities determination

The methods used for the bulk and dry densities followed BS 1377 (1990). The unit weight of the moist soils was divided by their volumes to obtain the bulk density. The dry density of the soils was calculated by dividing the dry weight obtained after oven drying, by their volume.

3.3.6 Specific gravity determination

The sample's specific gravities were measured in compliance with BS 1377 (1990): Part 2:8:3, the British Standard. An empty measuring cylinder was weighed as w_1, then soil was added to one-third of its volume and weight as w_2. The soil in the measuring cylinder was filled with water, and the weight as w_3. Thereafter, the measuring cylinder was emptied, cleaned and filled with distilled water and its weight was recorded as w_4. The formulas below were used to compute the specific gravity of the soils:

$$Specific\ Gravity = \frac{(w_2 - w_1)}{(w_4 - w_1) - (w_3 - w_2)}$$

3.3.7 Analysis of grain size

The particle size distribution was assessed using a hydrometer and a mechanical sieve. As stated in BS 1377 (1990): Part 2:9:3, particles bigger than 0.075 mm were submitted to the mechanical sieve technique utilizing various BS test sieves. The hydrometer approach was used for particles with a size less than 0.075 mm sieve by utilizing Stoke's Law to determine their settling velocity. The procedure is outlined in Part 2:9:4 of BS 1377 (1990). For the hydrometer analysis, jar cylinders, stopwatch, sodium hexametaphosphate dispersant solution, and hydrometer bulb were utilized.

3.3.8 Atterberg consistency limits

The nature of fine-grained soil can be determined using the Atterberg consistency limit tests. It is composed of three limits: shrinkage, plastic, and liquid, in that order. In BS 1377 (1990), these consistency restrictions are explained as follows: Sects. 2:5:3, 2:4:3, and 2:5:4. The liquid limit of the soil samples under study was determined by first crushing a fresh, dry sample using a pulverizing machine. The material was then sieved through a 0.425 mm sieve, 300g of which was weighed, and water was added until a stiff paste was formed. After filling the Casagrande apparatus with the stiff paste, the surface was levelled and smoothed using a spatula. A cutting groove was produced in the soil, and the handle (blows) was spun until the groove closed (Note that the counter counts the number of rotations the handle makes as it strikes). A tiny paste was scooped into the moisture container using a spatula, weighed, and dried in the drying oven for 24 h before being reweighed to ascertain its water content. The minimum and maximum number of blows were 10 and 50, respectively. After that, the soil is dumped from the cup onto the glass plate, water is added, and the procedure is repeated.

Three balls were formed by scooping out a little portion of the paste from the Casagrande cup, rolling it into thread until it broke, and then placing the broken threads into three separate moisture cans. The balls were then set on a glass plate. The moisture content of each sample was then measured and used as plastic limits by oven-drying the cans containing the crumbled soil samples for 24 h.

The soil sample was well mixed with distilled water for the linear shrinkage calculation. Next, a linear container of a given length was taken and lubricated to facilitate the soil sample's easy removal. After that, the soil sample was placed inside the container, and its initial length was determined to be (L1) The container was allowed to cool in a desiccator before being oven-dried for around 24 h, after which the dried soil sample's ultimate length (L2) was measured. The following formula was used to estimate the linear shrinkage:

$$LS = \frac{L_1 - L_2}{L_1} * 100$$

3.3.9 Compaction test

Each sieved soil sample weighing three kilograms was poured into the mixing pan, and 1**20 cm^3 (4%)** of water was added to the soil in the mixing pan and mixed thoroughly using a spatula to ensure uniform moisture distribution. The soil was divided into three layers for the Standard Proctor and five for the Modified Proctor. The cylinder mould is then placed on a base plate, then a representative soil specimen is added to the mould and compacted with 25

evenly distributed blows of rammer for Standard Proctor. This represents the first layer. After the compaction, the volume of the soil in the mould was reduced, more soil specimen was added into the mould and compacted with another 25 evenly distributed blows. This was repeated the third time and excess soil was trimmed to ensure a smooth surface level. While the modified Proctor consists of five soil specimen layers and fifty-five uniform rammer strokes per layer. The soil-filled mould was weighed, and samples of the soil were obtained to evaluate the dry density and water content, respectively. This procedure was repeated four times for both standard and modified proctors respectively. The following mathematical expression was used to estimate both wet and dry density.

$$Wet\ density\left(\rho_w\right) = \frac{Mass\ of\ Compacted\ Soil(m)}{Volume\ of\ Mold(V)}$$

$$Dry\ Density\left(\rho_d\right) = \frac{\rho_w}{1 + \frac{Moisture\ Content(w)}{100}}$$

4 Results and discussions

4.1 Surface geophysics—Electrical resistivity

Multiple parameters, including porosity, soil water content, void ratio, grain size distribution, density, and permeability influence the subsurface characterization utilizing geoelectrical resistivity. For example, high air-filled void soil with low soil water content will have high geoelectrical resistance. In a lithology with finer grain size, the soil porosity will also decrease, raising the resistivity values. In addition, soil types with air-filled voids will have higher geoelectrical resistivity values than soil types with water-filled voids. The observed geoelectrical resistivity is often low in fine-grained subsurface materials, like clay, where the soil water content is higher. The predominant curve types generated from the forty vertical electrical soundings obtained within the study area are AQ, KA, KQ, and QH (Table 1 and Fig. 4). Three to four lithological layers are generally visible in the geoelectric sections of Figs. 5 and 6: fresh bedrock (up to 79,707 Ωm), weathered basement (sandy-clay) (55 – 767 Ωm), clayey-sand (127 – 639 Ωm), and Topsoil (93 – 3053 Ωm), this is consistent with Onawola et al. 2021, Raji & Abdulkadi, 2020 and Raji et al. 2019 [35–37]. The high variation in the resistivity of the topsoil is due to its composition which ranges from clay, sandy clay, clayey sand and compacted laterite (Table 2). The dispersed layer resistivity was used to evaluate the subsoil's appropriateness for civil engineering construction projects since higher layer resistivity indicates more subsoil stability and competence. In civil engineering work, zones of low resistivity are characteristic weak points that cause foundation failure because they contribute to rock instability in a complicated geological environment. Although the wetness of these conductive zones makes them potential locations for groundwater development, they also lead to foundation failure in civil engineering projects.

The proposed lithologic interpretation model based on resistivity values adopted are: over 1000 Ohm-meter interpreted as bedrock, low subsurface resistivities values of less or equal to 90 Ohm-m, characterized by water-absorbing clayey layer, clay-rich material or weathered basement; conductive fractured basement/partially weathered with resistivities range between 200 and 850 Ohm-meter (Olayinka et al. 2004, Idornigie 2006, Oladapo et al. 2008, Ademila 2021) [38–41].

The overburden thickness of over 18 m was recorded at VES 5, 6, 8, 9, 10, 11, 14, 15, 16 and 18 locations in the northwestern and northcentral portion of the research area (Figs. 6 and 7).

4.2 Surface geophysics -Magnetic

As illustrated in Fig. 1, the ground magnetic data acquisition was conducted on the seven traverses, gathering a total of one hundred and forty (140) magnetic data points. The profiles were generated using Microsoft Excel software to plot the magnetic field intensity (nT) on the vertical axis against the distance on the horizontal axis as shown in Fig. 8, The magnetic field changes drastically in the southern part of the research site. The method clearly defined the contact between the basement rocks with the biotite schist at the northern portion and the granite gneiss at the southern portion of the research area (Fig. 9). The interpretation was based on the geology of the study area and the fact that the presence of a possible concentration of iron-bearing minerals in biotite schist usually exhibits higher magnetic susceptibility

Table 1 Outline of the geoelectric value and lithological distribution across the research area

VES NO	Layers Apparent Resistivity $\rho_1/\rho_2/\rho_3/\rho_4$ (Ohms-meter)	Layers Thickness $h_1/h_2/h_3/h_4$ (M)	Layers Depth $d_1/d_2/d_3/d_4$ (M)	Curve Type	Inferred Lithologies $L_1/L_2/L_3/L_4$
1	1914/398/158/903	0.6/1.9/3.2/∞	0.6/2.4/5.6/∞	QH	Dry lateritic topsoil/sand/sandy clay/fractured basement
2	218/872/79707	2.5/1.5/∞	2.5/4.1/∞	A	Topsoil/laterite/fresh basement
3	1418/173/336/837	0.9/1.2/5.4/∞	0.9/2/7.5/∞	QH	Lateritic Topsoil/clayey sand /sand/fractured basement
4	3053/175/2953/1317	0.8/2.3/5.6/∞	0.8/3.1/8.7/∞	QH	Compacted lateritic topsoil/sandy clay/fresh basement
5	1043/213/297/1414	1.1/1.5/26.4/∞	1.1/2.7/29.1/∞	QH	Dry lateritic topsoil/sandy clay/sand/ fresh basement
6	1981/263/371/1916	1/2.3/24.2/∞	1/3.3/27.5/∞	QH	Lateritic topsoil/clayey sand /sand/fresh basement
7	6339/235/120/10629	0.4/6.4/9.2/∞	0.4/6.8/16.1/∞	QH	Compacted lateritic topsoil/clayey sand/sandy clay/fresh basement
8	2357/328/83/12067	1/7.5/11.1/∞	1/8.5/19.6/∞	QH	Dry lateritic topsoil/sand/clay/fresh basement
9	878.9/127/160/2052.6	0.7/1/31.6/∞	0.7/1.6/33.3∞	QH	Lateritic topsoil/sandy clay/fresh basement
10	1872/185/87/17225	1.3/9/13.5/∞	1.3/10.3/23.8/∞	QH	Compacted lateritic topsoil/clayey sand/clay/fresh basement
11	1109/389/70/448	0.5/7.4/22.2/∞	0.5/7.9/30.1/∞	QH	Compacted lateritic topsoil/sand/clay/sand
12	1331/132/384/210	1.1/1.3/4.7/∞	1.1/2.5/7.2/∞	QH	Dry lateritic topsoil/sandy clay /sand/weathered basement
13	2150/206/156/80	0.5/4.3/6.3/∞	0.5/4.8/11/∞	QQ	Dry lateritic topsoil/clayey sand /sandy clay/weathered basement
14	1772/232/55/3881	0.8/12.4/13.3/∞	0.8/13.2/26.5/∞	QH	Dry lateritic topsoil/clayey sand/clay/fresh basement
15	1890/490/158/58486	1/7/10/∞	1/8/18/∞	QH	Compacted lateritic topsoil/sand/ sandy clay/fresh basement
16	814/3751/518/312	0.6/7.6/22.4/∞	0.6/8.3/30.6/∞	KQ	Topsoil/compacted lateritic sand /sand/fractured basement
17	469/365/170/156	1/6.7/12.5/∞	1/7.7/20.2/∞	QQ	Topsoil/sand/weathered basement
18	254/633/191/167	1.7/4.5/14.4/∞	1.7/6.2/20.6/∞	KH	Topsoil/sand/clayed sand/ weathered basement
19	217/218/534/546	0.5/2.1/7.5/∞	0.5/2.6/10.1/∞	AK	Topsoil/ clayey sand)/sand/fractured basement
20	144/217/325	0.5/6.9/∞	0.5/7.4/∞	A	Topsoil/clayey sand/ fractured basement
21	255/134/563/934	0.6/1.3/4.8/∞	0.6/1.6/6.6/∞	HA	Topsoil/clayey sand/sand/fractured basement
22	193/258/147/136	0.5/2.1/7.5/∞	0.5/2.6/10.1/∞	KA	Topsoil/clayey sand/ sandy clay /weathered basement
23	593/272/667/959	1.1/3/13.6/∞	1.1/4.1/17.7/∞	HA	Dry topsoil/clayey sand/sand /fractured basement
24	232/230/664/832	0.5/1.1/13.4/∞	0.5/1.7/15.1/∞	AA	Topsoil/clayey sand/sand/fractured basement
25	344/387/150/142	0.5/1.1/3.5/∞	0.5/1.7/5.2/∞	QA	Topsoil/sand/sandy clay/weathered basement
26	279/171/430/574	0.5/1.2/7.9/∞	0.5/1.7/9.6/∞	HA	Topsoil/clayey sand/sand/ fractured basement
27	392/678/304/245	0.5/3.5/8.8/∞	0.5/4.1/12.9/∞	KA	Topsoil/sand/sand/weathered basement
28	454/751/766	1.1/6.5/∞	1.1/7.6/∞	AA	Topsoil/sand/fractured basement
29	252/353/381/326	0.5/1/11/∞	0.5/1.5/12.5/∞	AQ	Topsoil/sand/fractured basement
30	313/452/131/115	0.3/1.8/10.6/∞	0.3/2.1/12.7/∞	QA	Topsoil/sand/sandy clay/weathered basement
31	93/243/238/148	0.3/2.2/7.3/∞	0.3/2.6/9.9/∞	AQ	Clayey topsoil/clayey sand /weathered basement
32	225/261/366/233	0.5/1/2.1/∞	0.5/1.5/3.6/∞	AQ	Topsoil/clayey sand/sand/weathered basement
33	325/367/247/242	0.5/2/10/∞	0.5/2.5/12.5/∞	QA	Topsoil/sand/clayey sand/ weathered basement/
34	188/183/463/584	0.6/0.6/11.3∞	0.6/1.2/12.4/∞	AA	Topsoil/clayey sand/sand/fractured basement
35	345/369/167/154	0.5/2.1/12.6/∞	0.5/2.6/15.1/∞	KQ	Topsoil/sand/clayey sand /weathered basement
36	240/262/513/530	0.4/0.6/9/∞	0.4/1/10.1/∞	AQ	Topsoil/clayey sand/sand/fractured Basement

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Table 1 (continued)

VES NO	Layers Apparent Resistivity $\rho_1/\rho_2/\rho_3/\rho_4$ (Ohms-meter)	Layers Thickness $h_1/h_2/h_3/h_4$ (M)	Layers Depth $d_1/d_2/d_3/d_4$ (M)	Curve Type	Inferred Lithologies $L_1/L_2/L_3/L_4$
37	244/303/149/144	0.3/2.4/7.5/∞	0.3/2.7/10.1/∞	KQ	Topsoil/sand/clayey sand/weathered basement
38	251/279/155/143	0.4/2.1/5/∞	0.4/2.5/7.5/∞	KQ	Topsoil/sand/sandy clay/weathered basement
39	345/443/251/226	0.8/3.9/13/∞	0.8/4.7/17.7/∞	KA	Topsoil/sand/clayey sand/weathered basement
40	208/361/193/162	1/4.1/8.5/∞	1/5.1/13.6/∞	KA	Topsoil/sand/clayey sand /weathered basement

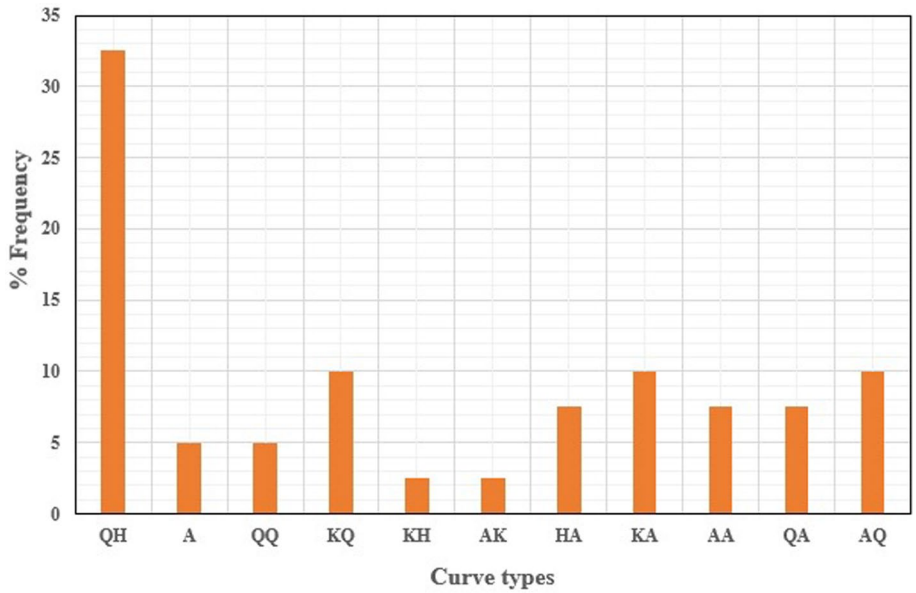


Fig. 4 Curve type percentage in the Research Area

and responds more noticeably in magnetic surveys, whereas, granite gneiss typically has lower magnetic susceptibility unless it has significant amounts of accessory magnetite. There is a general increase in the magnetic intensity (nT) from SSW to NNE with a minimum value of 33,303 nT, 33,358 nT, 33,375 nT, 33393nT, 33,381 nT, 33,378 nT and 33,344 nT on profiles 1–7 respectively. The maximum values of 33,354.2 nT, 33,372.6 nT, 33,399.2 nT, 33,399.9 nT, 33,392 nT, 33,389.5 nT, and 33,399.1 nT at shot points 380, 440, 380, 380, 320, 380, 340 m on profile 1–7 respectively (Fig. 8). The magnetic signals are indicative of possible basement contacts between two distinct granite gneiss and biotite schist rock types underneath the research region observed at short points 260, 220, 320, 180, 180, 220 and 80 m on profile lines 1–7 respectively (Figs. 8 and 9).

4.3 Geotechnical analysis

The geotechnical analysis were performed on the eight soil samples gathered from the study area. Additionally, in-situ testing was performed utilizing the dynamic cone penetration test (DCPT) at various sites within the study area. This comprehensive suite of analyses provided insights into both the mechanical and physical properties of the soil, aiding in the characterization and understanding of the subsurface conditions at the study site (Fig. 1).

4.3.1 Results of index tests

The natural moisture content of the soils in the research region is shown in Table 2, which ranges from 1.7% to 3.3% with a mean value of 2.5%. The natural water content of soils is a necessary physical characteristic in assessing their suitability for engineering construction projects. In the case of lateritic soils, natural moisture content can vary based on factors such as soil depth, drainage patterns, and annual rainfall. Cohesive soils change significantly in volume as a result of changes in moisture content. Olayemi [42] suggests that soils with a natural moisture content exceeding 15% are unsuitable for building foundations, while those with lower moisture content are considered suitable. Basement rock, characterized by numerous fissures, may contribute to soil wetness, potentially leading to interactions between foundation concrete and water. Such conditions are expected to significantly reduce soil shear strengths, increasing the likelihood of structural failures in above-ground buildings.

The density of coarse-grained soils directly influences their strength and stiffness characteristics. In the studied samples, bulk density ranged from 1.80 g/cm^3 to 1.99 g/cm^3, with a mean value of 1.91 g/cm^3 while the dry density ranged from 1.67 g/cm^3 to 1.86 g/cm^3, with a mean value of 1.76 g/cm^3. The high dry and bulk density recorded is indicative of more compact and dense soil, which typifies better load-bearing capacity. The soils in the study area are generally stronger, can support more significant loads and are more capable of resisting deformation under applied loads, allowing for more efficient and cost-effective shallow foundations. According to Bowles (2012), high

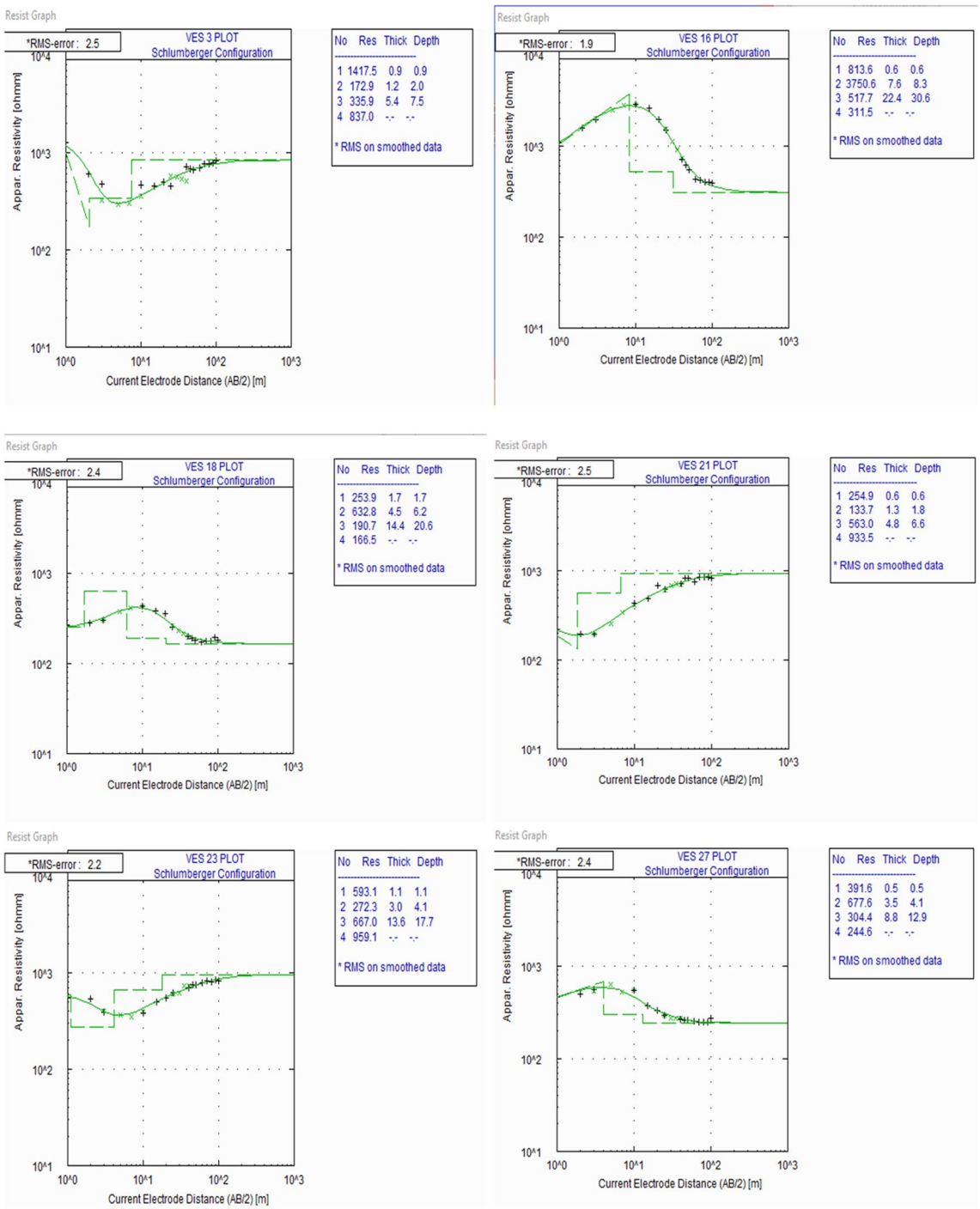


Fig. 5 Typical resistivity curves at the Research Site

bulk density is often desirable [43]. A range of 1.80 g/cm to 2.2 g/cm3 for sand and 2.0 g/cm3 to 2.4 g/cm3 for gravel may be considered for foundation projects.

The specific gravity of the examined soils, which ranged from 2.69 to 2.98 with an average of 2.84 are important parameters for determining the lateritic soils' mechanical strengths and in choosing the right engineering materials. The measured specific gravities of the soils under study, as reported by Jegede [44] align closely with typical values for lateritic soils found in Nigeria's basement complex. This consistency suggests true laterization in the investigated soils, as indicated by these specific gravity values.

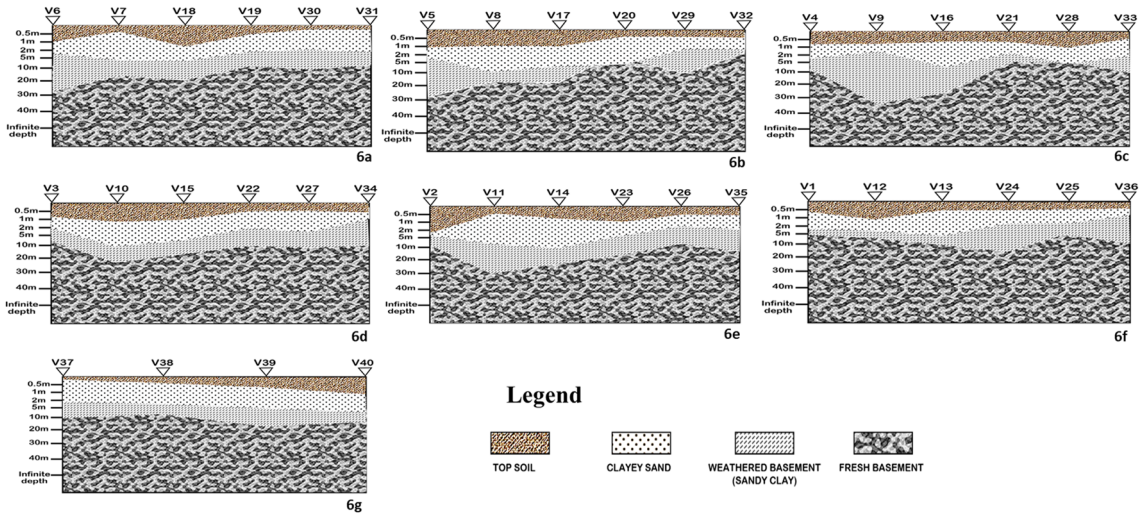


Fig. 6 Geo-electric section of the profiles 1–7

Table 2 Values of water content and other index properties

Soil Symbol	Water Content (%)	Bulk Density (g/cm^3)	Dry Density (g/cm^3)	Specific Gravity
G1	1.7	1.97	1.82	2.98
G2	2.1	1.92	1.76	2.86
G3	2.1	1.99	1.86	2.96
G4	3.3	1.83	1.62	2.69
G5	2.4	1.98	1.79	2.84
G6	2.8	1.95	1.77	2.79
G7	3.2	1.80	1.67	2.73
G8	2.2	1.86	1.81	2.87

4.3.2 Results of grain size analysis

The grain size distribution curves shown in Fig. 10 demonstrate the variance in particle sizes seen in the soils under study. Across all soils examined, sand dominates, with the percentage composition of gravel ranging from 9 to 35%, sand from 48 to 61%, silt from 6 to 12%, and clay from 11 to 20%. Based on the grain size fractions, the soils investigated are classified as clayey sand, as the sand content exceeds 50%, following BS (1990) [31]. Furthermore, the soils are categorized as SC, indicating they are well-graded, as determined by the grain size distribution analysis (Table 3).

The textural classification of soil, determined by the distribution of grain sizes (gravel, sand, silt, and clay), provides valuable insights into various soil properties such as strength, bearing capacity, compressibility, and sensitivity to frost action. Excessively fine-grained soils tend to be more vulnerable to changes in water content compared to coarse-grained soils [46].

According to BS (1990), FMWH (1997 and Olabode and Asiwaju-Bello (2018), soils with a percentage composition of fine-grained particles passing through a 0.075 mm sieve equal to or less than 35% can be considered suitable for construction materials, this includes foundations, buildings, airfields, and pavements [47, 48]. Given that the percentage of fine-grained particles in the examined soils ranged from 17 to 30%, they are deemed suitable for construction purposes.

4.3.3 Results of atterberg consistency limits

Table 4 provides an overview of the findings from the Atterberg limits test performed on the soil samples. The plastic limits were between 12.6% and 19.8%; with an average value of 16.0% the liquid limits between 22.3% and 34.1%, with an average value of 28.1% and the plasticity index between 9.7% and 14.3%, with an average value of 12.1%. According to British Standard [31], soils are classified based on their liquid limits: low plasticity (CL) refers to soils

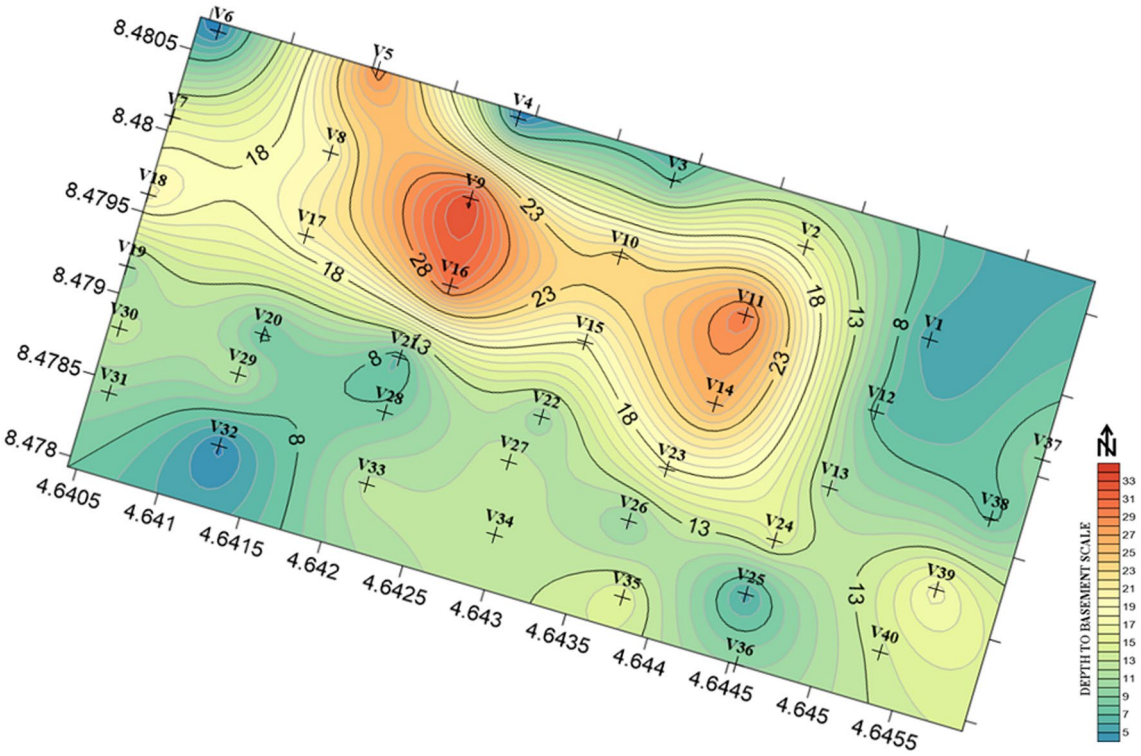


Fig. 7 Depth to Basement map

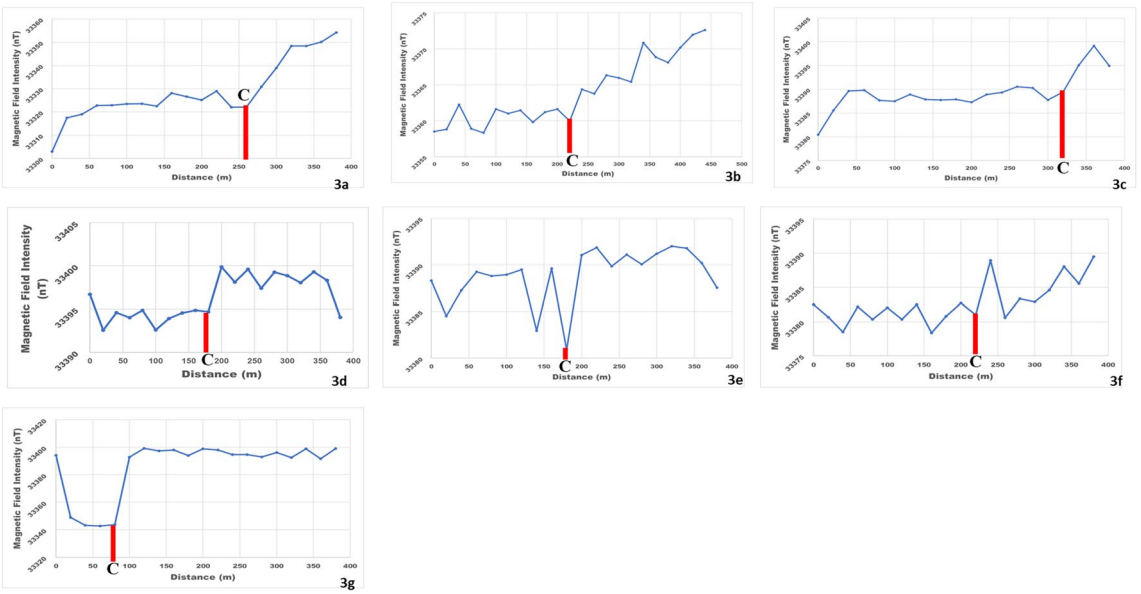


Fig. 8 SSW-NNE profiles of Total Magnetic Intensity Plot against distance with the proposed basement contact C

having a liquid limit of less than 35 per cent, 35% to 50% are intermediate plasticity (CI), 50% to 70% are high plasticity (CH), 70% to 90% are very high plasticity (CV) and greater than 90% are extremely high plasticity (CE). Based on the liquid limits observed in the studied soils, it is suggested that they exhibit medium to low plasticity (see Fig. 11). All of the soil samples were found to plot above the A-line, indicating the presence of inorganic clay, when the values of the soil samples were plotted as liquid limit versus the plasticity index on the plasticity chart (see Fig. 12). Soils with

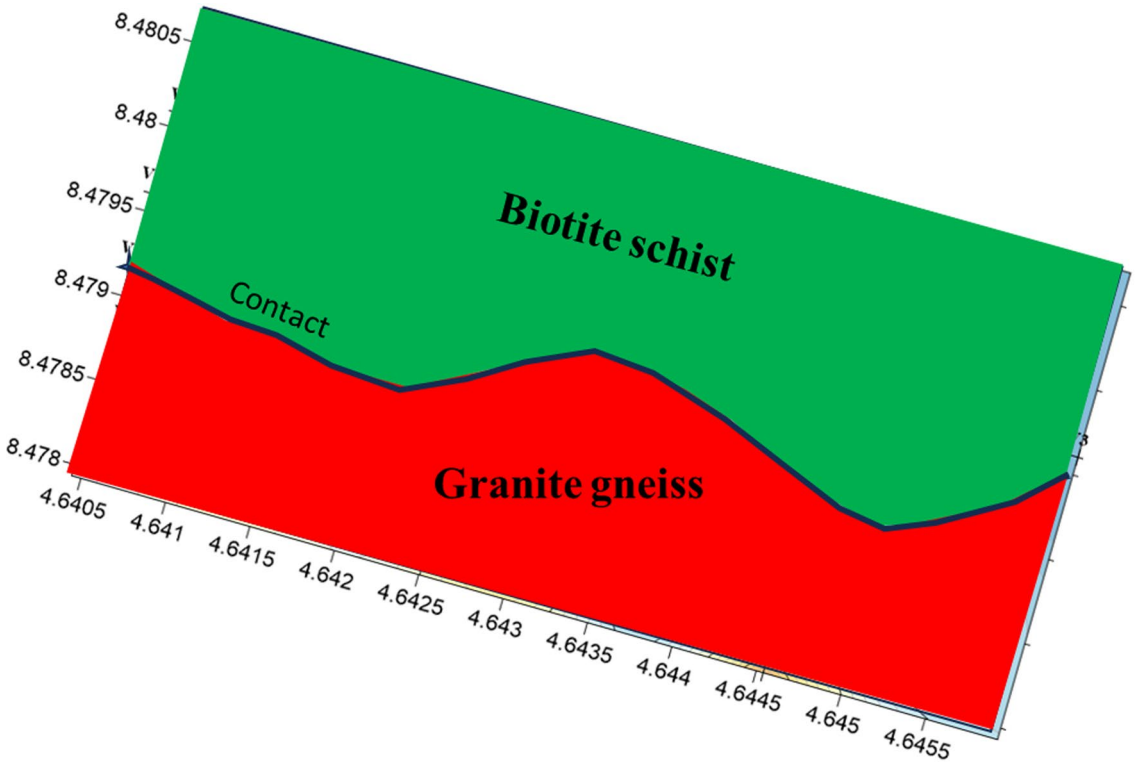


Fig. 9 Basement contact map of the study area delineated using magnetic method

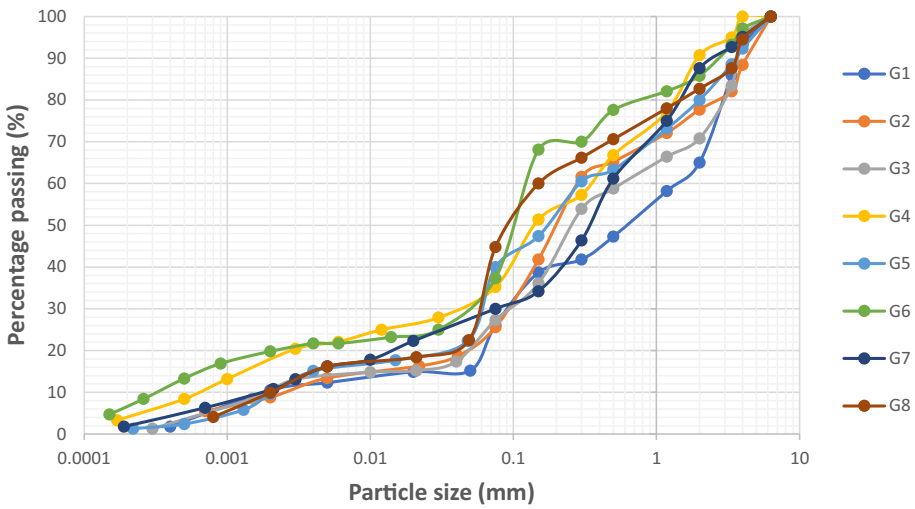


Fig. 10 Grain size distribution curves for the investigated soils

very high liquid limits have a greater tendency for shrinkage and poor volume stability, which can lead to structural failures [51]. The bearing capacity of soils tends to increase as the plasticity index decreases [52]. Among soil indices, the plasticity index is particularly useful and is critical when selecting soil for construction materials. Excessive plasticity often leads to fluctuations in soil behaviour due to plastic flow when subjected to loads [53]. The soils under investigation can be categorized as having medium to low swelling potential (Fig. 13) based on the classification by FMWH (1997) and BS (1990) [31, 47], which states that soils with a liquid limit of more than 50% and a plasticity index of more than 25% are problematic. However, according to Parkash and Jain [54], soils with a plasticity index greater than 17% contain highly plastic cohesive clays. Therefore, soils from the study area exhibit low plasticity, low compressibility, high shear strength, and bearing capacity.

Table 3 Grain size distribution of the investigated soils

Soil Symbol	(%) Grain Size Distribution					Soil Group (BS, 1990)	Soil Description (BS, 1990)
	Gravel	Sand	Silt	Clay	Fines		
G1	35	48	6	11	17	Clayey Sand	SC
G2	22	57	9	12	21	Clayey Sand	SC
G3	29	52	6	13	19	Clayey Sand	SC
G4	9	61	12	18	30	Clayey Sand	SC
G5	20	54	10	16	26	Clayey Sand	SC
G6	14	58	8	20	30	Clayey Sand	SC
G7	12	60	11	17	28	Clayey Sand	SC
G8	17	59	10	14	24	Clayey Sand	SC

Table 4 Values of Atterberg consistency limits of the investigated soils

Soil Symbol	Liquid Limit (%)	Plastic limit (%)	Plasticity Index (%)
G1	22.3	12.6	9.7
G2	24.5	13.9	10.6
G3	23.7	13.6	10.1
G4	34.1	19.8	14.3
G5	29.3	16.3	12.9
G6	32.5	18.9	13.6
G7	30.6	17.5	13.1
G8	27.8	15.6	12.2
Mean	28.1	16.0	12.1

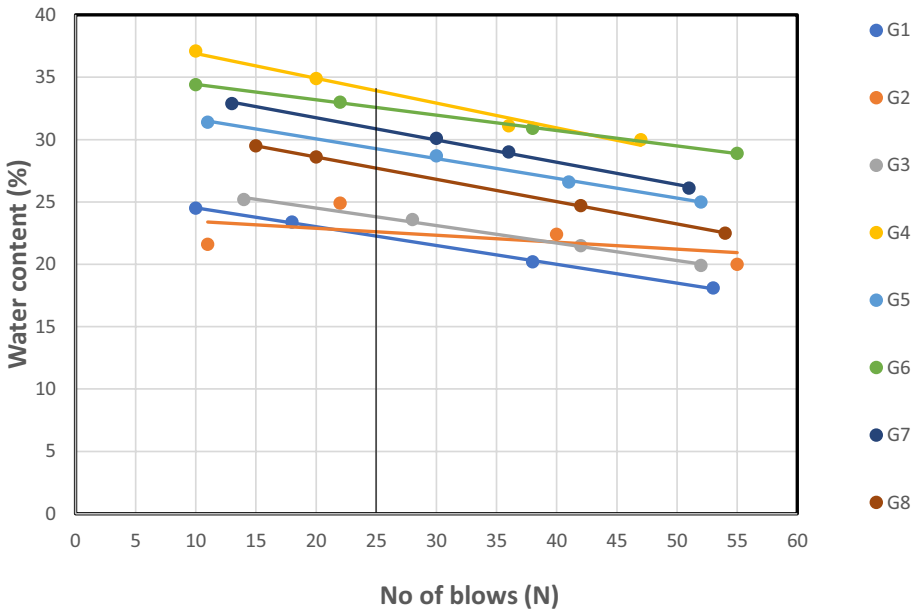


Fig. 11 Liquid limit graphs for the studied soils

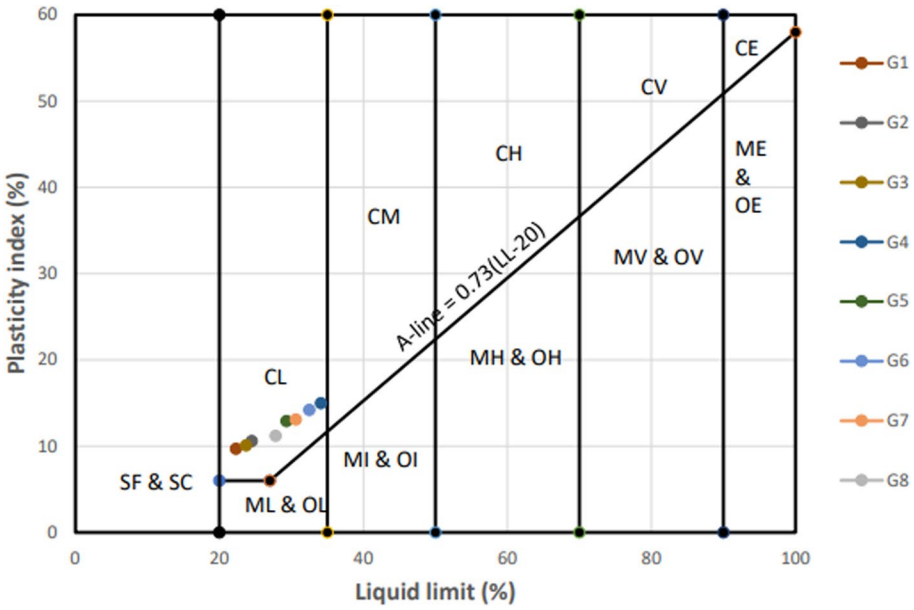


Fig. 12 Classification of the soil samples based on plasticity chart [31]

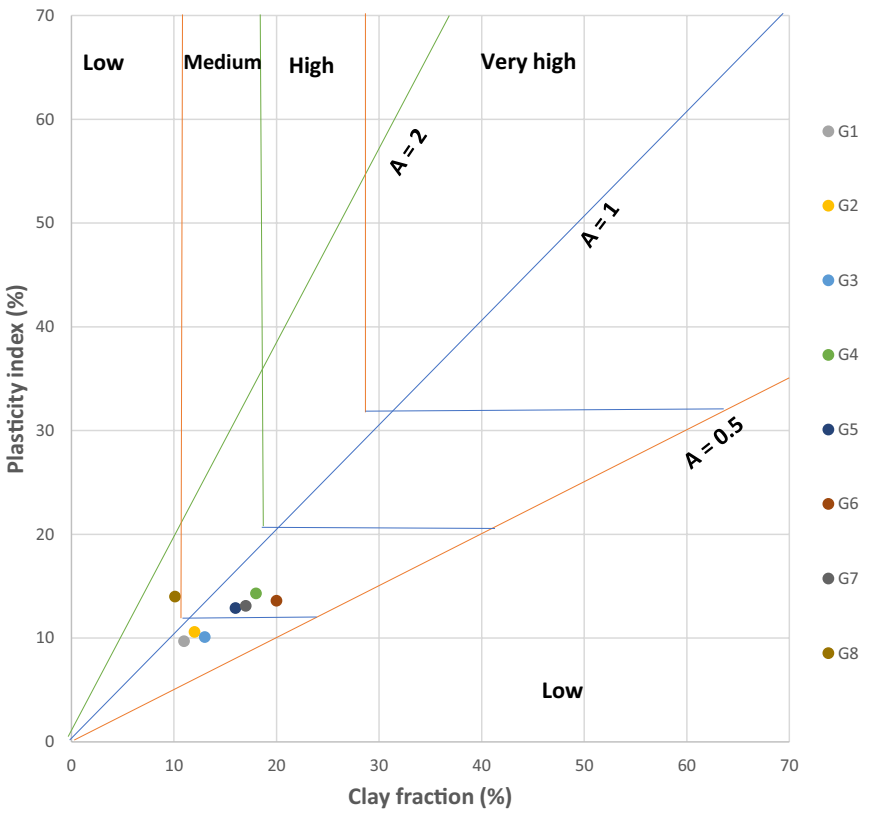


Fig. 13 Classification of the soil samples based on expansivity [56]

4.3.4 Results of compaction test

Compaction is a mechanical process aimed at rearranging and packing soil particles to reduce porosity and enhance dry density [57]. According to Jegede [44], compaction offers insights into how particles stabilize and densify under

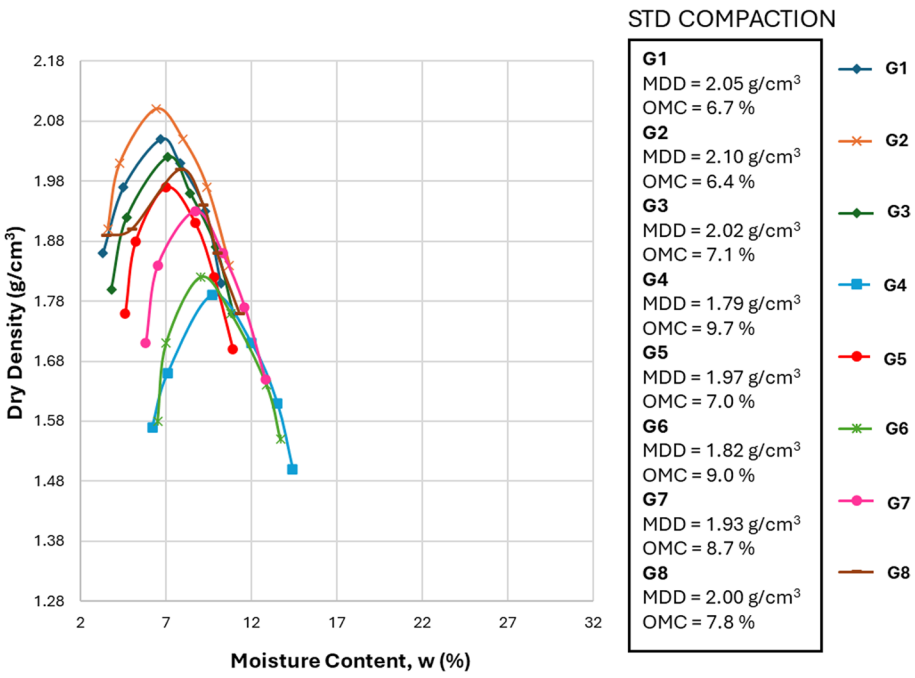


Fig. 14 The soil's standard proctor compaction curves

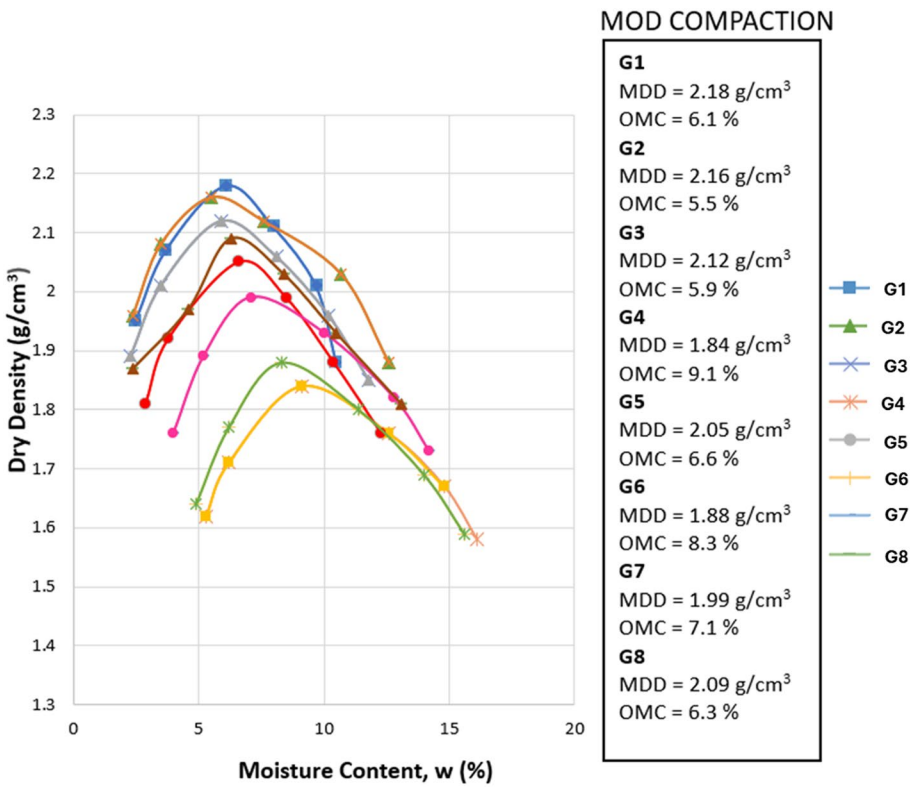


Fig. 15 The soils' modified proctor compaction curves

different energy conditions. When soil is compacted to its maximum dry density, optimum moisture content is achieved by applying a standardized compaction effort. The foundation's strength is determined by the soil's bearing capability [58]. The results of the compaction test are summarized in the dry density and moisture content curves (Figs. 14 and 15). The maximum dry densities (MDD) of the soil under investigation varied from 1.79 g/cm^3 to 2.10 g/cm^3, with a mean

Table 5 Foundation suitability of soils using a range of maximum dry density values [44]

Maximum Dry Density (g/cm^3)	Remark for Foundation	Sample Soils in the range
2.0826	Excellent	G1, G2, G3 and G8
1.9224 – 2.0826	Good	G5 and G7
1.7622 – 1.9224	Fair	G4 and G6
1.6020 – 1.7622	Poor	
1.1214 – 1.6020	Very Poor	

Table 6 Results of dynamic cone penetration for the studied soil samples

	Calculated Ultimate Bearing Capacity, Q (KN/m^2)									
Depth (m)	G1	G2	G3	G4	G5	G6	G7	G8	G9	G10
0.1	1122.5	269.4	942.9	493.9	583.7	134.7	269.4	89.8	134.7	89.8
0.2	2065.4	718.4	1526.6	628.6	1167.4	359.2	314.3	1077.6	673.5	538.8
0.3	2110.3	1796	2020.5	493.9	1347	628.6	493.9	1616.4	1212.3	1616.4
0.4				493.9	2065.4	2110.3	2110.3	2065.4	2110.3	2110.3
0.5				763.3						
0.6				2020.5						

value of 1.96 g/cm^3 and 1.84 g/cm^3 to 2.18 g/cm^3, with a mean value of 2.04 g/cm^3 for both standard and modified Proctor energies, respectively.

For foundation soils, [47] recommends a maximum dry density exceeding 1.7 g/cm3 and an ideal optimum moisture content of less than 18%. It is noted that each soil sample in the study area exhibits optimum moisture contents below and dry densities above the recommended limits (Table 5). However, upon a comparison of the test results with Wood's (1937) hypothesis [59], it is observed that three soil samples are classified as having excellent foundation materials, and two each are categorized as good and fair foundation materials respectively (Table 5). The soils under investigation exhibit excellent to fair dry densities at relatively low optimum moisture contents due to a high amount of coarse-grained soils with a small amount of fines. This composition fills the void spaces, thereby increasing stability and bearing capacity [44].

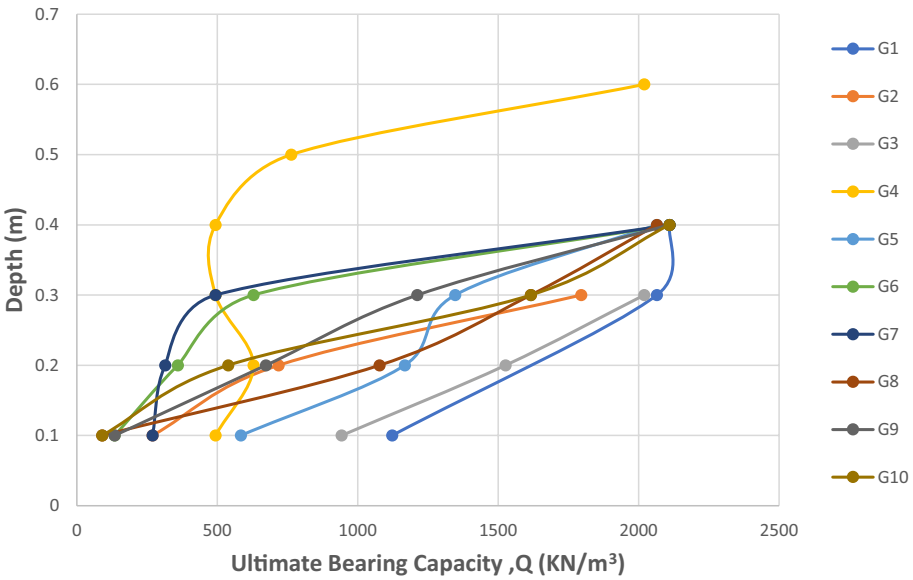


Fig. 16 Graph of ultimate bearing capacity against depths

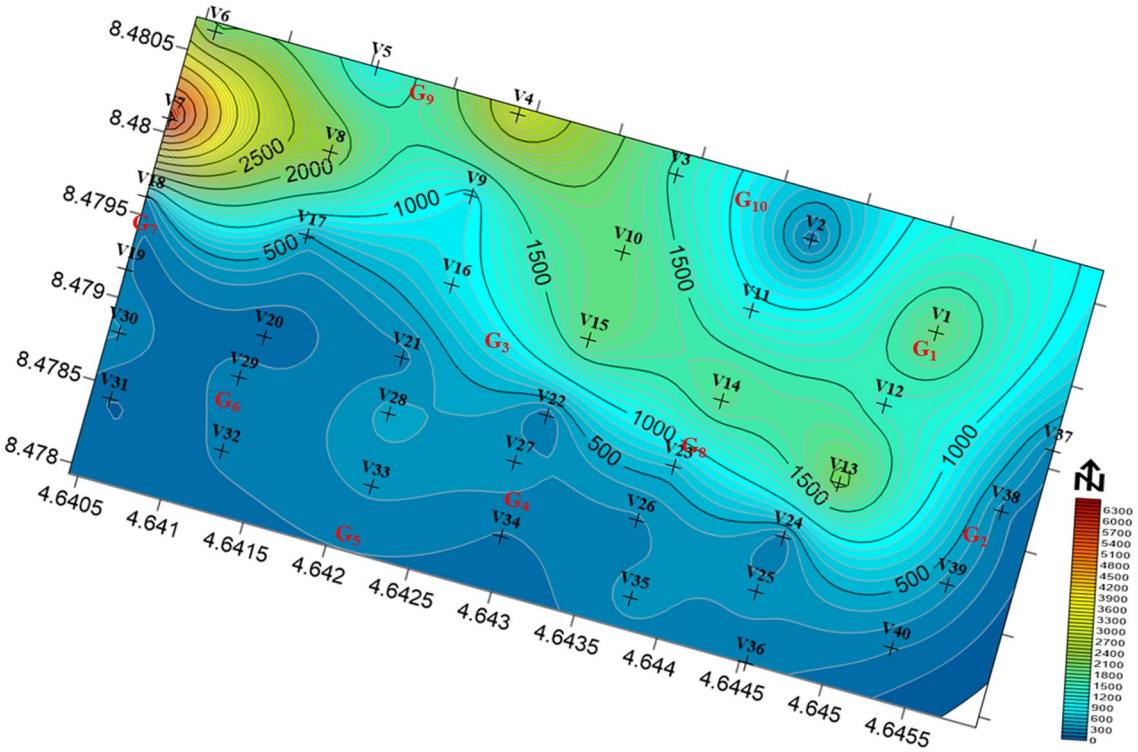


Fig. 17 Topsoil resistivity map of the study area

4.3.5 Result of dynamic cone penetration

The ultimate bearing capacity at various depths of soil penetration was calculated at 10 different locations (G1 – G10) as seen in Fig. 1 using the method proposed by Ampadu [34]. Table 6 displays the findings from the dynamic cone penetration testing. For the soils under study, there is a general increase in the calculated DCPT with depth at all the locations. The ultimate bearing capacity ranges from 1122.5 KN/m^2 to 2110.3 KN/m^2, 269.4 KN/m^2 to 1796 KN/m^2, 942.9 KN/m^2 to 2020.5 KN/m^2, 493.9 KN/m^2 to 2020.5 KN/m^2, 583.7 KN/m^2 to 2065.4 KN/m^2, 134.7 KN/m^2 to 2110.3 KN/m^2, 269.4 KN/m^2 to 2110.3 KN/m^2, 89.8 KN/m^2 to 2065.4 KN/m^2, 134.7 KN/m^2 to 2110.3 KN/m^2, and 89.8 KN/m^2 to 2110.3 KN/m^2 at G1 to G10 respectively (Table 6 and Fig. 16). When selecting the depth of a foundation, the primary consideration is ensuring that it is placed at a depth where the soil's bearing capacity can adequately support the load without experiencing excessive settlement or shear failure. This depth and bearing capacity are determined using in-situ techniques, as demonstrated in this study. The chosen depth and bearing capacity serve as guidelines for determining the minimum depth of the foundation.

4.3.5.1 Depth 0.1—0.2 m (100 −200 mm) The calculated bearing capacity (BC) from the dynamic cone penetrometer test (DCPT) of the soils at 0.1 m deep within the study area ranges from 89.8 kN/m^2 at both G8 and G10 locations to 1122.5 kN/m^2 at G1 position. With the minimum recommended value of 150 kN/m^2 to 350 kN/m^2 [32, 60], G6, G8, G9 and G10 locations will likely experience failures at this depth, it is therefore recommended that the localities are not suitable for a foundation at 0.1 m deep. However, suppose a foundation is proposed at these locations. Under such circumstances, excavating the foundation lines beneath the clay and silty clay units is necessary, subsequently laying concrete bases or the use of pilling. The bearing capacity of the soils at 0.2 m deep ranges from 314.3 kN/m^2 at G7 to 2065.4 at G1. The BC at all the locations is above the threshold as suggested by Ibrahim et al. [32], except G7 with a bearing capacity of 314.3 kN/m^2. This is an indication that the research area at 0.2 m deep is suitable for foundation laying except at locality G7. However, we recommended the same measure as earlier suggested for BC at 0.1 m if it is inevitable that a structure be erected at the G7 location.

4.3.5.2 Depth 0.3—0.6 m (300—600 mm) At the depth range of 300 – 600 mm, the soils' dynamic cone penetrometer test (DCPT) results for BC recorded a minimum value of 493.9 kN/m^2 at two locations G4 and G7 while the maximum recorded value of 2110.3 kN/m^2 at G1 (0.3 m deep), G6, G7, G9 and G10 at 0.4 m deep (Table 6 and Fig. 17). However, the BC at all the locations is above the required threshold, consequently, the site is good for the foundation.

A high resistivity zone is an indication of dense, compact, lateritic materials and this is observed at the northern portion of the study site except at the VES 2 location (Fig. 17). This zone is underlain by the biotite schist with topsoil resistivity ranging from 300 Ωm to 6300 Ωm. The contour value 700 Ωm in Fig. 17 mimics the basement contact that separates the two basement rocks. The low topsoil resistivity is observed in the southern and southeastern portions of the study area. However, high resistivity zones often correlate with higher bearing capacity and this is demonstrated at all the DCPT points except at G_4, G_5, G_6 and G_7 which are located within the low resistivity zone synonymous with soft, wet or clayey soils indicative of poor bearing capacity. However, low bearing capacity was observed at G_6, G_8, G_9 and G_{10} at 0.1 m but improves at a deeper depth The geotechnical analysis conducted at the study area imaged the subsurface with high resolution but was limited to a shallow level and sparse apart, this was complemented with the geophysical studies that imaged the subsurface to the basement rock revealing lithology of potential strata and basement contact that may undermine the integrity of the foundation. Although the resolution at the shallow level is coarse, it fills in the gaps between the geotechnical data points.

5 Conclusion

One of the most important stages in the construction of any building is the foundation design. It transfers the building's weight to the ground and keeps the structure together. In Nigeria, ground conditions should be taken into consideration while designing a building's foundation this will reduce the menace of building collapse. The geophysical methods deployed in the study area revealed the subsurface information which is critical for property development. The land magnetic method delineated a basement contact zone (geological discontinuity) that separates biotite schist in the north from granite gneiss in the south. The hydrostatic pressures on both sides of the rock, the kind and thickness of the fill material, and the breadth of the contact zone, however, were not ascertained but can be inimical to the success of the foundation if not properly handled. We hereby advise the builder to avoid placing the base of the structures across the contact zone.

Interpretation of the electrical resistivity profiles revealed three to four lithological layers in the research area. The resistivity values of the top layer of soil range from 93 to 3053 Ωm, with a depth of 0.3 to 2.5 m. The high topsoil resistivity zone was interpreted as dense and compacted laterite which correlates with high bearing capacity with some exceptions.

From the eight samples collected within the research area, the six parameters measured during the laboratory analysis include natural moisture content, specific gravity, bulk and dry densities, grain size analysis, Atterberg consistency limits, and compaction test. Two additional locations were subjected to dynamic penetrometer tests. The analysis of the results showed the following conclusions. First of all, Atterberg limits test findings revealed that the soils from the research area exhibit low plasticity, low compressibility, high shear strength, and bearing capacity, which indicate a potential for engineering construction work. Secondly, according to Wagner [49], the soils' grain sizes, as determined by the grain size analysis, exhibit clayey sand and gravelly sand. The same samples were also characterised as SC according to BS (1990) [31] categorization. This research area is ranked as suitable for development, as evidenced by the fine-grained particle composition percentage of 17–30%, which is lower than the 35% suggested by Olabode and Asiwaju-Bello [48]. Moreover, the study area is classified as having excellent to fair foundation materials with the majority of the samples falling within good foundation materials, according to the results of the Proctor compactions tests, both standard and modified. Lastly, in all locations within the research area, there exists a zone of extremely high bearing capacity at depths of 0.3–0.6 m. However, since bearing capacity tends to increase with depth, future foundations in the research area should be constructed at depths greater than 0.6 m. This ensures a more secure and stable foundation capable of withstanding the applied loads effectively.

In addition, the stability of the structure in the study region depends on the existence of clays located in the upper 2.5 m of the subsurface. Clay soils can take in water, hold it in their intricate chain structures, and release it gradually as pressure builds up. This further suggests that any construction constructed on this kind of soil material rests on material that is prone to swelling and eventually shrinks.

In light of the findings from the analysis of the data obtained from the field, the following were proposed:

i. Digging of the soil to a depth where it is sufficiently consolidated and competent to support any type of construction.
ii. The use of a shallow foundation type, such as a raft foundation in areas of lower competence and strip footing in competent areas with large bearing capacities.
iii. Since there is no near-surface geologic structure in the study region, high-rise structures are permitted. However, laying of foundation across the contact zone should be avoided and attention must be taken when selecting the foundation's materials.
iv. Deep foundations should be used at a suitable depth, particularly along the wet lateritic layer.

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Author contributions GEO designed and managed the project, all the authors participated in the desk work planning and Field acquisitions. AKO and TAI conducted the laboratory analysis of samples and interpretation of the result (geotechnical), SBA processed the resistivity data, and SBA and TAI processed the magnetic data. All the authors contributed to the interpretation, reading and approval of the final manuscript.

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Data availability All of the data used in this work is accessible from the authors and can be provided upon request.

Declarations

Consent for publication All of the researchers' and workers' consent was sought out, and they consented to be published.

Competing interests The authors declare no competing interests.

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