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Soil Bearing Parameters and Settlement Prediction Using CPT Data in Southwestern Nigeria

Olumuyiwa Olusola Falowo[1*], Abayomi Solomon Daramola[2], William Kunle Olabisi[2]

[1]*Department of Applied Geology, Federal University of Technology Akure, 340252, Ondo State, Nigeria*
[2]*Department of Civil Engineering Technology, Rufus Giwa Polytechnic Owo, 341104, Ondo State, Nigeria*

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ABSTRACT

The study was aimed at deriving important soil parameters relevant to civil engineering construction in schistose quartzite/quartzite dominated soil of RUGIPO, Ondo State, Nigeria, using cone penetration test (CPT). Key engineering parameters' findings revealed a high degree of overlapping values within the two geological formations. However, both soils are very dense, stiff/hard soil, with a high dominance of coarse particles (sand) over the finer soil matrix (silt). Soil from quartzite has better bearing pressure (avg. 405 kN m^{-2}) than schistose quartzite (324 kN m^{-2}), even above the regional average of 346 kN m^{-2}. The total settlement of the soils (avg. 26 mm), for structural loads of 100–200 kN and foundation width of 1–2 m, gave values less than permissible limits for clay (100 mm) and sand (50 mm) dominated soil, with schistose and quartzite displaying an average of 27 and 24 mm. The CPT material index with a regional average of 2.40, with schistose quartzite and quartzite showing average values of 2.39 and 2.42, and average soil permeability of 6.19×10^{-7} m s^{-1} and 4.498×10^{-7} m s^{-1}, respectively. The soils have a mean CBR value of 38%, with the CBR of schistose quartzite (35%) less than that of quartzite (42%). The regression models of all the parameters generally gave positive correlation coefficients, especially CBR and M_o (0.6522), R_D and CBR (0.6536), S_u and q_c (0.9849). The practical implications of the result indicated that both soils are competent for construction (pavement, foundation, and embankment construction) in both geological formations.

1 INTRODUCTION

The lowest portion of a structure is its foundation, and its function is to distribute the structure's weight to the subsurface[1,2]. A well-designed foundation transfers the weight through the soil without overstressing the soil. Overstressing the soil can result in either excessive settlement or shear failure of the soil, both of which can cause damage to the structure[3,4]. Hence, all structures, such as buildings and

[1*] Corresponding author. *E-mail address*: oofalowo@futa.edu.ng
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embankments, require a foundation[5,6]. All foundations settle since nothing is absolutely rigid, but understandably, some settle more than others. The principles for the design of a foundation are that the settlements should be limited with respect to the imposed load so that the structure does not become damaged[7,8]. In civil engineering, foundations are shallow, deep, or piled. In general, the strength and stiffness of homogeneous soil increase with depth. And so, one advantage of a deep foundation and a pile is that they are founded in stronger and stiffer soil, where the tip of a pile often rests on very stiff and strong soil or rock[9–11]. Another advantage is that shear stresses between the soil and the sides of a deep foundation or a pile contribute to the load capacity, however, in a shallow foundation, the contribution of the side shear stresses is negligible[4,12]. If the foundation is rigid (like concrete), the settlement will be even, and the bearing stress will vary across the foundation. If, on the other hand, the foundation is flexible (like an earth embankment), the bearing stress will be uniform, but the settlement will vary[1].

Accordingly, as the foundation settles, the bearing pressure keeps rising as the foundation gets deeper[13,14]. Therefore, by applying a factor of safety (or a load factor) to the bearing capacity, the bearing pressure must be lowered to an acceptable level in order to keep the settlements within a certain allowable limit. In general, the bearing capacity, allowable bearing pressure, undrained immediate settlement, final consolidation settlement, and variation of settlement over time must be calculated for sustainable structures in order to guarantee that the foundation has a sufficient margin of safety to counter any failure[15–17]. Slope stability and total retaining wall stability may be calculated using the critical state strength with safety factors that account for the errors in determining the pore pressures and soil properties[1]. This method works well for soil structures where ultimate stability is the primary design goal; however, it is not appropriate for foundations or other structures when the primary design criterion is the extent of ground movements or settlements. There is no constant relationship between bearing capacity and permitted bearing pressure because the main issue is that, even for a given soil, the ratio of stiffness, which regulates ground motions, to strength, which regulates ultimate failure, is not constant[18]. There are three main types of soil settlement brought on by loads: immediate settlement, also known as elastic settlement, which occurs when dry soil and moist, saturated soils undergo elastic deformation without a change in moisture content[2,19]. Equations from the theory of elasticity are typically used to calculate immediate settlement. Primary consolidation settlement occurs when the water that fills the vacuum spaces in saturated cohesive soils is expelled, causing a volume change. Saturated cohesive soils exhibit secondary consolidation settling, which is caused by the soil fabric's plastic adjustment[7,10].

The phenomenon of structural collapse is becoming a recurrent thing in the country, which many reports had attributed to insufficient knowledge of the subsurface geology of the sites, inaccurate, poor design and construction processes/techniques[20–28]. However, the lack of geoinformation on the geology and soil in many engineering sites accounts for the majority of the collapsed structures[15,29]. Many structures, such as bridges, internal roads, culverts, lecture theatres, etc., in the study area had reportedly collapsed at one time or another due to excessive foundation pressure or overloading, settlement, and erosion. The results of these collapses had stressed the school management financially in the area of repair, rehabilitation, and reconstruction. Even though the management of the institution was taking proactive steps in rehabilitating the defaulted structures, it is pertinent to characterize the foundation soil in the study area and derive important engineering parameters/properties of the soil necessary for the successful design and construction of structures in the institution.

Thus, engineering site characterization is an important process of design and construction, especially in the area of determining the engineering nature and structural competence of soil and rocks, as well as all other aspects of the site such as drainage, elevation, topography, etc[30]. By taking cognizance of these factors, important decisions during design and construction of an engineered construction project can be taken, while early potential hazards can be identified and incorporated during design processes. Hence, this study aims to estimate and develop empirical correlation of soil parameters derived from direct cone penetration test in schistose quartzite/quartzite environment, for structural design of buildings and utilities[5].

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The objectives of the study are encapsulated in the determination of soil foundation parameters that will assist in the design of structures in the study area, using CPT. These parameters included bearing capacity, cone resistance and corresponding SPT-N value, undrained shear strength, relative density, unit weight, angle of friction, constrained modulus, material index, permeability, California Bearing Ratio (CBR), settlement, and subgrade reaction modulus[31,32]. These parameters are essential in structural design and construction. Therefore, the study will serve as a guiding light for every construction work in the study area. Thus, a number of things that affect a foundation's bearing capacity include: subsurface stratification; the subsurface's shear strength characteristics; location of the ground water table; environmental factors; building size and weight; depth of excavation; type of structure[33,34]. These are determined to prevent adverse environmental impact or structural failure, or prevention of post construction problems associated with poor civil engineering construction processes. Hence, it is important that the allowable bearing capacity at a given site be determined based on the findings of soil exploration at that site, instead of using past experience or assumption of foundation parameters that usually becomes problematic during construction.

The major factor for deployment of CPT in this study was because of its capability to offer the indispensable geotechnical data for the site-specific information on the subsurface conditions at the project site, including the geostratigraphy and evaluation of input parameters[35–37]. An alternative approach is the use of CPT results to provide direct assessments of bearing capacity and/or settlements[34,38,39]. The CPT, in contrast to other common in-situ tests, is simple, fast, relatively economical, and it supplies continuous records with depth. The results are interpretable on both empirical and analytical basis, and a variety of sensors can be incorporated with cone penetrometer34[34,37–39]. Evaluating bearing pressure from CPT data[38–39] is one of the earliest applications of CPT sounding and includes two main methods: direct and indirect. Direct CPT approach applies the measured values of cone bearing with some modifications regarding the influence of foundation width to the cone diameter ratio. Indirect CPT methods employ friction angle and undrained shear strength values estimated from CPT data based on bearing capacity and/or cavity expansion theories[35–37]. Thus, data from the CPT can be used directly in foundation design or the estimation of soil parameters. Many empirical correlations have been developed from CPT in-situ approaches, even though they may have low accuracy[11,40,41]. This error is usually compensated for by using large safety factors[23,29,30]. Laboratory testing, in contrast, may be able to produce more accurate estimates of shear strength if sampling and testing are done well, but it is costly and time consuming. The application of CPT results is usually a better alternative and is now used to a larger degree than laboratory testing.

2 METHODOLOGY

2.1 Study area

Rufus Giwa Polytechnic Owo, Ondo State is situated along kilometer 30 Akure – Benin highway, about 3 km from Emure town and 25 km away from Akungba town, in the northern part of the state. It is located approximately on the following coordinates in Universal Traverse Mercator (UTM) Northing 798,500 – 801,500 m and Easting 781,000 – 784,000 m (Fig. 1). The institution is readily accessible along Akure-Benin and Ikare-Owo highways. It is located within the tropical rain forest of Nigeria, with an annual temperature range of 24 to 28 °C and the mean annual rainfall is over 1,500 mm[42]. The yearly rainfall and other advantageous climatic and geologic influences guarantee sufficient groundwater recharge in the area.

The area is underlain by the basement complex rocks, consisting of high-grade metamorphic rocks, which comprises a variety of gneisses, migmatite, schists, and marbles. The gneisses and migmatite are ubiquitous and are intimately mixed that they are hardly separable on the field and dominated the western/northwestern part of the area (Fig. 2). The gneiss may be divided into two major types: the biotite gneiss and the banded gneiss. The banded gneiss consists of alternating bands of dark (melanocritic) and light coloured (leucocratic) band. The gneiss-migmatite-quartzite complex constitutes the basement into which other crystalline rocks are emplaced. Also noticeable in the area are quartz schist/schistose quartzite and quartzite[43]. The schist, which is also a metamorphic rock composed mainly of quartz formed by the

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action of heat and pressure on sandstone, the quartzite schist/schistose quartzite is a rock that splits into layers whose minerals have aligned themselves in one direction.

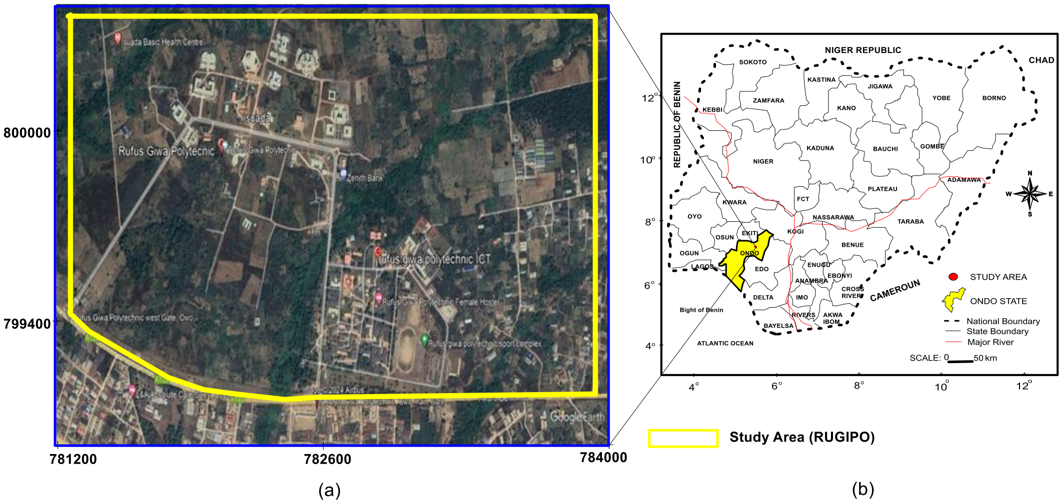

Fig. 1. Research location on (a) Google map (b) Ondo State and Nigeria.

The topography is slightly undulating with rounded low hills (found at low rise areas and sport complexes in central and eastern zones) and occasionally elongated ridges indicating the characteristic residue setting of a typical basement terrain, with an average height ranging between 20–50 m above sea level. The superficial deposit within the basement complex terrain varies in thickness from 2 m to more than 50 m and is mostly clayey loamy topsoil and clayey sand and sandy clay soil, which can be sandwiched by reddish brown lateritic hardpan (less than 3 m) soil in many places. The drainage is generally disjointed and limited in length and aerial extent, as a majority are found within the relatively lower elevation regions.

2.2 Data acquisition and analysis

Geotechnical site characterization is a vital first step towards the evaluation of subsurface conditions and determination of soil layering, geomaterial classification, and the evaluation of soil engineering parameters for the analysis and design of foundations, retaining walls, tunnels, excavations, embankments, and slope stability[14,16,19]. The study used in-situ test Dutch cone penetration tests at thirty three locations within the main campus of the institution (Fig. 2). The CPT and its upgraded versions (like piezocone, CPTu and SCPT) have widespread applications in an extensive range of soils[44,45]. Despite being mostly restricted to softer grounds, the CPT may now be conducted on stiff to extremely stiff soils and, in certain situations, soft rock, due to new massive pushing equipment and stronger cones[46–48]. The specialized mechanical friction-cone penetrometer was used to measure the cone and sleeve resistance in this study, capable of penetrating extremely stiff soil. The standard dimensions of the cone include a 60-degree tip angle and a 10 cm^2 (1.55 in^2) projected end area. The standard rate of penetration is 2 cm s^{-1}. It has advantages over some other in-situ testing methods, including being quick and enabling continuous profiling of the soil, providing repeatable and reliable data, being very economical and productive, and having a solid theoretical basis to support its interpretation[33,49,50]. For mechanical cones, tests were conducted in compliance with ASTM D 3441[50]. The end-bearing resistance, friction resistance, and friction ratio, which is calculated by dividing friction resistance by end-bearing resistance were plotted against depth using the penetrometer data (Appendix A1). The data collection/processing workflow sequence adopted for the study is shown in Fig. 3.

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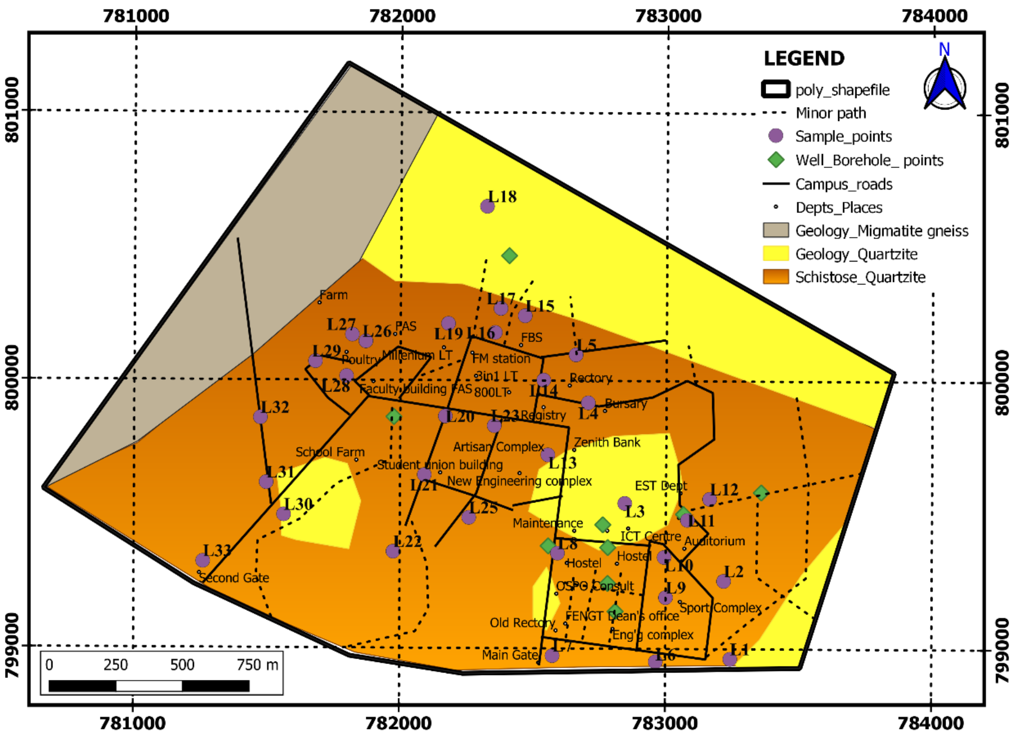

Fig. 2. Geological map of the campus showing predominant schistose quartzite formation and CPT locations (Source: field mapping exercise carried out by the authors).

For a variety of soil types, several semi-empirical equations have been established to obtain geotechnical data from the CPT. The reliability and usefulness of these correlations varies[13,51]. Characteristically, the cone resistance, q_c is high in sand and low in clay, and the friction ratio. The friction ratio, F_r. ($F_r = f_s / q_c$ where f_s is sleeve resistance and q_c is cone resistance) is low in sand and high in clay. Although the CPT can direct the soil's mechanical qualities, it is not anticipated to offer accurate soil predictions based on physical attributes such the distribution of grain sizes[52,53]. CPT data give a repeatable aggregate behaviour of the soil in the immediate vicinity of the probe. As a result, the term "SBT"[52–54] refers to the process of predicting soil type using CPT.

The ultimate bearing capacity, q_{ult} was determined using Equation 1 for clay/silt derived from schistose quartzite zones, according to Trofimenkov[55], assuming factor of safety of 3, irrespective of the dimension of the foundation structure. The equation is also suitable to strip footing on clays and sandy clays. The mechanical CPT with q_c and q_{ult} in kg cm^{-2}.

However, Equation 2 gives bearing capacity with reference to the expected load and width of footing. In this study, a width of 2 m and D_e was taken as the terminal depth of the CPT.

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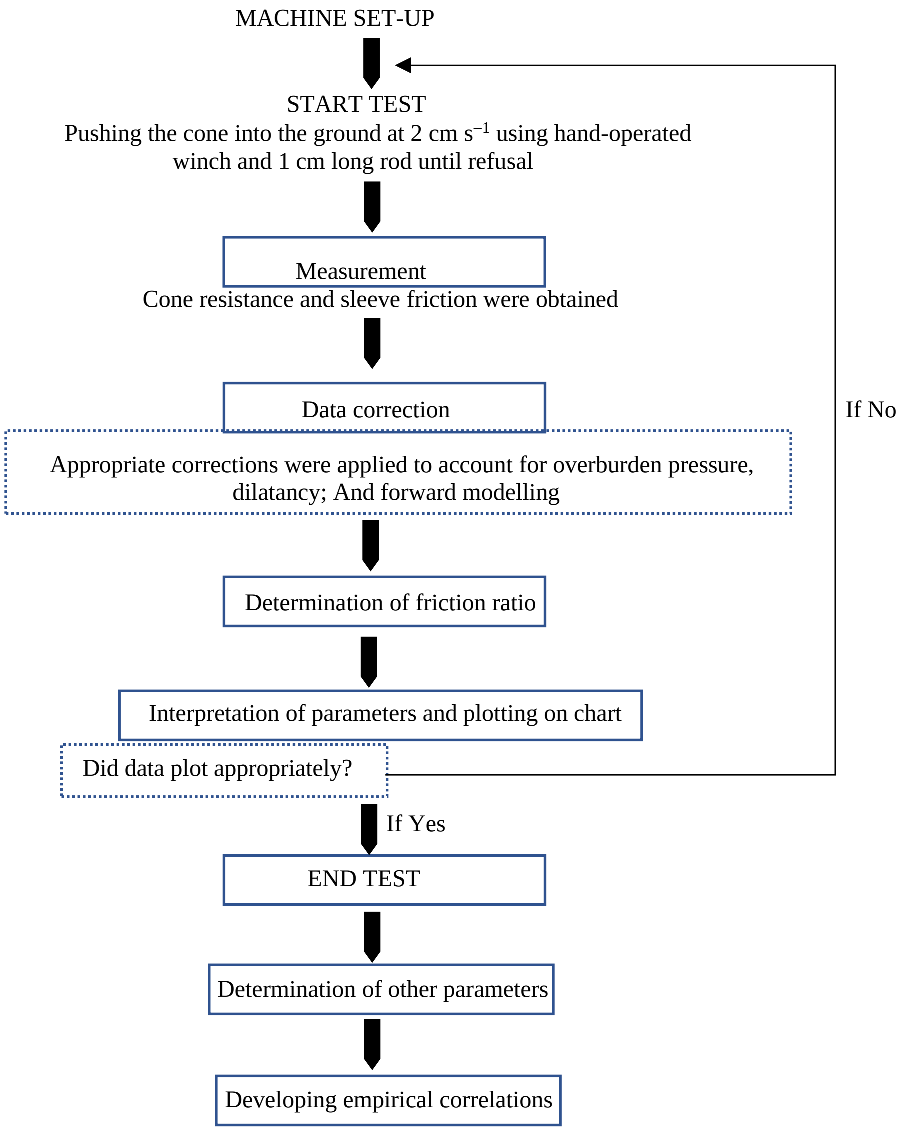

Fig. 3. CPT data collection and processing workflow sequence adopted for the study.

$$q_{ult} = \left(\frac{q_c}{33}\right)^{0.9} \tag{1}$$

$$q_{ult} = q_c(\mathrm{B}/12)\left(1+\frac{D_e}{\mathrm{B}}\right)C_w \tag{2}$$

where D_e = depth of embedment, however for silty sand it is reduce by 0.5,

C_w is correction factor.

The undrained shear strength, S_u, which often denotes or corresponds to an average soil strength[13] was calculated using Equation 3, where N_{kt} is the undrained shear strength constant and σ_v is the vertical effective stress. Typical values of N_{kt} vary from 10 to 18, with 14 being the default value. The N_{kt} tends to

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increase with increasing soil plasticity and decrease with increasing soil sensitivity, hence higher values of N_{kt} give more conservative estimates of S_u. Hence, for this study, the default value of 14 was used, where

$$S_u = \frac{q_c - \sigma_v}{N_{kt}} \tag{3}$$

The soil relative density was estimated using Lunne et al.[35] as shown in Equation 4, where Q_{tn} is the normalized cone resistance, C_{Dr} is the relative density constant and is usually taken as approximately 350. Relative Density (D_r) for coarse-grained soils is often used as an intermediate soil parameter. Soil relative density is only calculated for SBT_n zones 5, 6, 7, and 8. The value C_{Dr} = 350 is considered most appropriate for unaged quartz sands that are younger than 1,000 years[45]. C_{Dr} increases significantly when the age of the soil exceeds 10,000 years.

$$D_r = \sqrt{Q_{tn}/C_{Dr}} \tag{4}$$

The effective stress friction angle, $\emptyset'$, was determined using Equation 5, according to Mayne and Kulhawy[38,56], which is applicable for SBTn zones 5, 6, 7 and 8. The constrained Modulus (M_o) was estimated using Equation 6[45].

$$\emptyset' = 17.80 + 11 \log q_c \tag{5}$$

$$M_o = \alpha_M (q_c - \sigma_{vo}) \tag{6}$$

For $I_c > 2.2$ $\quad \alpha_M = q_t \quad (\text{if } q_c < 14, \text{ use } \alpha_M = 14)$

For $I_c < 2.2$ $\quad \alpha_M = 0.0188 \times 10^{0.55 I_c + 1.68}$

CPT material index (I_c) (Equations 7–10) can be used to identify soil type (as shown in Table 1), and the corresponding the characteristic values for estimating footing load-displacement response, where σ_{vo} is overburden stress and σ^I_{vo} is effective overburden stress.

$$I_c = \sqrt{(3.47 - \log q_{tn})^2 + (\log F_r + 1.22)^2} \tag{7}$$

$$\text{where } q_{tn} = (q_c - \sigma_{vo})/\sigma^I_{vo} \tag{8}$$

$$F_r = f_s/(q_c - \sigma_{vo}) \tag{9}$$

An approximate estimate of the soil coefficient of permeability, k, can be made from an estimate of soil behaviour type using the CPT SBT-I_c charts, while Equation 10 was used to express the average connection between soil permeability (k) and SBT-I_c. The estimates derived from the CPT-based SBT charts are shown in Table 1. Although these estimations are only approximations, they can serve as a reference for potential permeability variances when dissipation testing is not performed.

$$k = 10^{(0.952 - 3.04 I_c)} \text{ , when } 1.0 < I_c \leq 3.27 \tag{10}$$

The CBR in % was determined by using Equation 11 according to Eslami and Gholami[33] empirical equation involving CBR and cone resistance. The q_c is the average value taken from each of the locations, while the constrained modulus (M_o), expected settlement (S_e) under loading of 200 kN and footing width of 2 m, and modulus of subgrade reaction (K_s), were calculated using Equations 12 and 13, respectively, where ΔP is footing pressure, B is footing width.

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$$\text{CBR}(\%) = 0.454\, q_c \tag{11}$$

$$S_e = \frac{\Delta \text{PB}}{2q_{c(\text{average})}} \tag{12}$$

The coefficient of subgrade reaction (K_s) was estimated from the result of total settlement[7], as shown in Equation 13, where ΔP is net applied stress and S_e is settlement resulting from applied stress, p.

$$K_s = \frac{\Delta \text{P}}{S_e} \tag{13}$$

Table 1. Estimated soil permeability (k) based on the CPT SBT chart[53]

Soil classification	Zone number	Range of CPT index (I_c) values	k (m s^{-1})
Organic clay soils	2	$I_c > 3.22$	1×10^{-7}
Clays	3	$2.82 < I_c < 3.22$	3×10^{-10}
Silt mixtures	4	$2.54 < I_c < 2.82$	1×10^{-8}
Sand mixtures	5	$1.90 < I_c < 2.54$	1×10^{-6}
Sands	6	$1.25 < I_c < 1.90$	1×10^{-4}
Gravelly sands	7	$I_c < 1.25$	1×10^{-2}

3 RESULTS AND DISCUSSION

The results summary of the foundation parameters and total settlement values of schistose quartzite and quartzite derived soil from the CPT are shown in Table 2. The CPT was terminated at refusal depths of 0.75–2.0 m with an average (avg.) of 1.0 m. The obtained q_c and R_f values varied from 20–201 kg cm^{-2} (avg. 134 kg cm^{-2}), and 1.76–3.76 (avg. 2.42), respectively. These regional average values of q_c and R_f correlate with 129 kg cm^{-2} and 2.45 obtained by Falowo et al[23] in migmatitie/granite gneiss derived soils in Akoko Area of Ondo State at a depth of 1.2–1.9 m. Similar work by Farinde and Oni[34] in University of Ibadan, southwestern Nigeria, underlain by quartzite, banded gneiss and augen gneiss, recorded q_c and R_f values of 25–250 kg cm^{-2}, and 2.2–2.4, respectively. These values correspond to sand/gravel dominated soil. In addition, Imeokparia and Falowo[43] recorded similar values in Owo/Ose area of Ondo State, Nigeria underlain by migmatite, gneiss, and granite, with q_c values generally above 100 g cm^{-2} at 3 m depth. In comparison with previous works, showed that migmatite, granite/granite gneiss derived soil gives high q_c at shallower depth than what is recorded in this study; while relatively low (12 kg cm^{-2}) — medium (72 kg cm^{-2}) values of q_c were recorded at deep depth (greater than 5 m) in Lagos, as recorded by Oyedele and Okoh[26] in Magodo area; Olorode et al.[27] at Lagos State Polytechnic. The unit weight of the soil ranged from 18.34–19.45 kN m^{-3}. The ultimate bearing capacity (q_{ult}) varied from 222–499 kN m^{-3} (avg. 346 kN m^{-2}), with schistose quartzite and quartzite recording average values of 324 kN m^{-2} and 405 kN m^{-2}. However, for 2 m (width) foundation footing, with depth of emplacement taking as the refusal depth, the q_{ult} ranged between 1,842–6,566 kN m^{-2}, with quartzite (on the average) showing higher bearing capacity (4,229 kN m^{-2}) than schistose quartzite (2,936 kN m^{-2}). These are within the range recorded by Falowo[29,30] in Okitipupa (avg. 3,050 kN m^{-2}) between 1.0–4.0 m; in Owo/Ose[44] local government area (avg. 3,927 kN m^{-2}) at shallow depth of 0.68–0.80 m; while relatively lesser values were recorded in Ore town, southwestern Nigeria (2,550 kN m^{-2}) at depth of 0.6–3 m. The standard penetration test (SPT-N) values are an important parameter in geological/geotechnical engineering structural design[14]. It is the number of blows required to drive a sampler into the ground. It provides a rough measure of soil density. The SPT-N varied from 18 (schistose quartzite) 50 (quartzite), which signifies soil with high bearing

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pressure, corresponding to very stiff to hard soil (Table 3). The undrained shear strength values are between 573 to 1,404 kN m^{-2} with regional average of 573 kN m^{-2}, with schistose quartzite and quartzite showing values ranging from 573–1,259 kN m^{-2} (avg. 872 kN m^{-2}) and 697–1,404 kN m^{-2} (avg. 1,118 kN m^{-2}), respectively. The S_u is a soil property that measures the maximum stress a soil can bear before shearing or failing[38]. Hence, all the soil showed high shear resistance, as shown in the works of Nwankwoala and Warmate[25], and Kodicherla and Nandyala[41].

The relative density (R_D) of the soil ranged from 4.79 (schistose quartzite) to 7.5 (quartzite) with an average of 6.1. The angle of friction is the resistance a soil offers to motion, especially frictional movement. The angle of friction varied between 45.6° to 56.2° (avg. 53.7°), while 53.8° and 52.6° were recorded for the schistose quartzite and quartzite, respectively, with a high degree of overlapping values. The angle of friction of most soils ranged between 10–80°. The higher the value, the coarser the soil. Relatively lower values were recorded in the study conducted by Falowo et al[23] (26°) in the Akoko area with a shear strength of 96.5 kN m^{-2}. Hence, relating the SPT-N values and the friction as shown in Table 4, the soil is generally very dense, with a high dominance of coarse particles over the finer soil matrix.

Table 2. Summary of the geotechnical properties of the investigated soils

CPT-Location	East (m)	North (m)	Geology	R_{Df} (m)	q_c (kg cm^{-2})	R_f (%)	γ (kN m^{-3})	q_{ult} (kN m^{-2})	$q_{ult\text{ for B = 2 m}}$ (kN m^{-2})	SPT- N	S_U (KN m^{-2})	D_r
L1	783,243	798,963	SQ	1	128	2.01	19.33	332	3,136	32	895	5.99
L2	783,218	799,254	SQ	0.75	130	2.98	19.38	337	2,920	33	909	6.03
L3	782,845	799,543	Q	2	201	2.55	19.42	499	6,566	50	1,404	7.50
L4	782,706	799,918	SQ	1	128	2.01	19.25	332	3,136	32	895	5.99
L5	782,659	800,097	SQ	0.75	110	3.76	19.45	290	2,470	37	769	5.55
L6	782,963	798,952	SQ	0.75	125	2.87	18.86	325	2,807	31	874	5.92
L7	782,575	798,973	SQ	1	120	2.34	18.34	313	2,940	30	839	5.80
L8	782,593	799,356	Q	1.75	180	1.98	18.99	451	5,513	32	1,258	7.10
L9	783,000	799,192	SQ	0.75	128	3.72	18.52	332	2,875	43	895	5.99
L10	782,994	799,342	SQ	0.75	130	2.15	18.67	337	2,920	33	909	6.03
L11	783,081	799,481	Q	2	100	2.60	19.32	266	3,267	25	697	5.29
L12	783,164	799,560	SQ	2	100	2.71	19.22	266	3,267	25	697	5.29
L13	782,555	799,723	Q	0.75	125	2.88	18.96	325	2,807	31	874	5.92
L14	782,538	800,002	SQ	0.75	120	2.21	18.67	313	2,695	21	839	5.80
L15	782,467	800,242	SQ	1	124	1.88	18.69	323	3,038	31	867	5.89
L16	782,355	800,179	SQ	1	120	2.75	18.5	313	2,940	30	839	5.80
L17	782,376	800,269	SQ	1	120	2.24	19.21	313	2,940	30	839	5.80
L18	782,323	800,650	Q	0.75	140	1.86	18.92	360	3,144	35	979	6.26
L19	782,179	800,212	SQ	0.75	82	2.56	18.64	222	1,842	21	573	4.79
L20	782,169	799,866	SQ	1	180	2.02	18.95	451	4,410	45	1,259	7.10
L21	782,091	799,647	SQ	1	176	1.76	18.87	442	4,312	31	1,231	7.02
L22	781,975	799,360	SQ	0.75	130	2.26	18.73	337	2,920	33	909	6.03
L23	782,353	799,831	SQ	0.75	118	2.66	19.2	309	2,650	30	825	5.75
L24	788,275	799,577	SQ	0.75	125	2.22	18.34	325	2,807	31	874	5.92
L25	782,259	799,488	SQ	0.75	105	1.76	19.08	278	2,358	18	734	5.42
L26	781,869	800,145	SQ	0.75	120	2.11	18.55	313	2,695	30	839	5.80
L27	781,818	800,170	SQ	0.75	130	2.99	19.32	337	2,920	43	909	6.03
L28	781,797	800,015	SQ	0.75	118	2.44	18.88	309	2,650	30	825	5.75
L29	781,680	800,070	SQ	0.75	125	2.46	19.42	325	2,807	31	874	5.92
L30	781,563	799,497	Q	0.75	118	2.40	18.95	309	2,650	30	825	5.75
L31	781,497	799,617	Q	1	186	2.25	19.31	465	4,557	47	1,301	7.22
L32	781,474	799,859	Q	1	198	2.30	19.25	492	4,851	50	1,385	7.45
L33	781,260	799,322	Q	1	192	2.31	18.92	478	4,704	48	1,343	7.33

Note: SQ-Schistose quartzite, Q-quartzite

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Table 2. Continued

CPT-Location	Geology	ϕ (°)	M_0 (KN m^{-2})	I_C	K (m s^{-1})	USCS	CBR (%)	S_e (mm) PB = 400 kN m^{-1}	S_e (mm) PB = 100 kN m^{-1}	K_S (kN m^{-3})
L1	SQ	52.11	22,495	2.43	3.76965×10^{-7}	SM	34	27	7	7,350
L2	SQ	55.52	22,855	2.28	1.02744×10^{-6}	SM	47	20	5	10,094
L3	Q	48.94	35,308	2.46	2.99287×10^{-7}	SM	59	16	4	12,642
L4	SQ	52.15	22,495	2.43	3.78354×10^{-7}	SM	30	30	8	6,566
L5	SQ	54.67	19,335	2.21	1.67535×10^{-6}	SM-SC	34	27	7	7,350
L6	SQ	55.75	21,976	2.27	1.15978×10^{-6}	SM	40	23	6	8,624
L7	SQ	52.32	21,088	2.40	4.66781×10^{-7}	SM	31	30	8	6,664
L8	Q	49.43	31,622	2.50	2.32294×10^{-7}	S	38	24	6	8,232
L9	SQ	56.15	22,504	2.18	2.12668×10^{-6}	SM-SC	46	20	5	9,898
L10	SQ	56.09	22,856	2.41	4.29444×10^{-7}	SM	44	21	5	9,604
L11	Q	45.62	17,532	2.48	2.62692×10^{-7}	SM	21	44	11	4,508
L12	SQ	45.66	17,532	2.48	2.64798×10^{-7}	SM	17	54	13	3,724
L13	Q	55.66	21,976	2.27	1.15916×10^{-7}	SM	35	27	7	7,546
L14	SQ	55.71	21,096	2.37	5.47553×10^{-7}	S	35	26	7	7,644
L15	SQ	52.29	21,792	2.58	1.24277×10^{-7}	SM	30	31	8	6,468
L16	SQ	52.23	21,088	2.40	4.63121×10^{-7}	SM	27	34	9	5,880
L17	SQ	51.86	21,087	2.40	4.47051×10^{-7}	SM	30	31	8	6,370
L18	Q	56.24	24,616	2.44	3.4252×10^{-7}	SM	47	20	5	10,094
L19	SQ	53.91	14,408	2.39	4.8380×10^{-7}	SM	30	31	8	6,370
L20	SQ	53.93	31,648	2.43	3.64423×10^{-7}	SM	48	19	5	10,388
L21	SQ	53.87	30,944	2.55	1.626×10^{-7}	S	50	19	5	10,780
L22	SQ	56.04	22,856	2.41	4.295×10^{-7}	SM	39	24	6	8,428
L23	SQ	55.20	20,743	2.37	5.72638×10^{-7}	SM	33	28	7	7,056
L24	SQ	56.20	21,977	2.39	4.8365×10^{-7}	SM	37	25	6	8,036
L25	SQ	54.73	18,455	2.49	2.35681×10^{-7}	S	35	27	7	7,448
L26	SQ	55.81	21,096	2.37	5.47827×10^{-7}	SM	32	29	7	6,958
L27	SQ	55.57	22,855	2.28	1.02793×10^{-6}	SM-SC	39	23	6	8,526
L28	SQ	55.45	20,744	2.37	5.74915×10^{-7}	SM	33	28	7	7,154
L29	SQ	55.31	21,975	2.39	4.81463×10^{-7}	SM	35	26	7	7,644
L30	Q	55.40	20,744	2.37	5.74485×10^{-7}	SM	32	29	7	6,958
L31	Q	53.90	32,703	2.45	3.29486×10^{-7}	SM	45	21	5	9,702
L32	Q	54.23	34,815	2.37	5.44242×10^{-7}	SM	52	18	4	11,270
L33	Q	54.26	33,760	2.46	3.03931×10^{-7}	SM	53	17	4	11,466

Note: SQ-Schistose quartzite, Q-quartzite

The obtained constrained modulus (M_o) values are between 14,408 (schistose quartzite)–35,308 kPa (quartzite), with regional average of 23,605 kPa. The obtained M_O values fall within the threshold of coarse to fine sand dominated soil, ranging from 20,000–40,000 kPa[32]. The CPT material index (I_c) is used to identify and predict soil behaviour. A higher I_c value the better or competent is the soil for foundation, and embankment construction[18,19]. The I_c ranged between 2.18 to 2.58 (avg. 2.40), with schistose quartzite and quartzite showing average values of 2.39 and 2.42, respectively. Thus, using Table 1 interpretation, 94% of the soil fall within Zone 4, while 6% for Zone 5. This implies that the soils are sand-silt mixtures. The soil permeability (k) ranged from 1.24×10^{-7} to 2.13×10^{-6} m s^{-1} (avg. 5.73×10^{-6} m s^{-1}). The obtained values of k are in agreement with corresponding values of I_c — k relationship given in Table 1. However,

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the values of k for both soils are overlapping, but the schistose quartzite has relative higher k (avg. 6.19×10^{-7} m s^{-1}) than quartzite (avg. 4.498×10^{-7} m s^{-1}).

In order to determine the soil characteristics with respect to pavement construction, the CBR was estimated. The in-situ CBR gives the in-place CBR values (at field condition), which can accurately be used for pavement design thickness and overlays. The CBR measures the strength or competency of the subgrade for pavement construction[41,57]. The pervious the surface, the lesser the CBR value; the harder/impervious the soil, the higher the CBR value. The minimum standard value for subgrade soil is 5%, subbase 30%, and base 80%. The CBR of the soil ranged from 17–59% (avg. 38%), with schistose quartzite and quartzite having values varying from 17–50% (avg. 35%) and 21–59% (avg. 42%). This implies that the soil is good and competent, with high bearing capacity for use as subgrade and subbase layer in pavement and embankment construction[58]. Similar values from CPT were recorded in a study conducted by Arbianto et al[39] which recorded CBR of 38.6% in Surakarta and its surroundings. Thus, this range is very common in the basement complex of southwestern Nigeria, as reported by Olaonipekun and Tanimola[24] (45%), Falowo et al[23] (52%), Oyedele and Okoh[26] (36%), Falowo[29] (68%) in Okitipupa, Imeokparia and Falowo[43] (88%) in Idoani, and Falowo[30] (69%) in Ore. The total settlement of the soil (including elastic and consolidation) for the structural load of 200 KN and foundation width of 2 m, ranged from 16–54 mm (26 mm), with schistose and quartzite displaying an average of 27 and 24 mm. However, for the structural load of 100 kN and a foundation width of 1 m, which is common to many lecture theatres and administrative buildings in the institution, the expected settlement ranged between 4–13 mm (avg. 6.6 mm). Hence, the soil settlement is far below the permissible limit of 50 mm for clay and 25 mm for sand dominated soils. Thus, the design load should not be more than 200 kN, and if it becomes expedient to increase it, then the width of the footing must be increased accordingly. Consequently, an appropriate foundation design of structures in the institution should take into cognizance these values so that the settlement would not be excessive or highly differential. Nonetheless, related low settlement values, obtained in the basement complex of southwestern Nigeria, especially within Ondo State corroborated the values obtained in this study, as reported by Falowo et al[23] (0.78–21 mm) in Akoko area, Falowo[29] (10.7 mm) at Okitipupa, and Falowo[30] (43.9 mm at depth of 1.0 m) in Ore. The subgrade reaction (K_s), measures the stiffness of soil. It is a parameter that depends on characteristics or combination of soil foundation, loading intensity and distribution, and size/shape of the contact surface. It is a useful parameter in pavement and footing design[58,59]. The obtained K_s values varied from 3,724 (quartzite) to 12,642 kN m^{-3} (schistose quartzite) (avg. 8,104 kN m^{-3}). The mean values for schistose quartzite and quartzite are 7,709 and 9,041 kN m^{-3} respectively. The range of values of K_s is usually between 27,145–108,579 kN m^{-3}, with coarse soil (27,145–108,579 kN m^{-3}) and fine soil (6,786–40,717 kN m^{-3}). The regional range signifies a mixture of fine and coarse grains, possibly silt and sand. This assertion is rightly supported by the USCS of the soil, which is generally sand silt (SM), except samples 27, 59 (SM-SC), and 8, 13, 21, and 25 (S). The modulus of subgrade is used in pavement design, slab foundation, footing of bridges, etc. The subgrade reaction modulus is a combined soil-structural parameter, as its value depends on the ground stiffness and the size of the loaded element[60,61].

Table 3. Classification of soil based on SPT-N value[1]

SPT-N Value	Consistency
< 2	Very soft
3–4	Soft
5–8	Medium stiff
9–15	Stiff
16–30	Very stiff
> 30	Hard

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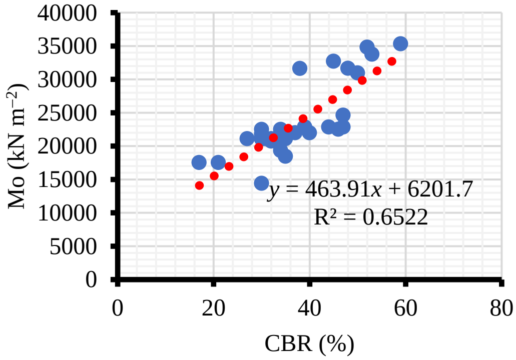
Table 4. Correlation of SPT-N value, relative density, and friction angle[1]

SPT-N Value	Soil packing	Friction angle (°)
4–10	Loose	30–45
10–30	Compact	35–40
30–50	Dense	40–45
> 50	Very Dense	> 45

Fig. 4 shows the relationship between some of the parameters; CBR and M_o, R_D and CBR, S_u and q_c, ∅ and q_c, CBR and I_c, q_c and I_c, K and $S_{e(100\ kN)}$, K and $S_{e(200\ kN)}$, S_u and M_o. All the relations given by their regression models or trends give the positive correlations (R^2) of 0.6522, 0.6536, 0.9849, 0.0009, 0.0162, 0.0687, 0.0349, 0.0297, respectively. The models for CBR and M_o, R_D and CBR, S_u and q_c, q_c and I_c, and S_u and M_o follow the same trend, showing direct proportionality, because as one increases, the other also increases; while ∅ and q_c, and CBR and I_c, show the same trend of inverse proportionality.



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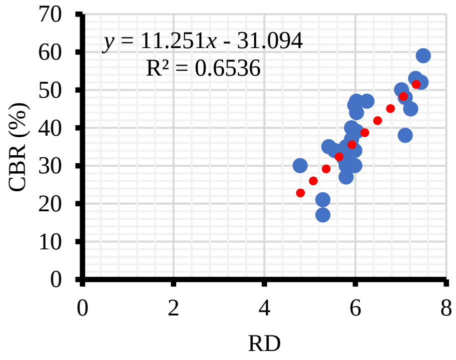

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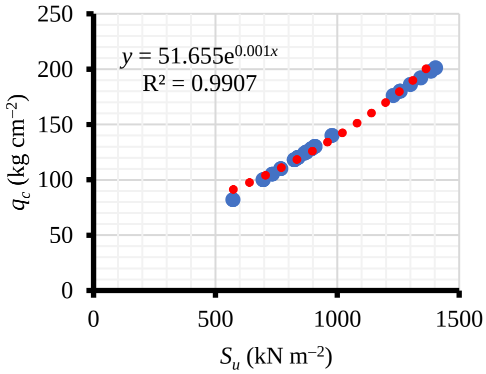

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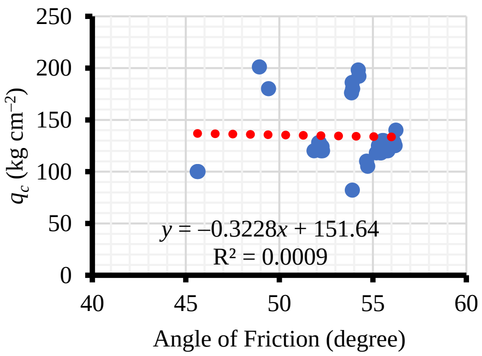

(d)

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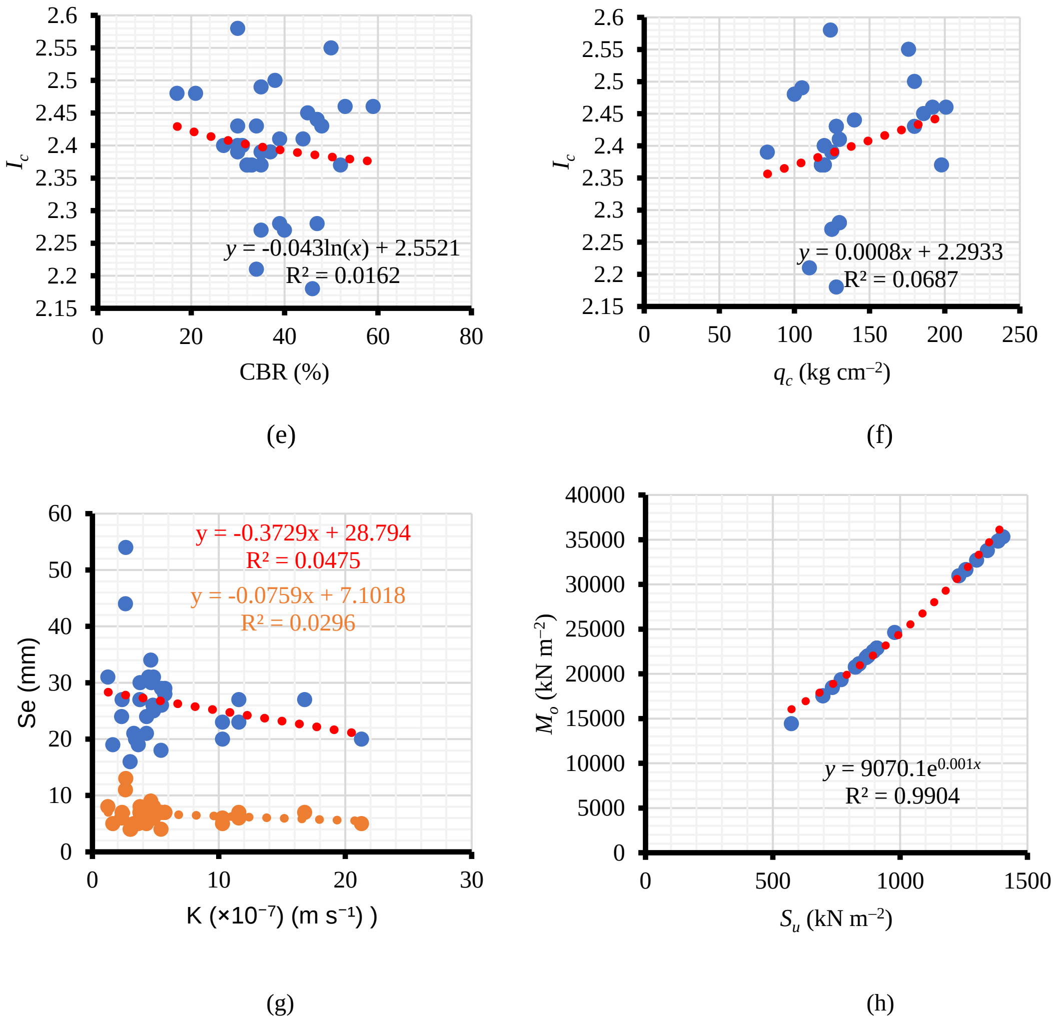

Fig. 4. Regression models for (a) CBR and M_o (b) R_D and CBR (c) S_u and q_c (d) Ø and q_c (e) CBR and I_c (f) q_c and I_c (g) K and S_e (h) S_u and M_o.

CBR and M_o, R_D and CBR, S_u and q_c, and S_u and M_o showed a very strong correlation coefficient signifying a strong association between these parameters, as they are controlled or influenced by the same phenomenon[62–64]. CBR is an important parameter in pavement design, so its strong positive correlation with M_o, R_D indicates that it will have a strong affinity with unit weight, compressibility, and permeability. Furthermore, the R_D also has a good association with CBR, however the magnitude and intensity of their association depends on geology, composition, degree of compaction, and texture of the soil. The relationship between K and $S_{e(100kN)}$, and K and $S_{e(200kN)}$ showed near horizontal trend at various loadings of 100 and 200 KN. However, both trend lines gave weak correlation coefficients, signifying that other factor(s) might still come to play in boosting this relationship. The permeability of embankment and foundation soil affects the rate of settlement. In low permeability soil material, increased loading will increase pore water pressure, which would invariably reduce the bearing pressure and stability of embankment structure. Hence, the drainage/dissipation rate and settlement profile of soil depend on the permeability of the foundation soil.

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4 CONCLUSIONS

The effective design and construction of civil engineering structures require the determination and understanding of soils' foundation parameters in terms of bearing capacity, settlement, and other important geotechnical indices. Hence, the evaluation of these parameters within the study area showed that:

i. Schistose quartzite and quartzite showed close and/or overlapping values, although, varying from very stiff to hard soil, very dense, with high domination of coarse particles over the finer soil matrix (generally of SM classification). The CPT material index (I_c) with a regional average of 2.40, suggested a good soil for foundation and embankment construction, with schistose quartzite and quartzite showing average values of 2.39 and 2.42, and soil permeability (avg. 6.19×10^{-7} m s^{-1}) than quartzite (avg. 4.498×10^{-7} m s^{-1}), respectively.

ii. The ultimate bearing capacity of the soils is averaged 346 kN m^{-2}, with quartzite (405 kN m^{-2}) recording higher values than schistose quartzite (324 kN m^{-2}). However, for 2 m (width) foundation footing, quartzite (on average) also shows higher bearing capacity (4,229 kN m^{-2}) than schistose quartzite (2,936 kN m^{-2}). The total settlement of the soils (avg. 26 mm), for structural loads of 100–200 kN and foundation width of 1–2 m, gave values less than permissible settlement for clay (100 mm) and sand (50 mm) dominated soil, with schistose and quartzite displaying an average of 27 and 24 mm.

iii. The soils have a mean CBR value of 38%, with CBR of schistose quartzite (35%) less than that of quartzite (42%). This implies that the soil is good, competent, with high bearing capacity for use as subgrade and subbase layer in pavement and embankment construction, however, quartzite showed relative competence over schistose quartzite.

iv. The statistical models developed from this study would be very useful in the derivation of these parameters quickly at field conditions. The model relationship between all the parameters correlated gave positive correlation coefficients; CBR and M_o (0.6522), R_D and CBR (0.6536), S_u and q_c (0.9849), $\emptyset$ and q_c (0.0009), CBR and I_c (0.0162), q_c and I_c (0.0687), K and $S_{e(100\,kN)}$ (0.0687), K and $S_{e(200\,kN)}$ (0.0349), S_u and M_o (0.0297). The models for CBR and M_o, R_D and CBR, S_u and q_c, q_c and I_c, and S_u and M_o follow the same trend, showing direct proportionality, while $\emptyset$ and q_c, and CBR and I_c, show the same trend of inverse proportionality.

The obtained values in the study area corroborated earlier studies conducted by Olaonipekun and Tanimola[24], Nwankwoala and Warmate[25], Falowo et al[23], Farinde and Oni[34], Oyedele and Okoh[26], Olorode et al[27], and Falowo[29–30]. It is therefore recommended that it is very expedient that appropriate foundation design of structures in the institution should take into cognizance these values so that the settlement would not be excessive or highly differential. It is also recommended that further laboratory studies should be conducted to ascertain the consistency of these empirical/model values and correlations using standard laboratory techniques.

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CONFLICT OF INTEREST

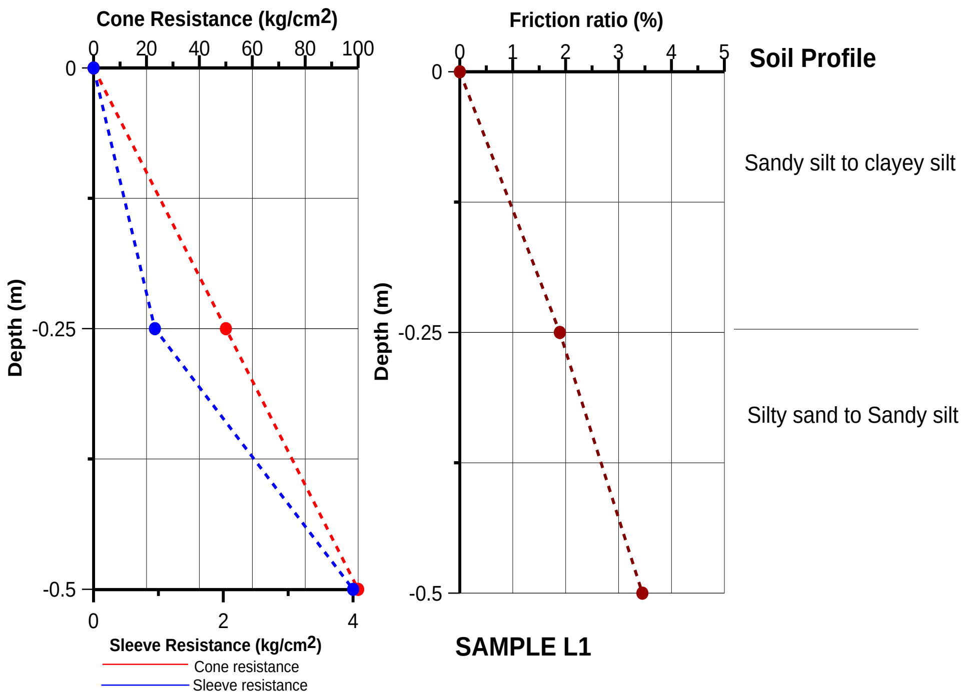
The authors agree that this research was conducted in the absence of any self-benefits, commercial or financial conflicts and declare the absence of conflicting interests with the funders. The authors confirmed that to the best of their knowledge, there is no conflict of interest or common interest with any institution, organization or a person that may affect the review process of the paper.

AUTHORS' CONTRIBUTIONS

Conceptualization: O. O. Falowo
Data curation: A. S. Daramola
Methodology: O. O. Falowo
Formal analysis: W. K. Olabisi
Visualisation: O. O. Falowo, A. S. Daramola, & W. K. Olabisi
Software: O. O. Falowo
Writing (original draft): O. O. Falowo, A. S. Daramola, & W. K. Olabisi
Writing (review and editing): O. O. Falowo, A. S. Daramola, & W. K. Olabisi
Validation: O. O. Falowo, A. S. Daramola, & W. K. Olabisi
Supervision: O. O. Falowo, A. S. Daramola
Funding acquisition: O. O. Falowo, A. S. Daramola, & W. K. Olabisi
Project administration: O. O. Falowo

APPENDICES OR SUPPLEMENTARY MATERIAL

A. Typical plots of cone resistance, sleeve resistance, and friction ratio with respect to depth, also showing the geological profile, at some locations or points 1, 3, 13, 16, 30, and 33



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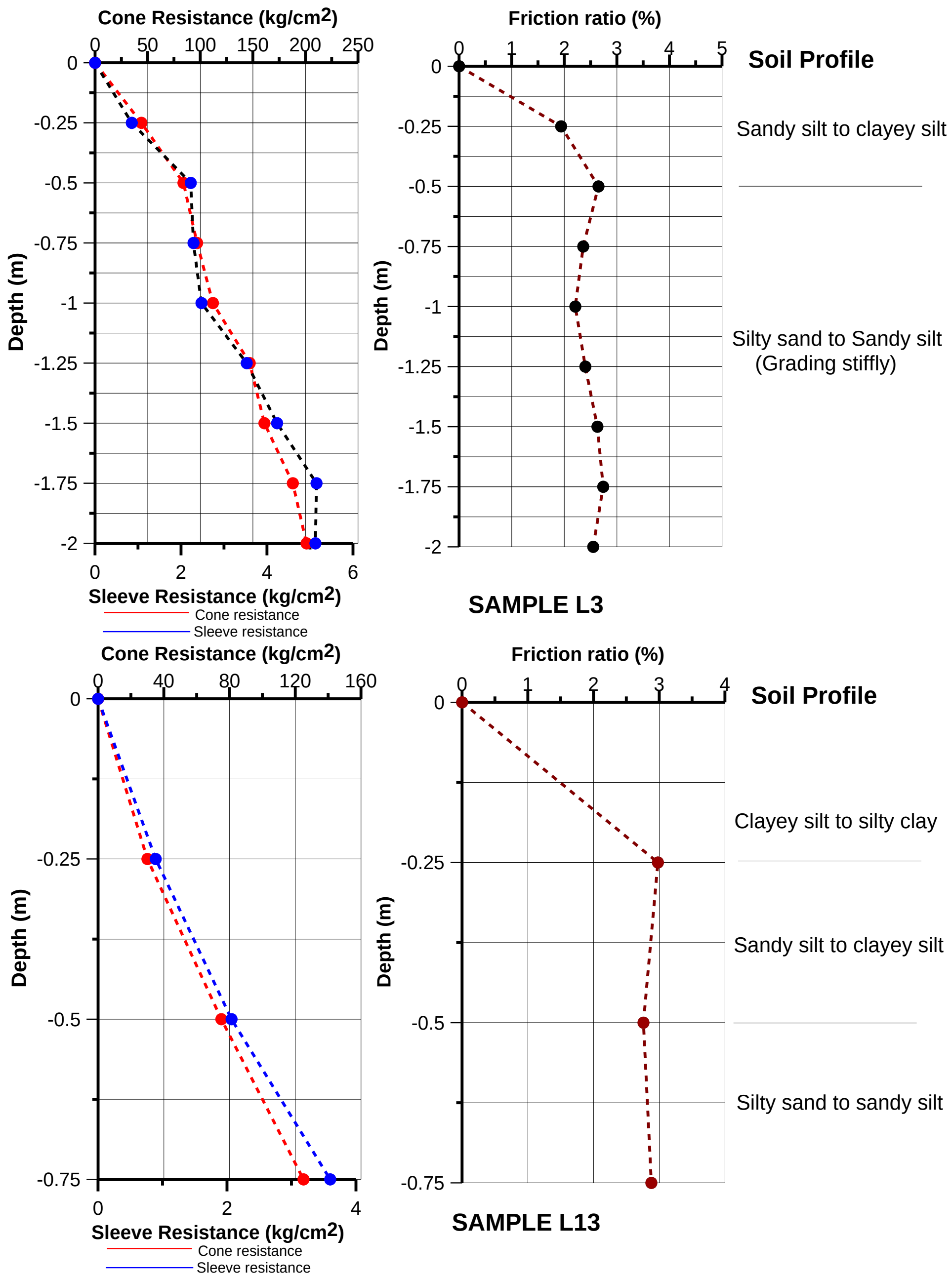

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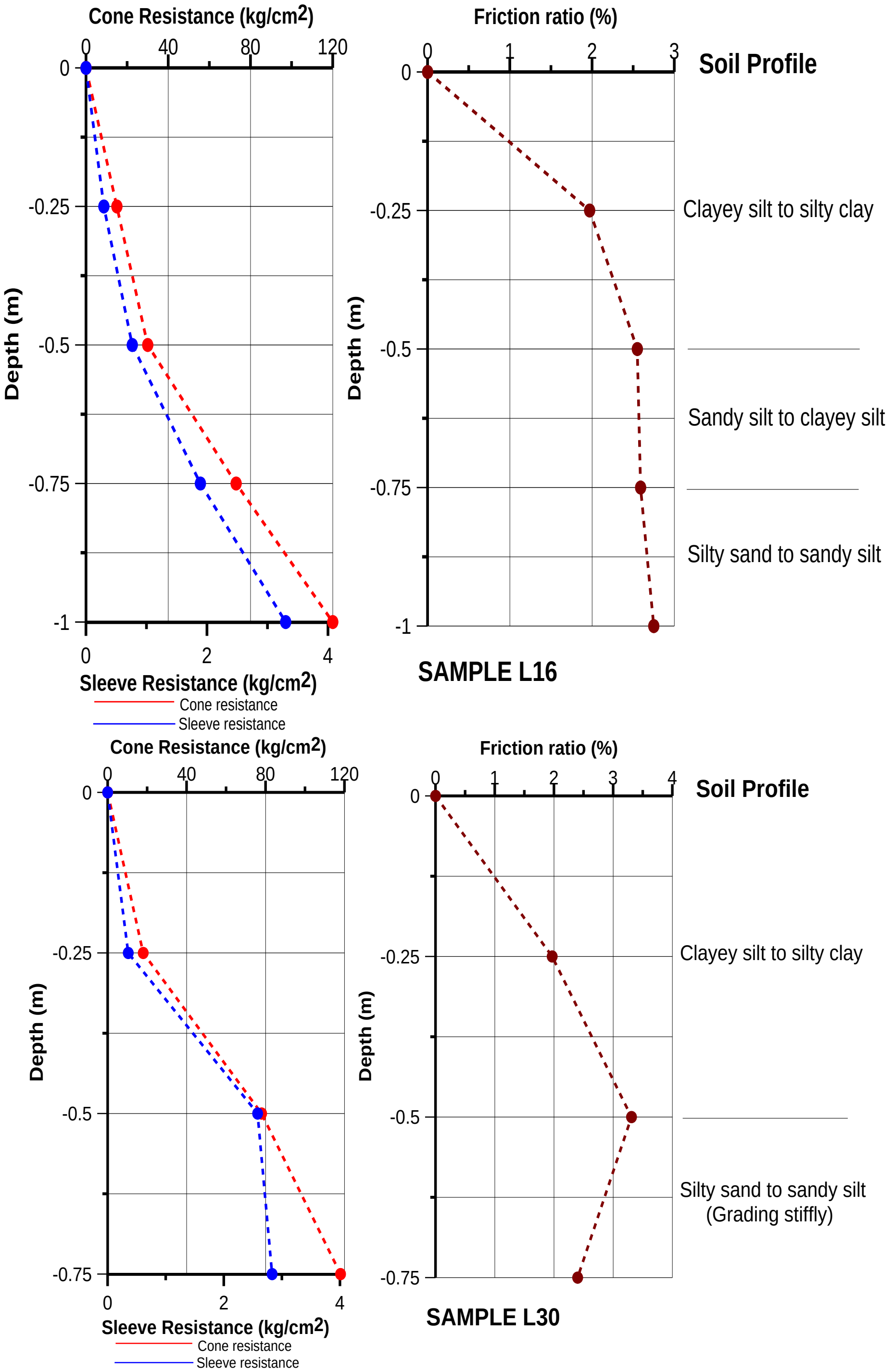

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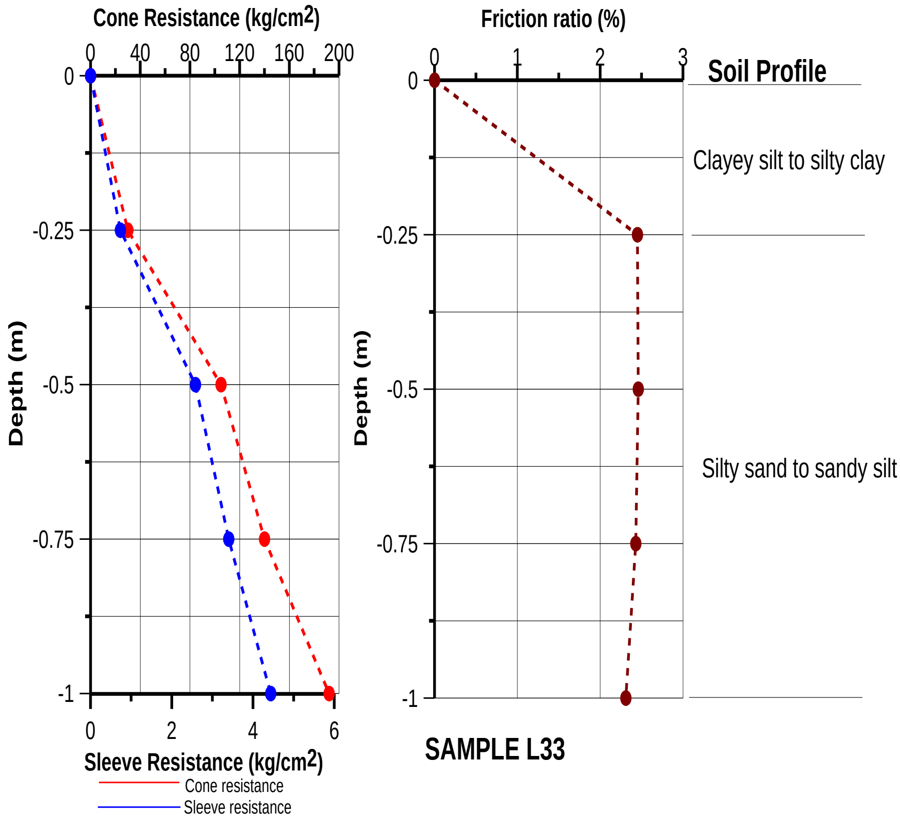

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