Use of Accelerated Procedure to Estimate the Behaviour of Shallow Foundation.

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Abstract-- **This paper describes a rapid foundation assessment procedure that employs simple field and laboratory analyses to determine the suitability of a construction site at Calabar, southeastern Nigeria. While the laboratory tests were limited to gradation, Atterberg limits, specific gravity and triaxial shear strength, simplified Terzaghi-Meyerhof's and Terzaghi-Peck's equations were employed for the bearing capacity and settlement analyses, respectively. Results indicated that the foundation soil (medium dense sand) classified as silty sand (SM), following Unified Soil Classification System. Sand-sized particles ranged from 60 – 74 %, plasticity index from 21 to 30 and specific gravity values from 2.61 to 2.62. Analyses showed that the site recorded increases in values of both ultimate capacity (q_f) and safe bearing capacity (q_s) with depth; q_f ranged from 373 kN/m^2 at depth of 0.5 m to 634 kN/m^2 at depth of 3.0 m, and q_s 149 kN/m^2 at depth of 0.5 m to 254 kN/m^2 at depth of 3.0 m. The calculated values of q_s (149 – 254 kN/m^2) fall within established range of presumed bearing values for medium dense sand, which is 100 – 600 kN/m^2, thus, indicating that (1) the foundation soil would have good stability, although moderate to high immediate compressibility may be expected (compression index of the soil ranged between 0.4 and 0.5), and (2) the testing program and analytical procedure were both, to some extent, reliable.**

Index Term-- **Project site, Rapid geotechnical assessment, Foundation, Bearing capacity analysis, Nigeria.**

I. INTRODUCTION

As a policy in Nigeria, prior to any engineering project, a site geotechnical investigation must be carried out. This became very necessary following frequent incidences of collapsed buildings especially in the southern and the Niger Delta areas of the country (Figure 1). Previous soil characterizations in these areas [1, 2, 3, 4, 5, 6] have observed that the area is characterized by widespread and irregular distribution of weak soils whose strength is further reduced by the presence of expansive clays in most locations.

Bearing capacity analytical procedures for foundation stability abound. However, most existing procedures require that series of field and laboratory tests be conducted in order to generate most components of the adopted equation(s). Often times, these set of tests are time consuming, uneconomical, complex and require state-of-the-art equipment which are not readily available in Nigeria at the moment. This predicament has resulted to situations that range from total omission of site characterization prior to site construction in Nigeria to neglect of the bearing capacity analysis aspect of the site investigation.

In this paper the bearing capacity and settlement estimations on a project site (soil geotechnical investigation) are presented. Both bearing capacity and settlement estimations carried out were based on the field observations and data generated by simple and economical but relevant geotechnical laboratory and empirical analyses.

General description of the area

The project area is situated directly behind the present location of the Central Bank of Nigeria (CBN), Calabar, southeastern Nigeria (see Figure 1). It currently serves as a relaxation park and generally slopes to the southern direction; forming terrace-like outlook. The sediments in the Calabar area belong mostly to the Cenomanian Odukpani Formation. Occurrences of Paleocene to Recent deposits are, however, noted southwards into the Niger Delta basin.

Details of the near surface lithologic and field descriptions of the soils of the Calabar area have been presented elsewhere by previous researchers [3]. However, in the surveyed site, the top soil is dark coloured and organic, and extends from 0 to about 0.5m depth. It supports grass growth, and was moist and mouldable in the field; sampling was in the mid rains. Underlying the top soil cover is dense brownish silty sand, with slight dry strength and insignificant amount of gravel-sized particles. This soil was also mouldable in the field but lost its moudability with moisture decrease. The soil profile of the site is shown in Figure 2.

The depth to the watertable was not ascertained in the field due to the relatively shallow depth of investigation and absence of water wells within the premises of the project site. However, the site is about 1km north of Marina (a local inland waterway), whose water level may serve as datum. The area, like the rest of the places in southeastern Nigeria, has two main seasons, dry and rainy seasons.

Methodology

Soil sampling

Soil samples were collected from 6 sampling points (designated as HB 1-6 in this study) and at varying depths: at 1m, 2m and 3m for grain size distribution and Atterberg limits tests; 1.5 m for the specific gravity tests, while those for the triaxial shear strength tests were collected at the depth of 1.5 and 3.0 m. The sampling points were selected using grid method. A 6 in. diameter hand auger was deployed for the sample collection for all other tests excepting the triaxial shear strength. 2 in diameter tubes were utilized for the collection of soil samples for the triaxial shear strength tests. The samples collected in the field were moved to the laboratory wrapped in polyethylene bags. This was to prevent moisture alterations.

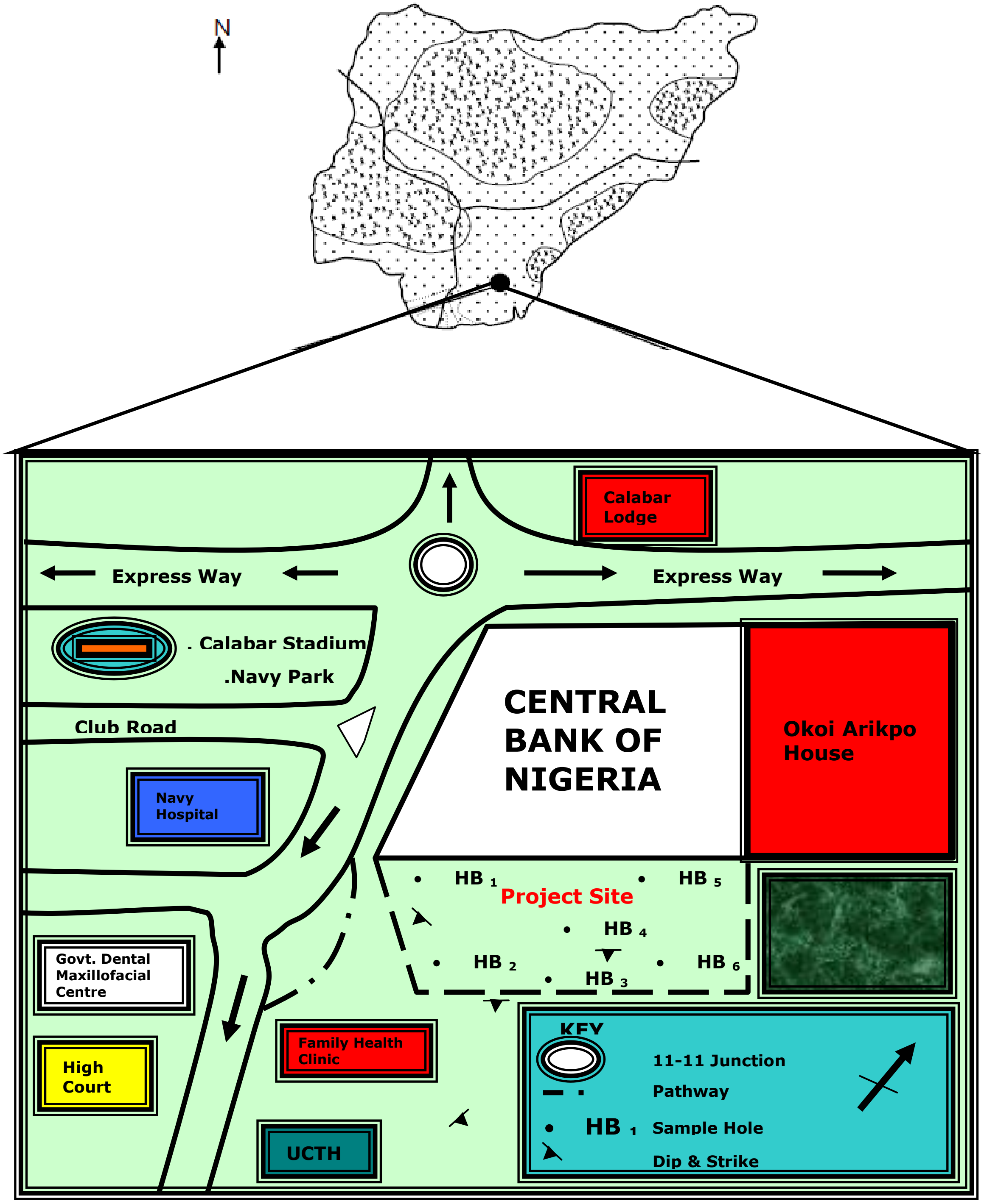

Fig. 1. Map of Calabar (southern Nigeria) showing the project site (Map not drawn on Scale)

Laboratory tests

The tests (grain size distribution, Atterberg limits, specific gravity and shear strength) carried out in this study followed procedures specified by Lambe [7], British Standard Institute [8] and Bailey [9]. The shear strength test was essentially an unconsolidated undrained (UU) shear strength test; without pore pressure measurement. Each test specimen was subjected to all-round confining pressure, δ_3 and loaded to failure with increased vertical pressure, δ_1. With the values of δ_3 and corresponding δ_1, the Mohr circles and envelopes of failure were constructed, and were used to determine the angle of shearing resistance, ϕ and cohesion, c of the specimens.

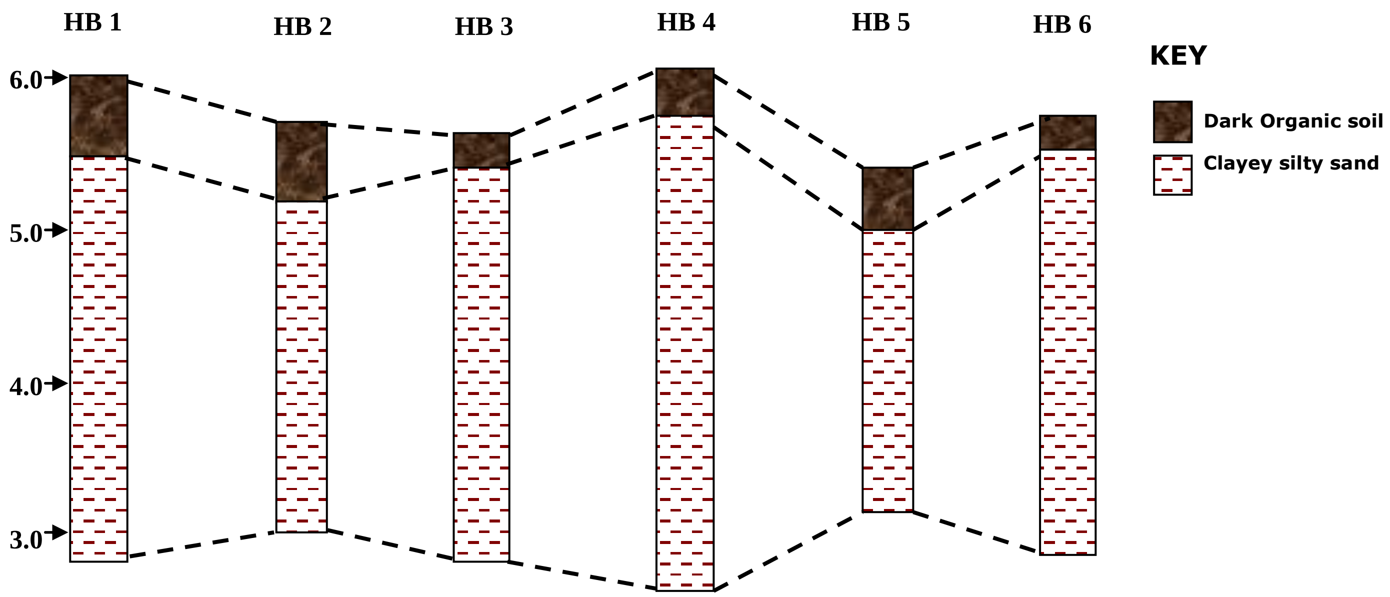

Fig. 2. Soil profile of the construction site (*height measured in metres above sea level*).

Bearing capacity analysis

Several bearing capacity equations are in existence. While some depend mostly on the nature of the soil (for example, whether the soil is cohesive or non-cohesive/cohesionless) under investigation others take into consideration the structural and foundation designs [10]. The general bearing capacity equation utilized in the study is that given by Tezerghi and Meyerhof [11], as follows;

$$q_f = cN_c + q_oN_q + \tfrac{1}{2} B\gamma N_\gamma \quad (1)$$

Where; q_f is ultimate bearing capacity

q_o is surcharge (i.e., weight of soil above the foundation level).

γ is unit weight of soil

c is cohesion

B is width of foundation.

N_c, N_q and N_γ are bearing capacity factors and they depend on cohesion (c) and angle of internal friction (ϕ).

Surcharge (qo) and unit weight of soil (γ) are given below as equations (2) and (3), respectively:

$$qo = \gamma D \quad (2)$$

$$\gamma = \rho g \quad (3)$$

Where; D is depth of foundation

ρ is specific gravity

g is acceleration due to gravity (approx. $10 m/s^2$)

Safe bearing capacity was estimated using the expression below, according to Sowers and Sowers [11]:

$$q_s = q_f/SFM \quad (4)$$

Where, q_s is safe bearing capacity

SFM is safe minimum permissible safety factor.

Deductions and assumptions

The values of c, ϕ and γ were deduced from laboratory test results. To be conservative, the minimum values of the laboratory derived parameters ($25 kN/m^2$, 15° and 2.61 for c, ϕ and γ, respectively) were used for the computation of the bearing capacity values of the foundation. Values of the bearing capacity factors (i.e., N_c, N_q and N_γ) were deduced from bearing capacity factors chart (Meyerhof curve, see Figure 3), and are as follows; 11, 4 and 3.5 for N_c, N_q and N_γ, respectively. Width of the structural foundation (B) was assumed to be 1 m, while SFM was assumed to be 2.5. Sowers and Sowers [11] note that SFM value of 2.5 is effective and reliable for most range of structural projects. The required factor of safety depends on the type of structure, the type of soil and other factors and typically range between 2.0 and 3.5 [10].

Settlement analysis

Settlement was estimated using compressibility equation by Terzaghi and Peck [11], as given below;

$$C_c = 0.009\,(LL - 10) \quad (5)$$

Where; C_c is compression index

LL is liquid limit

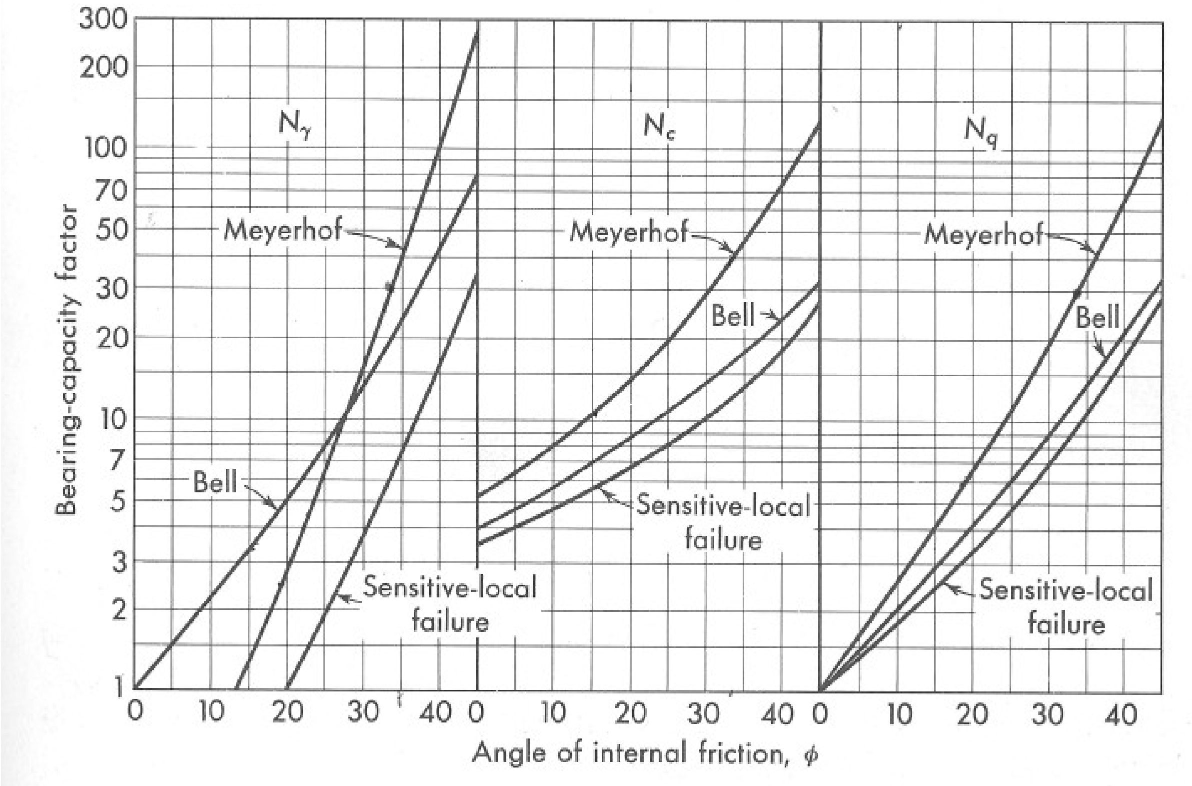

Fig. 3. Bearing capacity factors for general bearing capacity equation.
Source: Sowers and Sowers [11]

RESULTS AND DISCUSSION

Grain size distribution

The results of the gradation analyses of the soil samples are summarized in Table I, while the average depth distribution of the particle-size ranges is given in Figure 4. Table I indicates that the soil samples are sand dominated. Plots of the mean values of the grain sizes in Figure 4 buttress the fact that soil samples were characterized by high percentage of sand (even with increase in depth), while the fines fraction slightly decreased with depth. No significant depth variation was shown in the percentage of gravel. Well graded sand is most often imcompressible and reasonably permeable, thus, serve well as foundation material. When the fines fraction is the expansive type, it most often constitutes engineering problems, especially when it also occurs in significant amount.

Table I
Range of grain size distribution of the foundation soil at six sampled points.

		Parameter	
Depth (m)	Fines (%)	Sand (%)	Gravel (%)
1.0	28-36	60-68	2-4
2.0	22-33	62-74	4-8
3.0	22-24	71-74	2-5

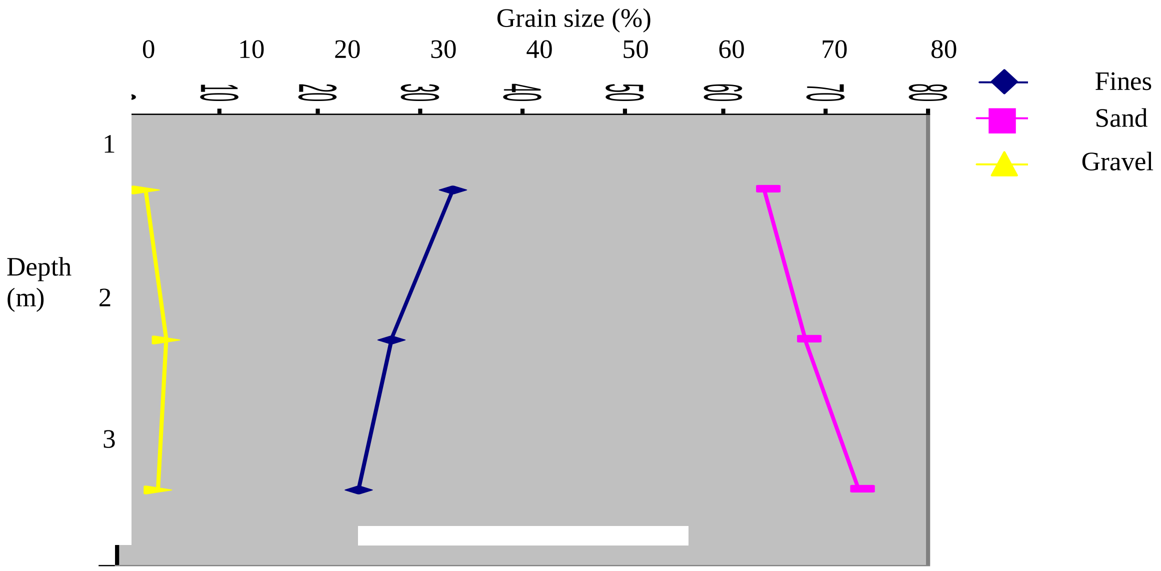

Fig. 4. Mean values of grain size distribution with depth.

Atterberg limits and specific gravity

The summary of the results of the Atterberg limits and specific gravity tests carried out on the studied soil samples are presented in Tables II and III, respectively. Results of gradation tests had shown that the amounts of fines are low, Atterberg limits tests, however, gives indication that the fines have high values of liquid limits that even persisted with depth (see Table II). These high Atterberg limits reveal that the predominant clay type is, most probably, montmorillonite or illite [11]. A combination of the results of the gradation and Atterberg limits tests indicate that the soil is silty sand (SM), following Unified Soil Classification System.

Plots in the Plasticity chart (Figure 5) also indicate that the fines of the material are, most probably, silts of high LL or clay of low plasticity clay(s). It has been observed that although the expansive clay types abound in the southern Nigeria, especially in the Niger Delta, occurrences of non expansive types should also be noted [3].

The fact that LL may correlate well with compressibility in soil has earlier been highlighted [11]. Previous study indicates that soil which has LL and PI of its fines (particles passing 75 µm) greater than 10 and 35, respectively, would likely be troublesome in construction projects [10].

Specific gravity (G_s) values of the soil (excluding air and water) indicate predominance of stable minerals (most probably quartz) that would have fair durability in construction project. G_s of the soil range from 2.61 to 2.62, and may be an indication of fair durability as foundation material. Potentially durable construction aggregate should have G_s value of 2.625 or above [12]. G_s is also an important parameter in the bearing capacity analysis of foundation [11].

Table II
Range of Atterberg limits of the foundation soil at six sampled points.

	Parameter		
Depth (m)	LL	PL	PI
1.0	60-66	35-39	24-30
2.0	56-63	33-37	21-26
3.0	53-58	32-35	19-24

Table III
Mean value of specific gravity results and
Natural moisture content of the foundation soil.

Sampling point*	Specific gravity
HB 1	2.61
HB 2	2.62
HB 3	2.61
HB 4	2.62
HB 5	2.61
HB 6	2.62

*sampling depth 2m.

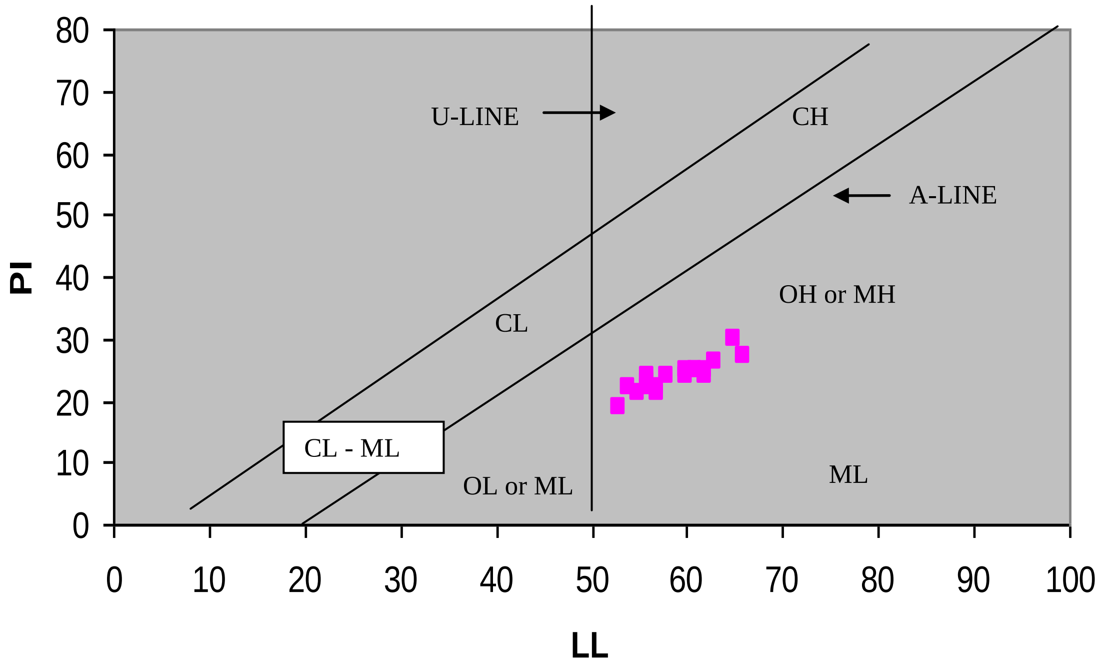

Fig. 5. Mean values of Atterberg limits with depth.

Triaxial shear strength

The summary of the strength parameters [angle of shearing resistance (ϕ) and cohesion (c)] deduced from the laboratory triaxial shear strength tests are presented in Table IV. Results show that ϕ and c have insignificant horizontal variation and are low in comparison with most stable soils [11]; generally below 25^o and 35kN/m^2, for ϕ and c, respectively. These relatively low c and ϕ values indicate that the soil may experience moderate to fair bearing capacity as foundation material, although other factors like soil type, moisture condition, structural design and foundation contribute to determine the general bearing capacity of foundation materials [11].

Table IV
Summary of the strength tests result of the foundation soil.

Sampling point*	Strength parameters	
	ϕ (o)	c (kN/m^2)
HB 1	24	30
HB 2	15	35
HB 3	18	35
HB 4	16	30
HB 5	20	25
HB 6	15	50

*sampling depth of 4m.

Suitability of the investigated site for construction project

The estimated bearing capacity and settlement data of the investigated site are presented in Tables V and VI, respectively. Bearing capacity analysis shows that the soil (as a non-cohesive soil) would be reasonable stable for use as foundation material, especially when compared with the presumed bearing values of most types of cohesive soils (Table VI). Bearing capacity problems may, however, arise when there is excessive moisture fluctuation, which when it occurs would result in expansion or settlement of the foundation soil.

Table V
Summary of bearing capacity analysis of the foundation soil.

Depth (m)	qo (kN/m2)	qf (kN/m^2)	qs (kN/m^2)
0.5	13	373	149
1.0	26	425	170
1.5	39	477	191
2.0	52	529	212
2.5	62	582	233
3.0	78	634	254

Table VI
Presumed bearing values of different types of soils

Category	Types of rocks and soils	Presumed bearing value (kN/m²)
Non-cohesive soils	Dense gravel or dense sand and gravel	> 600
	Medium dense gravel, or medium dense sand and gravel	<200 to 600
	Loose gravel, or loose sand and gravel	<200
	Compact sand	>300
	Medium dense sand	100 to 300
	Loose sand	<100#
Cohesive soils	Very stiff bolder clays & hard clays	300 to 600
	Stiff clays	150 to 300
	Firm clay	75 to 150
	Soft clays and silts	< 75
	Very soft clay	Not applicable
Peat		Not applicable
Made ground		Not applicable

Source: http://environment.uwe.ac.uk/geocal/foundations/gifs/tremmie.gif

depends on degree of looseness.

Table VII indicates that the high LL exhibited by the fines content of the soil, although less in amount, may introduce considerable amount of settlement in the foundation. Suitability of the soil as foundation may be improved by compaction. Compaction reduces void spaces in soil, decreases pore pressure and its consequences in construction projects, thus, enhance its suitability, especially as fills [13]. Foundation depth of range 1.5 to 2.5 m may be most economical, considering the cost of deeper excavation. The choice of the depth range may also be supported by the fact that the soil indicated reasonable values of both ultimate and safe bearing capacities within the depth range.

Table VII
Summary of compressibility analysis.

Depth (m)	LL	Cc	Term*
1.0	61	0.5	High compressibility
2.0	56	0.4	High compressibility
3.0	53	0.4	High compressibility

*after Sowers and Sowers [11].

CONCLUSIONS

The simplified site geotechnical investigation and empirical analysis carried out in this study have provided insights into the effectiveness of the adopted procedure for foundation assessment. It was evident that the testing programme aided field observations and was useful in the classification of the medium dense sand foundation soil classified as silty sand (SM), following Unified Soil Classification System.

Bearing capacity analyses indicated that the estimated safe bearing capacity q_s (149 – 254 kN/m^2) falls within the established range of presumed bearing values for medium dense sand (similar to the tested soil), which is 100 – 600 kN/m^2. Hence, to some degree of certainty, the study was able to establish bearing characteristics of the site, buttressing the fact that the testing program and analytical procedure were both effective and reliable for rapid foundation assessment.

The implementation of the suggested procedure would, however, likely be limited to shallow foundation for lightweight structures and may not be a perfect replacement of the standard site characterization procedure, which most often requires consolidation testing and deeper investigation.

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