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Local Geology, Shear Strength Properties and Bearing Capacity of Coastal Plain Sands in Uyo Metropolis, Akwa-Ibom State, Southeastern Nigeria

Abidemi Olujide Ilori[1*], Charles Etim Akpan Uko[1] and Ini Christopher Umoh[1]

[1]*Department of Civil Engineering, University of Uyo, Uyo, Akwa Ibom State, Nigeria.*

Authors' contributions

The article is a collection of all the three authors' experiences in the practice of Foundation/ Structural Engineering within Uyo metropolis, Akwa Ibom State, Southeastern Nigeria. It was supported by additional field and laboratory tests results carried out by the authors AOI and ICU prepared the manuscript, which was read by authors CEAU and ICU. All the authors approved the final manuscript.

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ABSTRACT

The Bearing capacity of the soil within Uyo metropolis in South-Eastern State of Akwa Ibom was investigated in this study. The soil belongs to Coastal Plain Sand often called the Benin Formation in the geology of Niger Delta. Both Field and Laboratory methods were employed in the study. The field method consisted of Cone Penetration Test (CPT) with a 2.5 ton Dutch Guada cone penetrometer, and the Light weight penetrometer LRS 10. For the CPT, depth of investigation was refusal depth which varies from about 9.0 m to 20.0 m. The depth of investigation by the LRS 10 was not more than 6.0 m. The direct parameter the LRS 10 evaluates is the relative density. Soil sounding with the LRS 10 indicated for all the sites a 'loose to medium' consistency. No dense or very dense stratum was encountered. The Laboratory method employed was the Direct shear box tests This was used to determine the cohesive property and angle of shearing resistance of the soil, that is the C- Ø property. The cohesion varies very widely; with a value ranging from a zero value to 54 kN/m^2. The angle of shearing resistance ranges from 8º to 30.7º, with more than ninety

*Corresponding author: Email: bidemiini@gmail.com, abidemiilori@uniuyo.edu.ng;

percent falling below 28º, indicating a highly compressible soil that is prone to local shear failure. Ultimate bearing capacities are as low as 100.93 kN/m^2 and as high as 571.1 kN/m^2. Settlement associated with safe bearing pressure estimated from CPT data ranged from 0.35 cm to 3.89 cm. while that from laboratory gives lesser values, thereby making that from the field value conservative.

Keywords: Local geology; coastal plain sands; angle of shearing resistance; local shear failure; bearing capacity.

1. INTRODUCTION

The ultimate bearing capacity equation of Terzaghi [1] in simplest form for a rectangular footing in a cohesive soil with some degree of angle of internal friction for a case of general shear failure is given by

$$Q_u = cN_c\left(1 + 0.3\left(\frac{B}{L}\right)\right) + D_f\gamma\, N_q + \frac{1}{2}\,\gamma B N_\gamma\left(1 - 0.2\left(\frac{B}{L}\right)\right) \quad (1)$$

Where

C = Effective cohesion value of the soil
D_f= Depth of the foundation level from the ground surface
B = Width of the footing
L = Length of the footing
γ= Effective unit weight of the soil.

The ultimate bearing capacity of the soil depends on

- The bearing capacity factors, $N_c\, N_q\, N_\gamma$ which are a function of the angle of shearing resistance of the soil.
- Relative density of the soil, which the equation assumes to be dense or very dense for general shear failure,

Vesic [2] investigated the relationship between foundation failure, relative density of the soil, and depth to width ratio. Part of his result is the chart that relates the depth to width ratio versus the relative density of the sandy soil he used in the experiment. For shallow foundations; the ratio of depth to width is often less than one ($^D/_B < 1$). From the chart, the higher the ratio of depth to width of the footing and with soil relative density of about 0.35, punching shear failure dominates, between relative density of 0.35 to 0.75, local shear failure dominates, above relative density value of 0.75, general shear failure dominates. Terzaghi's bearing capacity equation (equation 1 above), commonly used for estimating ultimate and hence 'safe' or allowable bearing capacity assumes general shear failure. For soils that have relative density in the mixed state or transition from local shear failure to general shear, Peck, et al. [3] developed a chart from which the bearing capacity factors can be extracted based on relative density, Standard Penetration Test value (SPT), and angle of shearing resistance. This chart for the bearing capacity factors N_q and N_γ are developed on the following assumptions.

1. Purely local shear failure occurs when Ø < 28º.
2. Purely general shear failure occurs when Ø > 38º.
3. Transition curves for values of Ø between 28º and 38º represent the mixed state of local and general shear failures.

For local shear failure, the bearing capacity factors are modified by using a modified angle of shearing resistance as $\overline{\emptyset} = \tan^{-1}(0.67\tan\,\emptyset)$ and modified cohesion value, $\bar{C}$ = 0.67C.

Equation (1) is modified taking the new angle of shearing resistance and cohesion values as

$$Q_u = 0.67c\bar{N}_c\left(1 + 0.3\left(\frac{B}{L}\right)\right) + D_f\gamma\,\bar{N}_q + \frac{1}{2}\,\gamma B\bar{N}_\gamma\left(1 - 0.2\left(\frac{B}{L}\right)\right) \quad (2)$$

Where $\bar{N}_c$, $\bar{N}_q$, $\bar{N}_\gamma$ are modified bearing capacity factors. Which are actually bearing capacity factors with modified angle of shearing resistance which is smaller than that for the soil.

Vesic [4] Proposed the following equation in which he introduced factors that account for soil compressibility. The equation is

$$Q_u = C' N_c F_{cs} F_{cd} F_{cc} + qN_q F_{qs} F_{qd} F_{qc} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma c} \quad (3)$$

Where

F_{cc}, F_{qc}, $F_{\gamma c}$ are soil compressibility factors,
F_{cs}, F_{qs}, $F_{\gamma S}$, are shape factors,
F_{cd}, F_{qd}, $F_{\gamma d}$ are depth factors,
N_c, N_q, N_γ are Terzaghi's bearing capacity factors.

These factors are used to modify respectively the cohesive term, CN_c , the surcharge term, γDN_q , and the geometric term, γBN_γ of Terzaghi fomula.

Expressions governing the computation of the shape and depth factors are presented in the Appendix of this article. Terzaghi's bearing capacity factors and the modified bearing capacity factors values are obtained from standard text such as [5].

Other factors that could modify the ultimate bearing capacity value is the inclination factor which generally reduces the value of the bearing capacity as the inclination increases.

The magnitude of the angle of shearing resistance, the soil consistencies are the results of the processes that were responsible for the formation of the soil; that is the geology. The influence of geology, angle of shearing resistance and, relative densities, on the bearing capacity of the soils around Uyo metropolis is the main object of this study.

1.1 Objectives of Study

1. Determine the C- Ø properties from undisturbed soil samples obtained from building sites at depths in which the foundations were placed.
2. Determine the relative densities of the soil on the sites
3. Calculate the appropriate Ultimate bearing capacity based on the results in (1) and (2) above; and also the associated settlements.

1.2 Description of Study Area and Geology of Local Sediments

Uyo town, is the capital of Akwa –Ibom State located in the Southeastern Nigeria, within the oil rich Niger Delta. Uyo lies approximately between latitude 4º56', and 5º6' N and between longitude 7º48' and 8º02'E. Its location in the Niger delta region necessarily leads to extensive urbanize-tion that is characterized by infrastructural developments which involve construction of single and multi-floor buildings among other civil engineering facilities. The two or more floors buildings impose reasonable loads on the supporting soil. The city relief is generally flat with little undulating plains except in areas that have deep ravines which are located mostly in the North eastern part of the city.

According to Abam [6], the Quaternary sediments which are the structural foundation materials in the Niger Delta were deposited in a wide variety of hydrologic conditions resulting in unique geomorphologic units which have rendered them both vertical and laterally heterogeneous in form and engineering properties.

Akpokodje [7], On the basis of similarity in geotechnical, geological and geomorphological characteristics recognized four major superficial soil groups.

Based purely on geomorphological criteria described by Allen [8], six major geomorphic units can be identified, namely: Beaches and Barrier Islands, Mangrove swamp forests, Coastal Plain Sands, Warri-Sombreiro Deltaic plain, Lower Niger Flood plain, and Niger flood zone.

According to the [9] base map of Akwa- Ibom State, the geology of Uyo is dominated by the Tertiary–Recent (Quaternary) sediments Coastal Plain Sands [10]. The Coastal Plain Sands is one of the six major geomorphic units mentioned above. Petro-graphically [8], the sands are poorly graded, medium to mostly fine grained, friable, with clay and silt. The grains are sub angular to well rounded, and are believed to have been deposited in a continental fluviatile to deltaic environment. The sands covering most of the areas in this study are continental sands.

The soils in the six geomorphological units are superficial soils of Quaternary age. Underlying them is the Benin Formations one of the three main Formations that constitutes the Niger delta geology. Unlike some other areas of the Niger delta, the ground water table in Uyo is at an average of about 20 m below the ground surface in most areas. Ground water level does not influence the bearing strength of shallow foundation soils

2. MATERIALS AND METHODS

Undisturbed and disturbed soil samples were obtained at both proposed building sites and also building sites under construction. The buildings are of multi floor level. Samples were obtained in the range of 1.2 m to about 2.0 m depth, from trial pits.

The undisturbed soil samples were placed in direct shear machine, and consolidated drained test were performed on them, while index properties tests were performed on the disturbed samples for classification purposes. Classification tests carried out include mechanical sieve analysis, Atterberg limits and natural moisture content tests. All these were carried out in accordance to relevant American Society of Testing and Materials (ASTM) standards.

The digital direct shear machine was model 30-WF6016 T2 (Wykeham Fragrance) attached to a digital data logger which records all parameters automatically during a testing operation and the test were performed in accordance with [11]. Strain controlled test was the type that was carried out with the sample sets. The machine applied strain at the rate of 0.5 mm per minute. The Direct shear test machine is equipped with strain gauges transducers to measure both horizontal and vertical deformations from the start to the end of the test. It has facility to determine both the peak and residual shear strength of soil, but the latter was not determined for the specimens in this study. Normal load applied were 50 kg, 100 kg, and 150 kg to most of the soil samples whereas a load sequence of 10, 25, and 50 kg were applied on sample from one site only. Porous plates were placed on top and bottom of the soil sample to allow drainage from soil.

Soil consistencies were determined for some of the sites using German type light weight penetrometer designated LRS 10.

Cone penetration test (CPT) data were acquired from three of the sites investigated. CPT tests and eight LRS 10 tests were carried out on one site; while CPT tests only on the other two. The CPT data were acquired with 2.5 Tonne Guada Dutch cone. Its cone resistance values are presented. One of the sites where only CPT data were acquired has a Christian worship center on it and an expansion of this structure is being contemplated, so also the site where both the CPT and LRS data were acquired, this has a residential bungalow building on one side of the site. Two Standard penetration test data are presented. These were obtained from SPT drilling records from the two sites.

The tests carried out on different sites are presented in Table 1. The approximate geographical coordinates for each site are also listed in the same Table. Fig. 1 also presents these locations on a simplified map of Uyo metropolis.

2.1 Data Analysis

2.1.1 Penetration tests

2.1.1.1 *Light weight penetrometer* and *Cone Penetration Tests (CPT)*

The Light weight penetration (LRS 10) acquires data in blows per 10 cm [12]. Gives guidance on both qualitative and quantitative interpretation of the LRS 10 readings. It gives an equation that estimates relative densities of different soil strata in situ. The equation is of the form

$$I_D = 0.21 + 0.230 log n_{10} \qquad (4)$$

where,

I_D = relative density, and
n_{10} = the number of blows per 10 cm.

The relative density index values of the soil were also converted to equivalent Cone Penetration Test (CPT) values using equation proposed by Kulhawy and Mayne [13]. The expression is given as

$$D_r(\%) = 68\left[Log\left(\frac{q_c}{\sqrt{P_o\sigma_o'}}\right) - 1\right] \qquad (5)$$

Where

$\mathrm{D_r} =$ relative density in %
$\mathrm{q_c} =$ Cone penetration resistance, CPT $(\mathrm{kN/m^2})$
$\mathrm{P_a} =$ atmospheric pressure $= 101.4\ (\mathrm{kN/m^2})$
$\sigma_o' =$ effective overburden pressure, $(\mathrm{kN/m^2})$

Modified [14], equation is used to estimate the safe bearing pressure. The equation is given as;

$$q_s = 2.7q_c \quad (\mathrm{kPa}) \qquad (6)$$

where q_c is the cone point resistance in kg/cm^2 and q_s in kPa.

Equation (6) was developed for a settlement of 25 mm.

The CPT data were used to estimate bearing capacity values and for settlement computations based on consolidation principles.

Settlements were estimated using the equation

$$S = M_v\sigma\, H \qquad (7)$$

Where

S = settlement.
$\sigma =$ imposed stress = Ultimate bearing, allowable, or safe bearing stress
H= thickness of the soil layer on which the load is applied which is taken as 1.5B representing zone of significant stress (with a stress limit of 0.1σ). Soil thicknesses and stratification were also determined by cone resistance values.
M_v = coefficient of volume compressibility.

M_v is estimated approximately as reciprocal of constrained modulus, E, from direct shear box test or CPT values or equivalent cone resistance values for the LRS 10 penetrometer blows. In their investigation of CPT data for offshore sands in the North Sea; [15] came up with some relationships that can be used to estimate 'E' from CPT cone values. For Normally consolidated sands, they proposed the following relationships

$$E = 4q_c;\ if\ q_c < 10\ MPa \tag{8}$$

Or

$$E = 2q_c + 20: if\ 10\ MPa < q_c < 50\ MPa \tag{9}$$

And q_c is in MPa. The value of q_c is taken as the average cone resistance over the 1.5B depth beneath the footing, which is 3.0 m (if a 2.0 m width footing is assumed). These equations were used to estimate 'E' values in this work.

2.1.2 Direct shear box data

Different soil parameters were evaluated from direct shear box tests experiments. These include;

2.1.2.1 Shear strength parameters

The angle of internal friction or shearing resistance, and cohesive values are determined from direct shear box tests. These parameters are effective values and therefore conservative. The peak normal force and peak shear stress method rather the average normal peak shear method was used to estimate both parameters. The first method is reported [16], to give a smaller value than the latter method, thereby making the values determined for both parameters conservative. Due to the volume of calculations involved in using the shear parameters to estimate bearing resistance, excel worksheet developed by the lead author was

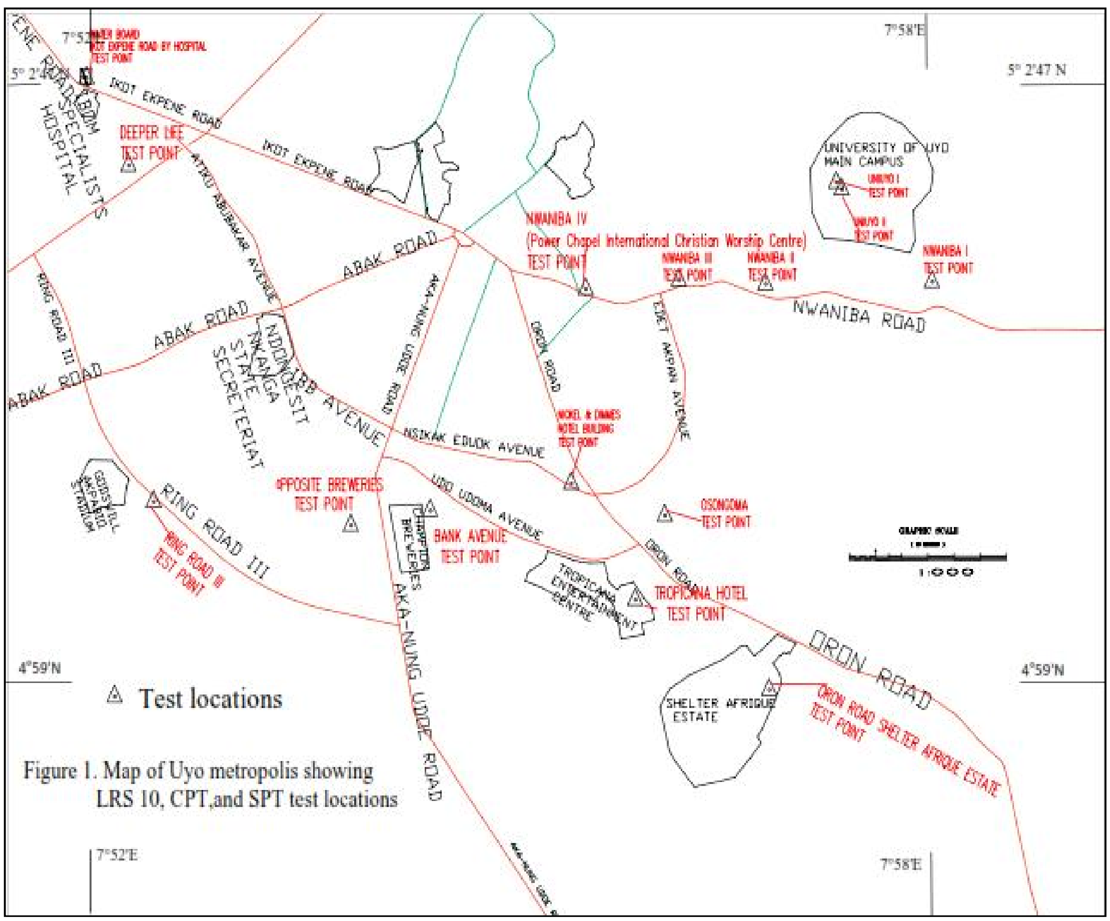


Figure 1. Map of Uyo metropolis showing LRS 10, CPT,and SPT test locations

Fig. 1. Locations on a simplified map in this study

Table 1. Test locations, and test types

Study locations	Approximate geographical coordinates	Index properties, and sieve analysis	Direct shear test (Digital)	Direct shear test (Non-digital)	Light weight penetrometer test (LRS 10)	Cone Penetration Test (CPT)	Standard Penetration Test (SPT)
Osongsoma	5º0'15.64"N,7º57'9.12"E	2	3				
Ring Road III	5º0'17.50"N, 7º53'24.76"E	2	3				
Nwaniba I	5º1'38.61"N, 7º58'35.26"E	1	2				
Opposite Breweries	5º0'40.92"N, 7º54'52.01"E	2	2				
Nwaniba II	5º1'42.48"N, 7º57'47.73"E	1	1				
Bank Avenue	5º0'14.20"N, 7º55'28.53"E	1	1			5	
UNIUYO III (1000) Seater Auditorium		1	1				
Nwaniba III	5º1'42.01"N, 7º57'11.30"E	1	2				
Tropicana Hotel	4º59'39.25"N,7º56'57.91"E	2		2	6		
Water board Ikot -Ekpene road by Ibom Specialist Hospital Uyo	5º2'47.86"N , 7º52'59.46"E	3		3			2
UNIUYO 1	5º2'23.75"N, 7º58'22.00"E	1	2				
UNIUYO 1I	5º2'21.05"N , 7º58'24.38"E	1	1				
Nwaniba IV (Power Chapel International Christian worship center)	5º1'42.79"N, 7º56'32.81"E	2				4	
Oron Road Shelter Afrique estate	4º59'19.03"N, 7º58'6.17"E	3		2	7		
Deeper Life Site	5º2'29.99"N , 7º53'16.90"E	3		2	5		
Nickel and Dimmes Hotel Building Site	5º0'28.28"N, 7º56'28.31"E	2			4		
Abak road	5º2'29.99"N , 7º53'16.90"E	4			8	5	
Off Dominic Utuks street close to Ravine	5º2'4.36"N, 7º56'25.71"E						2
Total numbers of test		**32**	**18**	**9**	**30**	**14**	**4**

Table 2. Constrained modulus, shear modulus, computed from direct shear test result for normal stress of 68.125 kPa at the bank avenue site

Reading No.	Vertical displacement dv (mm)	Horizontal displacement dh (mm)	Shear force T(N)	Corrected Area (Ac) (m^2)	Vertical stress ($\delta\sigma_v$) (kpa)	shear stress τ (kpa)	τ /N	Shear strain	Vertical strain	Constrained modulus (kPa)	Shear modulus (kPa)	Ratio of vertical strain to horizontal stain	Coefficient of volume compressibility (Per MPa)
1	0.017	0	0.2	0.003600	68.13	0.06	0.00	0.0000	0.0000				
2	0.02	0.006	8.7	0.003600	68.13	2.42	0.04	0.0001	0.0010	68131.81	24169.08	10.000	0.0147
3	0.023	0.006	9.2	0.003600	68.13	2.56	0.04	0.0001	0.0012	59245.05	25558.11	11.500	0.0169
4	0.06	0.58	33.1	0.003565	68.79	9.28	0.13	0.0097	0.0030	22929.99	960.43	0.310	0.0436
5	0.245	1.404	83.3	0.003516	69.76	23.69	0.34	0.0234	0.0123	5694.475	1012.53	0.524	0.1756
6	0.35	2.505	85.3	0.003450	71.09	24.73	0.35	0.0418	0.0175	4062.465	592.26	0.419	0.2462
7	0.43	3.818	106.5	0.003371	72.75	31.59	0.43	0.0636	0.0215	3383.936	496.50	0.338	0.2955
8	0.662	5.412	136.6	0.003275	74.88	41.71	0.56	0.0902	0.0331	2262.208	462.38	0.367	0.4420
9	0.782	7.23	160.3	0.003166	77.46	50.63	0.65	0.1205	0.0391	1981.043	420.15	0.324	0.5048
10	0.822	9.296	174	0.003042	80.61	57.19	0.71	0.1549	0.0411	1961.434	369.16	0.265	0.5098
11	0.872	11.598	194.9	0.002904	84.45	67.11	0.79	0.1933	0.0436	1936.903	347.19	0.226	0.5163
12	0.894	11.722	176.6	0.002897	84.67	60.97	0.72	0.1954	0.0447	1894.092	312.06	0.229	0.5280
13	0.982	12.566	171.3	0.002846	86.17	60.19	0.70	0.2094	0.0491	1755.038	287.39	0.234	0.5698
14	1.005	13.644	165.3	0.002781	88.18	59.43	0.67	0.2274	0.0503	1754.752	261.35	0.221	0.5699
15	1.235	14.261	160.5	0.002744	89.37	58.48	0.65	0.2377	0.0618	1447.219	246.06	0.260	0.6910
16	1.458	15.722	156.7	0.002657	92.31	58.98	0.64	0.2620	0.0729	1266.316	225.10	0.278	0.7897

used to carry out the calculations. The accuracy of the worksheet was tested with some examples from foundation engineering texts. One such example is quoted under the section "Compressibility Index, shear modulus, and bearing capacity" below.

2.1.2.2 Elastic modulus, shear modulus, and compressibility

The measuring accuracy of conventional laboratory tests has also improved and stress-strain measurements can now be performed at very low strain levels, during triaxial, simple or direct shear tests at small strain [17], this allow the computations of series of soil parameters often required for foundation analysis especially settlement computations. These include elastic modulus, shear modulus. Typical computations are displayed in Table 2. In the Table constrained modulus is computed as the ratio of normal stress to vertical strain. The inverse of constrained modulus approximates coefficient of volume compressibility (m_v), Shear modulus as ratio of shear stress to shear strain (G).

3. RESULTS AND DISCUSSION

3.1 Soil Indices and Classification

Tables 3, and 4, presents soil indices values, natural moisture content, in-situ bulk density , Atterberg limits, shear parameters for the different soils samples from different locations in the Uyo, metropolis.

The sieve analyses result from some of the tests samples are presented in Fig. 2. Percentage of soil passing through sieve no 200 (0.075 mm) is less than fifty percent putting all the soil sampled in the coarse grained range, but with some amount of clay or silt. Soil grain sizes range from medium to fine. Based on liquid limits value and plasticity index values, the soils were classified as SC (clayey sand), SM (silty sand), and the dual type, SC-SM (clayey sand – silty sand).

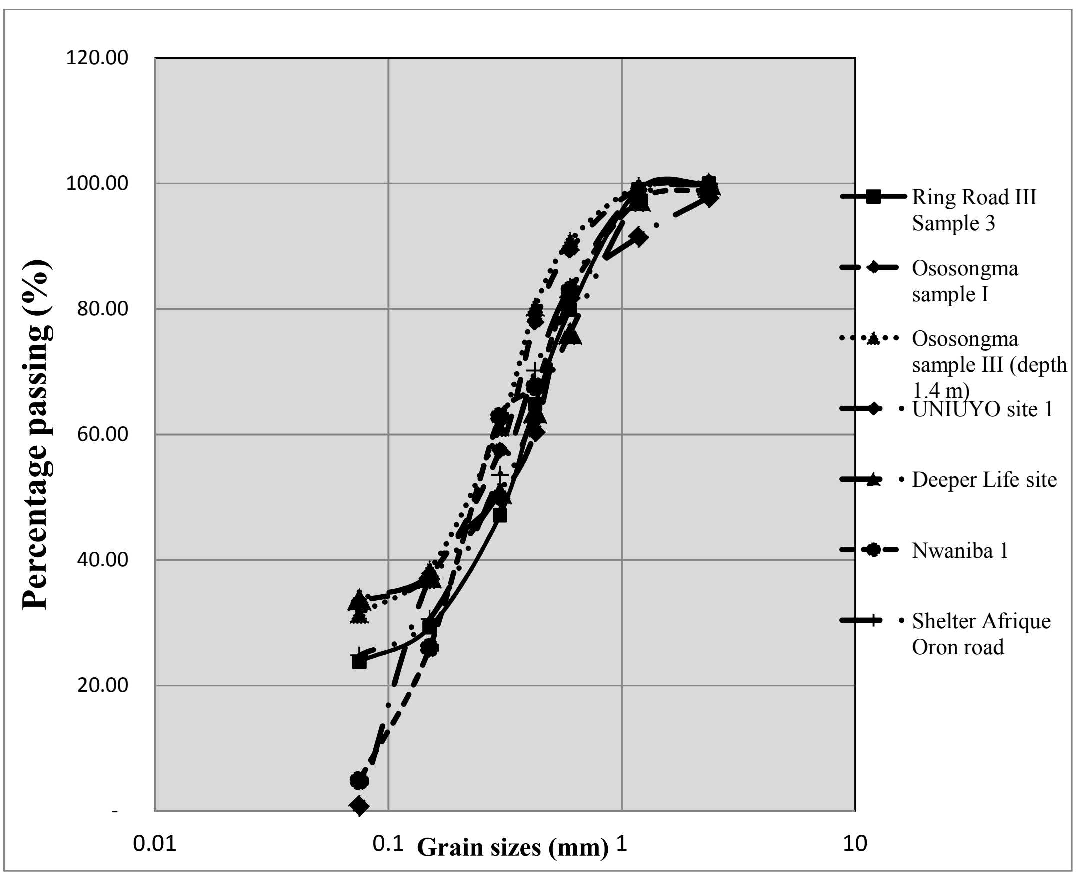


Fig. 2. Grain size analysis results of soils obtained from sites in study area

3.2 Shear Strength Values

The values of the angle of shearing resistance is between 8º to 31º, with about ninety percent less than 28º which represents the upper bound value of angle of shearing resistance for soils that will undergo local shear. The remaining ten percent of the soil falls within the mixed state or transition from local to general shear failure. Cohesive values of the soils tested is between zero (no cohesion) and value as high as 57 kN/m^2. Typical Direct shear box tests and analysis results are presented in Fig. 3 which shows shear stress versus horizontal displacement curve for Nwaniba I sample 1, while Fig. 4 presents determination of shear strength parameters from shear box for Ring road III sample 4 and Nwaniba I sample 1.

3.3 Soil Consistency

A classification based on relative densities or soil consistencies of some of the sites investigated with LRS 10 tests are presented in Table 5. The relative densities are determined at 10 cm thickness and are continuous to depth of investigation which is from ground surface to 6.0 m. For most of the site the “loose” soil consistency occurs within 0.0 m to 0.70 m. with exception of one test point in (Test No 1) in Shelter Afrique location in which the “loose” consistency goes up to 1.50 m. For the ‘Nickel and Dimes’ site “loose” consistency is dominant till 3.40 m depth before “medium” consistency soil is encountered which continues up to 6.0 m. Exception to this is at test No 1 where “medium” dense soil is encountered at 2.40 m depth. For all the other sites investigated, ’medium’ consistency dominate the subsurface up to 6.0 m.

For the CPT, the cone resistance values are plotted with depth. Figs. 5 and 6 presents the plots for CPT at test points 1 and 2 in the ‘Nwanniba IV’ (the Christian worship center), and at location 1 on the ‘Abak’ road site. Using the principle that “non- sharp peaks probably denotes soil of the same lithology”, the log signature for Nwaniba IV shows one sharp peak at 0.5 m depth, and no other one till refusal indicating same lithology from 0.50 m depth to refusal. However the soil profile can be divided into five layers based on values of cone resistances. The early part of the log shows high cone resistance values which is attributed partly to foundation construction works of the existing building on the site on but mainly to geology. A similar situation exists with respect to the CPT values in the ‘Abak road location. The log signature at location 1 here indicates five layers based on cone resistance values, while the Bank avenue log shows four layers, with the first layer having a high cone resistance. These are presented in Fig. 6.

The first layer on all the three sites where CPT data were acquired has a high cone resistance which decreases rapidly within one meter depth of the ground surface. While the Nwaniba IV site has the highest with a value of 170 kg/cm^2; all represent a “hard pan” on the soil surface, the Nwaniba IV showing the additional effect of construction and existing building load at the site as earlier stated.

On the three sites layer II is the foundation bearing stratum.

3.4 Lithology and SPT Boring

There is paucity of Standard penetration tests (SPT) data from the study area, since structures requiring deep foundation investigation (except bridges) are not very common in the area. Fig. 7 presents Standard penetration tests (SPT) log from two sites; the first from off Dominic Utuks street, the second from near the state water board site on Ikot- Ekpene road. For the 20 m bored depth for both SPTs, the SPT ‘N’ values is in the loose to medium dense range, with values from 6 to 25, except one, that is 36 at 20.0 m, which falls into the ‘dense’ consistency range. This is consistent with the results from LRS 10 test penetrometer within its depth of investigation which is 6.0 to 8.0 m.

From Fig. 7, SPT boring on both sites shows lithology of the area is made up of fine-medium grained sandy soil with silt or clay and sometimes fine gravel. Most of the soils from both sites classifies as SM, SC, SC-SM, SP-SM, SP based on Unified Soil Classification System (USCS). The poorly graded fine – medium sand (SP), terminates the lithology at 20 m on both sites. The soil classification also shows that the soil characterizing the 1.2 m to 2.0 m depth from which soil used for laboratory analyses were obtained continues significantly at depth.

3.5 Bearing Capacity

3.5.1 By direct shear test

From the values of C- Ø obtained from Direct shear test, computations of Ultimate bearing capacity was made using modified Terzaghi’s equation for local shear (equation 1), assuming a 2.0 m by 2.0 m footing. This was made for each of sample point at the different site location investigated within the Uyo metropolis. Ultimate bearing capacity values obtained ranged from 100.93 kN/m^2 to 490.72 kN/m^2. Using a factor of safety of 3, allowable bearing capacity for the soils is from 33.64 kN/m^2 to 163.57 kN/m^2.

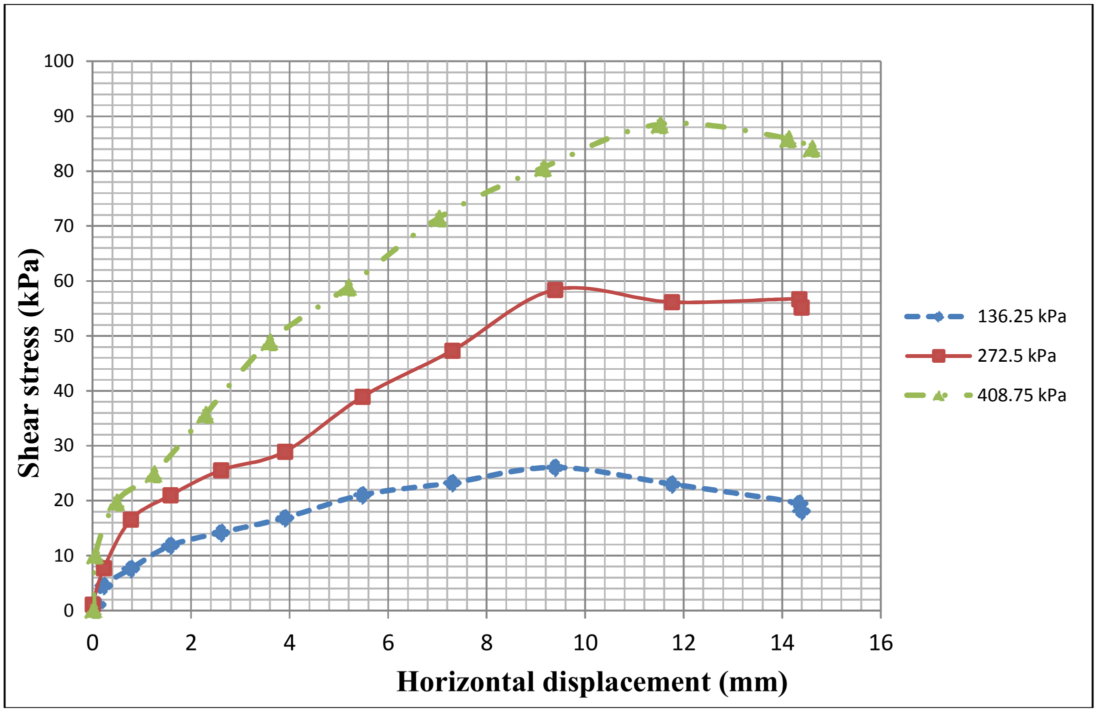


Fig. 3. Shear stress versus horizontal displacement curve for Nwaniba I Sample I

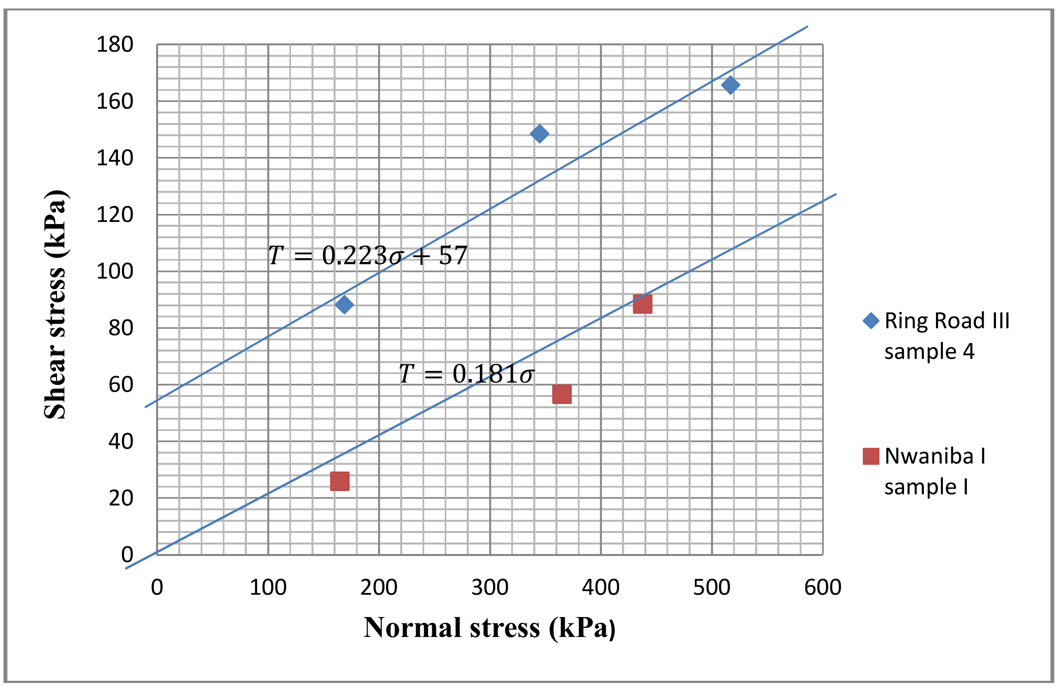
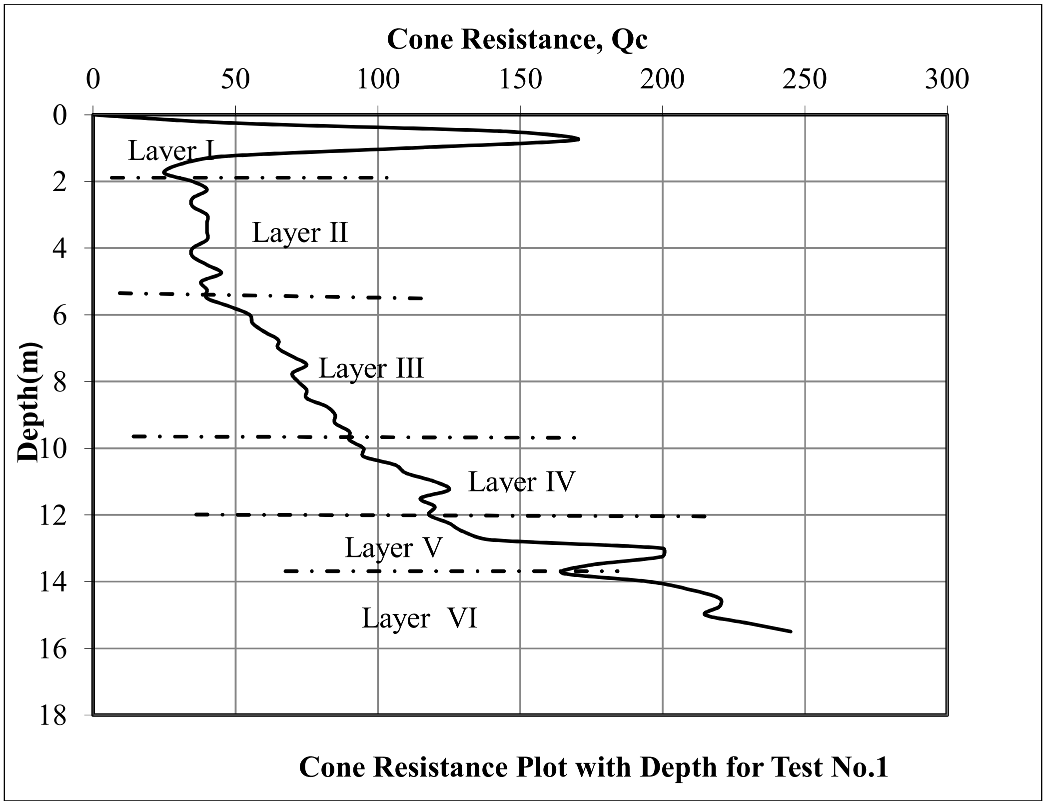


Fig. 4. Direct shear box test results for ring road III sample 4 and Nwaniba I sample I



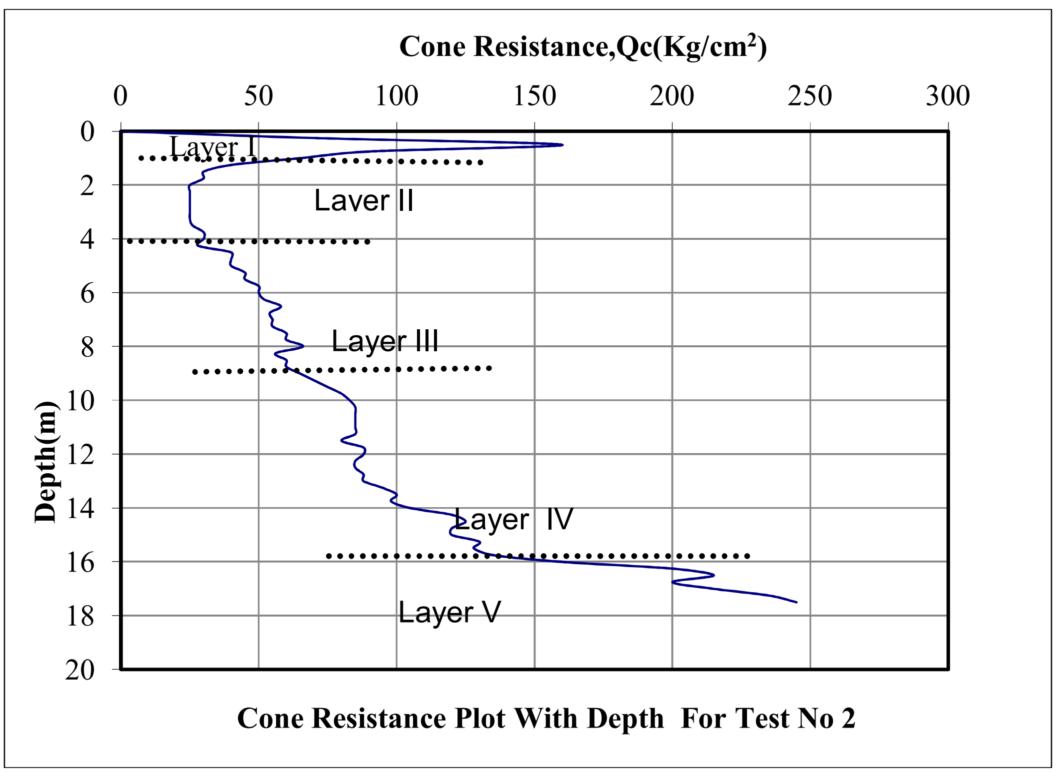
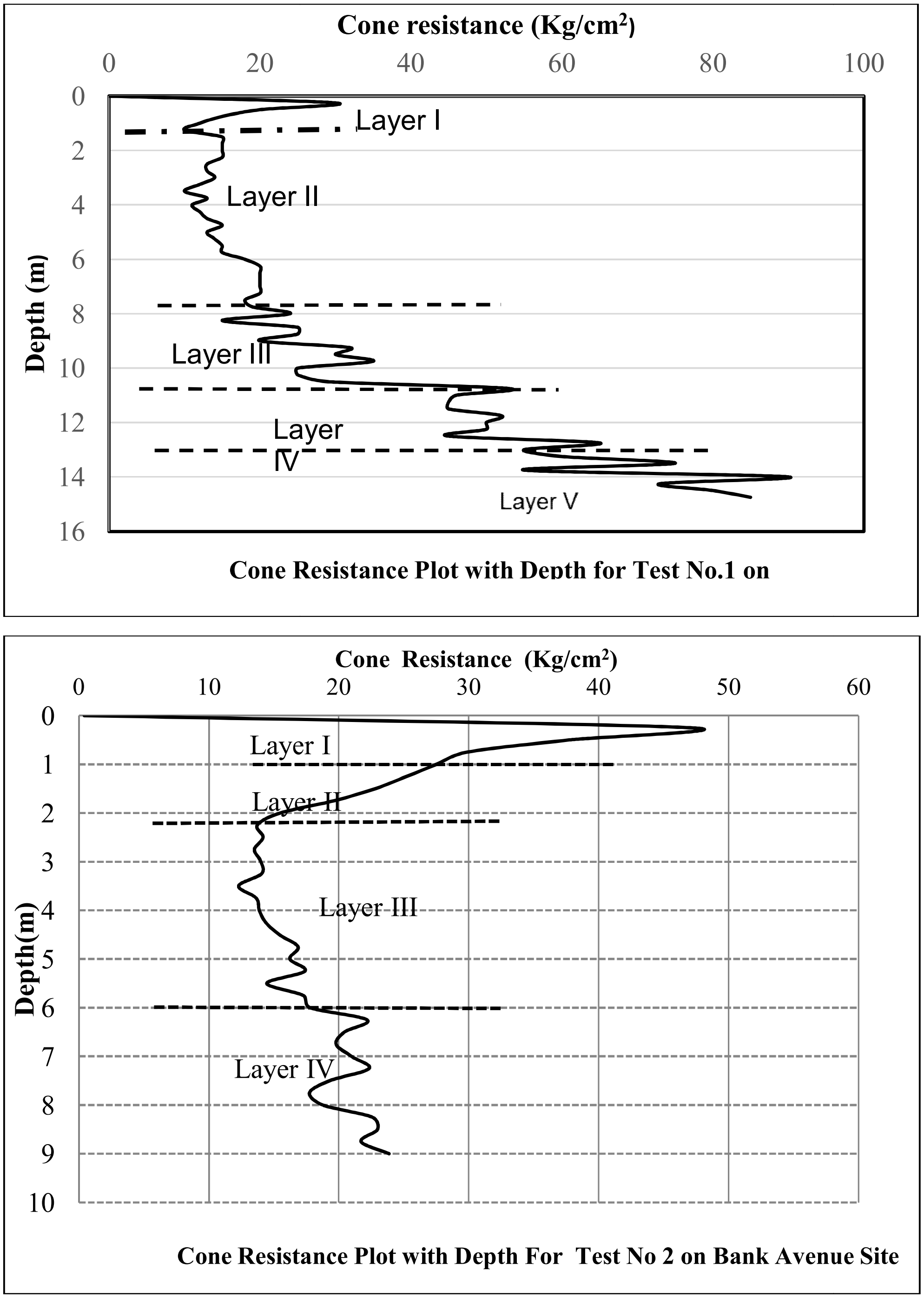


Fig. 5. Typical cone resistance pattern and soil layering at Nwaniba IV site location



Cone Resistance Plot with Depth for Test No.1 on

Cone Resistance Plot with Depth For Test No 2 on Bank Avenue Site

Fig. 6. Typical cone resistance pattern and soil stratification at Abak and Bank road sites

3.5.2 Estimates from LRS 10 tests, cone penetration test and standard penetration tests

The relative density values obtained were converted to equivalent cone resistance using the equation (5) above which was proposed by Kulhawy and Mayne [13] and equation (6) above proposed by Meyerhof [14] was used to estimate safe bearing pressure for all the LRS 10 test points, some values are displayed in Tables 3 and 4. Equation (6) was also used to estimate safe bearing pressure for cone penetration values obtained directly from the CPT. The safe bearing pressure values computed are all less than 100 kN/m^2.

Abak road site presents a unique check on the equivalent cone resistance calculated from the LRS 10 since the two equipment were deployed at this site. While 'CPT 1" at this location indicated a safe bearing capacity computed values of between 32.5 kN/m^2 and 40.5 kN/m^2 for the 1.0 m to 2.0 m depth ; for the same depth range CPT 2 at this location indicated values between 54 to 62.1 kN/m^2. The safe bearing pressure estimated from the equivalent cone resistance determined from LRS 10 test is between 44.57 kN/m^2 and 65.95kN/m^2. This is reasonably in close agreement with the values obtained from direct CPT. The values subsequently obtained from the LRS 10 can be said to be reliable.

The few SPT borings indicate that from the ground level up to 6.60 m depth, the 'N' value is from 6 to 11. This gives for a tolerable settlement of 25 mm an allowable bearing pressure of between 50 -100 kN/m2 for a 2.0 m width footing. This is based on chart by Terzaghi, et al. [18]. At within 2.0 to 3.0 m depth, the 'N' value is 6 to 7 which gives allowable bearing pressure value of 50 to 70 kN/m^2. These values are within the range of those estimated by both LRS 10 and CPT, except with respect to settlement.

3.6 Settlements

Both field and laboratory data were used in estimating likely settlement of structure assuming a 2.0 m by 2.0 m footing.

3.6.1 Estimation from laboratory data

The Direct shear box data was the laboratory data utilized in estimating settlement. Data from UNIUYO I, NWANIBA I, NWANIBA III, and Bank Avenue were utilized. Allowable bearing pressure values of each location were the stress change '$\Delta\sigma$'values used, with the compressibility values for the different normal stress developed during the Direct shear test. Table 6 presents the results of the computations. The table displays Ultimate and allowable pressures, normal stress and corresponding compressibility values and estimated settlements for each location.

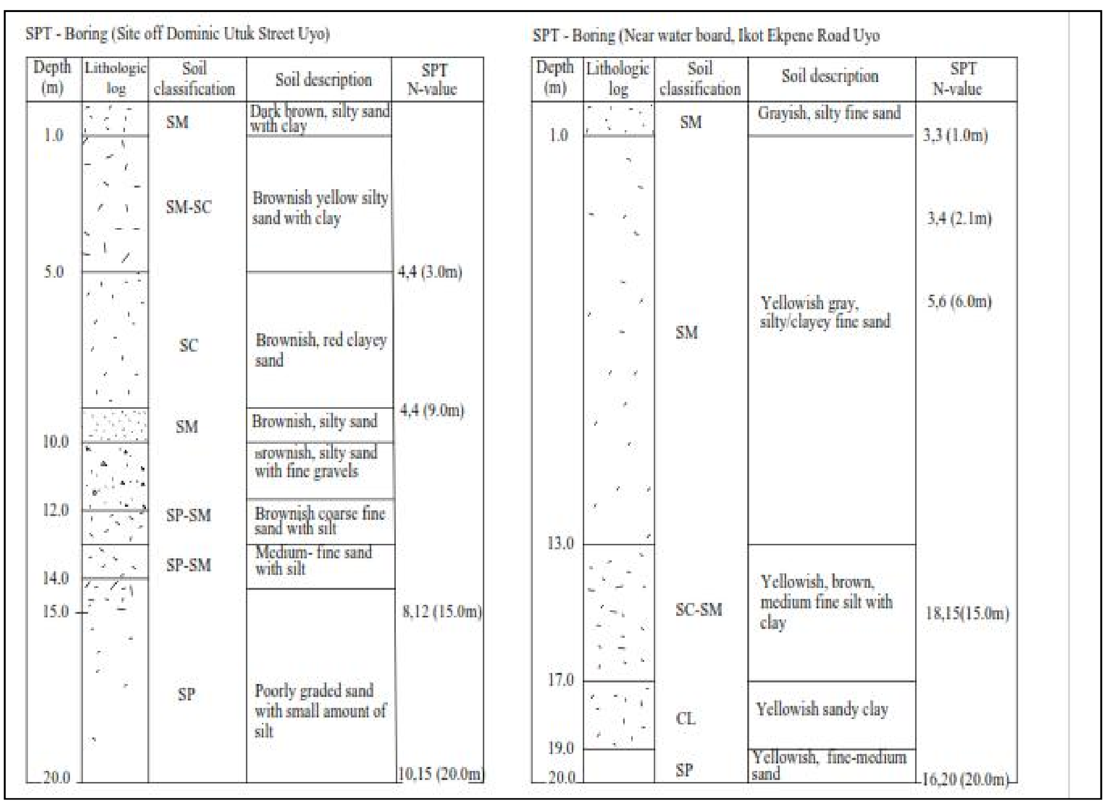


Fig. 7. Standard penetration test borings from two sites in the study area

During the early loading stage of the sample in a direct shear box experiment, the normal load is dominant and hence compressibility values estimated represents reasonable approximate to that from odeometer test. After that early stage of loading, the shear stress starts becoming significant, this causes a decrease in elastic modulus hence increase in compressibility values and hence settlement.

Values of compressibility for this early load stage ranges from 0.017 per MPa to 0.099 per MPa. Associated settlement ranges from 0.68 cm to 2.36 cm.

Another discernible trend from Table 6 is the general increase in compressibility values up to some maximum and then progressive decrease. A possible explanation for the trend is that in a direct shear experiment the shear stress developed at the second stage of normal load is usually higher than that developed at first stage of normal load, as stated above this leads to significant reduction in elastic modulus hence increase compressibility and then settlement. At higher normal load, shear stress developed though significant, but the associated normal stress becomes quite significant thereby offsetting the reduction in elastic modulus that is due to the shear stress, hence lower compressibility and lower settlement estimation.

For first two data presented in Table 6, the normal stress exceeds the estimated Ultimate Bearing Resistance. From the same Table, settlements computed for Nwaniba III shows excessive large values as the normal stress approaches the allowable bearing capacity, it goes from 1.457 cm at 137.32 KPa to 10.957 cm at 167.978 kPa. A similar trend is exhibited by the Bank Avenue site, in which settlement values goes from 1.52 cm at 139.42 kPa to 4.696 cm at 159.768 kPa. The allowable estimated for this location is 135.91 kPa. Normal stress is not up to the Ultimate therefore no statement can be made as regards trend in settlement values with respect to that value. Although similar trend of lower values of settlement at lower and higher normal stress and lager values at intermediate stresses is still exhibited. It follows from the above that settlement for design should be done at stress values less than the allowable.

At normal stress value from 140 kPa to 400 kPa (the intermediate stresses) the entire samples exhibit medium to high compressibility with values from 0.25 per MPa to 0.7 per MPa. At stresses higher than 400 kPa, M_v values is between 0.07 per MPa to 0.19 per MPa, which represents low compressibility. The degree of M_v values are based on [19] classification which is presented in Table 8.

The preceding suggest that if the soils are preloaded up to 400 kPa, before imposing a structure on the soils the accompanying settlement will be within allowable limits.

3.6.2 Settlement from field data

For the field method, settlement associated with safe bearing pressure computed from Meyerhof's formula using equivalent CPT values obtained from the LRS 10 test data and direct cone resistance data from CPT tests obtained at Abak Road, Power Chapel, and Bank Avenue were computed using equation 7. Table 7 presents the safe bearing pressures and associated settlement. Allowable bearing pressures computed from C- ∅ properties of the soil presented in the Tables 3 and 4 and associated settlement were also computed

For the few data used coefficients of volume compressibility, Mv estimated from equivalent CPT values compares well with that computed from direct CPT values for the Deeper life, and Tropicana sites with values of 0.178 and 0.070 per MPa respectively from equivalent CPT values and 0.18 and 0.077 per MPa for the sites from direct CPT. Settlements values are between 1.87 and 3.89 cm. For the Abak site one of the two data presented falls within the values computed for Mv and settlements while one is out of the range.

3.6.3 Laboratory versus field values

The settlements estimated for the field values were calculated with safe bearing pressure while allowable bearing pressure was used for laboratory calculations; the range of values of the latter (0.68 to 2.36 cm) falls well within that of the field values (1.87 to 3.89 cm). Similarly compressibility values from laboratory data at 0.017 per MPa to 0.099 per MPa are lower than the field values at 0.070 per MPa to 0.18 per MPa. The field values are conservative in both cases.

3.7 Compressibility Index, Shear Modulus and Bearing Capacity

Vesic [4] Equation for bearing capacity for soil that will fail in local shear takes cognizance of compressibility of the soil [5]. Presented an

Table 3. Tests locations, soil indices, shear parameters, and Ultimate bearing resistance

Site location	**Osongsoma**			**Ring Road III**			**Nwaniba I**		**Opposite breweries**		**Nwaniba II**	**Bank avenue**	**UNIUYO III (1000) seater auditorium**	**Nwaniba III**
Parameters	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 1**	**Sample 2**	**Sample 4**	**Sample 1**	**Sample 2**	**Sample 1**	**Sample 2**				
Unit weight (kN/m3)	16.5	16.5	16.2	16.7	16.2	16.5	16.5	16.4	17.41	15.21	15.21	17.6	16.71	16.71
Water content (%)	15.3	15.2	14.6	13.8	13.5	12.6	17.66	13.8	14.2	12.5	13.75	17.9	15	13.75
Liquid Limit (%)	36	36			21	21	27.5	25	40.6	35.5	21	42	35	20
Plastic Limit (%)	24	18.0			12	12	16.36	15	26.8	21.5	13	24	24	13
Plasticity Index (%)	12	18			9	8	11.14	10	13.8	14.0	12	18	11	7
Angle of shearing resistance, Ø (degree)	16	15	18	20	21	13	10.3	17.3	9.9	13.5	30.1	24.6	10.4	30.75
Cohesion, C (kN/m2	35	48.56	30	13	40	57	0	31	38	28	23.1	26.25	40.31	18.35
Depth (m)	1.2	1.2	1.4	1.2	1.2	1.2	1.2	1.2	2.20	2.0	2.0	1.2	1.2	1.2
Percentage passing sieve No 200		33			24		26	27	34.5	31.3	24	39.8	37.03	23
Soil Classification	SC	SC	X	X	SM	SM	SC-SM		SC	SC	SC-SM	SC	SC	SC-SM
Qu KN/m2 Terzaghi 's modified formula	331.43	414.82	320.9	204.60	445.91	445.24	42.47	321.46	313.93	274.70	571.1	407.73	293.83	433.39
Qu using Vesic compressibility factor	965.87													

Table 4. Further tests locations, soil indices, shear parameters, and Ultimate bearing resistance

Site location	Deeper life site (off Idoro road)	Tropicana hotel				Water board Ikot Ekpene road by Hospital			UNIUYO 1		UNIUYO 1I	Nwaniba IV (Power Chapel International Christian worship center)			Oron Road Shelter Afrique estate		
Parameters	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 1**	**Sample 2**	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 1**	**Sample 2**	**TP1**	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 1**	**Sample 2**	**Sample 3**
Bulk Unit weight (kN/m3)	16.77	16.98		19.1	19.32	15.000	15.000	15.000	16.72	16.81	16.5	24.3	24.1	24.0	18.41	17.44	17.42
Natural moisture content (%)	15.5	17.5	18.41	12	13.7	20.00	11.00	16.00	17.4		12	25.83	28.40	28.31	14.8	16.9	15.3
Liquid Limit (%)	51.8	55.3		28.4	27.7	13	12	12	38		35	25.43	33.00	36.93	34.2	40.3	33.7
Plastic Limit (%)	30.7	33.8		20.4	17.6	5	4	5	25.5		24	21.32	24.31	26.91	21.6	25.1	22.9
Plasticity Index (%)	21.1	21.5		8	10.1	8	8	7	12.5		11	4.11	8.69	10.02	12.6	15.2	10.8
Angle of shearing resistance (degrees)	14	14	14	8	15	16	17	13	20	20.7	9.7	X	X	X	14.1		4.3
Cohesion, C (kN/m2)	19	19	18.41	41	33	8	10	15	0	0	45.46				18		54
Percentage passing sieve No 200	37	33.8		19.6	25	24	17	23	25		27	23.0	30.0	30.7	24.8	35.8	28
Soil classification	SC	SC		SM	SC-SM	SC-SM	SM	SC-SM	SM		SC-SM	SC	SM	SM	SC	SC- SM	SC – SM
Depth(m)	1.4	1.6		1.2	1.5	1.2	1.2	1.5	1.2	1.2	1.2	1.5	1.5	2.0	1.5	1.7	1.5
Ultimate bearing capacity by Terzaghi modified formula. (kN/m^2)	204.16	212.42		281.51	321.01	118.74	144.43	146.33	100.93	100.93	337.85	X	X	X	201.02		311.961
Ultimate bearing capacity using Vesic compressibility factor. . (kN/m^2)									255.62	277.99							
Safe bearing pressure based on Meyerhof's equation(kN/m^2)	45.31	48.74		63.41	76.15							81	81	94.5	48.832		
Allowable bearing capacity (kN/m^2)	68.05	70.81		93.84	107.00	39.58	48.11	48.77	33.64	33.64	112.61				67.006		103.987

Table 5. Classification of soil consistencies with depth indicating relative densities at some test points in some of the sites in Uyo metropolis

Site	Afrique shelter. Test No. 1		Tropicana Hotel Test No 2		Abak Test No 4	Nickel and Dimes Hotel Test No 4
Soil Layer	I	II	1	II	I	I
Depth range(m)	0 - 1.80	1.90 - 6.0	0.0 - 0. 30	0.4 - 5.20	0.0 -6.0	0.0 -6.0
Blows per 10cm	2 - 4	5 -12	2	5-15	5 - 13	1 - 3
Relative density range	0.279 - 0.348	0.348 - 0.458	0.2729 -0.270	0.371-0.4805	0.371 -0.466	0.210 - 0.320
Soil Consistency	Loose	Medium	Loose	Medium	Medium	Loose

Table 6. Normal stress, constrained modulus and settlement computations, using data from Direct shear box test

Sample location	UNIUYO 1		NWANIBA I			NWANIBA III			Bank avenue		
Ultimate bearing capacity (kN/m^2)	100.930		42.469			433.391			407.730		
Allowable bearing stress (kN/m^2)	33.330		14.117			146.600			135.910		
*Vertical stress (δv) (kpa)	Reciprocal of constrained modulus (per MPa)	Settlement (cm)	Vertical stress (δv) (kpa)	Reciprocal of constrained modulus(per MPa)	Settlement (cm)	Vertical stress (δv) (kpa)	Reciprocal of constrained modulus (per MPa)	Settlement (cm)	Vertical stress (δv) (kpa)	Reciprocal of constrained modulus(per MPa)	Settlement (cm)
136.102	0.301	2.980	138.931	0.409	5.221	136.259	0.012	0.516	27.250	0.033	1.347
136.197	0.034	0.341	139.379	0.568	7.246	136.268	0.012	0.549	27.383	0.531	21.665
160.508	0.719	7.115	140.526	0.577	7.363	137.321	0.033	1.457	27.674	0.083	3.389
168.371	0.695	2.980	172.329	0.554	7.067	167.978	0.249	10.957	28.078	0.146	5.954
169.722	0.689	2.980	182.265	0.530	6.762	177.521	0.241	10.579	33.572	0.250	10.193
			182.801	0.532	6.788	179.257	0.240	10.579	34.368	0.156	6.344
									34.529	0.125	5.117
273.598	0.053	0.529	285.251	0.276	3.516	272.645	0.231	10.146	34.974	0.122	4.995
274.885	0.100	0.988	286.216	0.309	3.943	272.686	0.240	10.548			
312.250	0.227	2.242	286.630	0.323	4.124	273.608	0.245	10.786	68.132	0.017	0.688
325.160	0.225	2.223	287.962	0.368	4.701	333.408	0.340	14.959	68.132	0.044	1.778
329.953			288.047	0.371	4.737	352.135	0.324	14.238	68.790	0.176	7.160
			288.131	0.374	4.774	358.820	0.326	14.340	69.757	0.246	10.037
					0.000						
418.053	0.071	0.702	422.112	0.092	1.170	416.896	0.117	5.153	86.172	0.570	23.232
418.852	0.099	0.982	425.307	0.143	1.828	417.098	0.120	5.262	88.176	0.570	23.236
419.808	0.113	1.117	428.809	0.198	2.528	420.260	0.133	5.839	89.366	0.691	28.173
512.894	0.140	1.387	437.438	0.326	4.155	513.052	0.199	8.744	92.314	0.790	32.198
542.099	0.137	1.352	438.120	0.335	4.278	541.419	0.193	8.476			
555.198	0.137	1.360	438.227	0.337	4.297	562.518	0.190	8.342			
									137.323	0.001	0.059
									137.991	0.016	0.635

*Vertical stress developed during different normal load	139.422	0.037	1.521
placement on sample during a Direct shear test	141.416	0.058	2.364
experiment. The loads were 50 kg,100 kg, and 150 kg			
except for Bank Avenue in which the Normal loads were	159.768	0.115	4.696
10 kg, 25 kg and 50 kg.	161.689	0.118	4.816
	164.789	0.124	5.047
	168.165	0.164	6.668

Table 7. Settlement values computed based on various bearing capacities from equivalent CPT values derived from LRS and from direct CPT values

	Data from equivalent CPT values from LRS 10 data			**From direct CPT values**	
Site Location	**Test 5**		**Settlements (cm)**	**Bank Avenue site**	
				CPT Test 1	**CPT Test 2**
Deeper life site on Idoro road	E (MPa)	22.30		5.23	5.72
	Modulus of volume compressibility, (M_v) per MPa	0.178		0.2	0.18
	Safe bearing pressure (kN/m2)	45	2.41	63.7	74.3
	*Allowable bearing capacity (kN/m^2)	70.81	3.79		
	*Ditto	68.05	3.64	Settlement(cm)	
				3.66	3.89
Tropicana	**TEST No. 2**		**Settlements (cm)**	**Nwaniba road IV site**	
				CPT Test 1	
	E (MPa)	14.20		12.92	Settlement(cm)
	Modulus of volume compressibility, (M_v) per MPa	0.070		0.077	
	Safe bearing pressure (kN/m2)	75.71	1.60	67.5	1.56
	*Allowable bearing capacity (kN/m^2)	107	2.26		
	*Ditto	93	1.96		
	TEST No.6			**CPT Test 2**	
	E (MPa)	13.06		10.592	
	Modulus of volume compressibility, (M_v) per MPa	0.076		0.094	
	Safe bearing pressure (kN/m2)	81.32	1.87	94	2.66
Abak	**TEST No. 1**		**Settlements (cm)**	**CPT Test 1**	
	E (MPa)	45.65		5.071	Settlement(cm)
	Modulus of volume compressibility, (M_v) per MPa	0.022		0.2	
	Safe bearing pressure (kN/m2)	53.164	0.35	40.5	2.40
	TEST No.7			**CPT Test 2**	
	E (MPa)	46.21		6.10	
	Modulus of volume compressibility, (M_v) per MPa	0.076		0.164	
	Safe bearing pressure (kN/m2)	56.898	1.302	94	2.79

	Data from equivalent CPT values from LRS 10 data			From direct CPT values
Oron Road Shelter Afrique estate	Test 4		Settlements (cm)	
	E (MPa)	12.216		
	Modulus of volume compressibility, (M_v) per MPa	0.081		
	Safe bearing pressure (kN/m2)	67.00	1.65	
	*Allowable bearing capacity (kN/m^2)	103.98	2.55	
	*Ditto	51.4	1.26	

* Estimates from C-Ø values obtained from Direct shear box tests

Table 8. Typical values of compressibility for clays[1]

Soil description	Compressibility	M_v (m^2 MN) $^{-1}$	Cc
Heavily over consolidated clays	very low	<0.05	<0.10
Very stiff to hard clays	Low	0.05 - 0.10	0.10 - 0.25
Medium clays	Medium	0.10 - 0.30	0.25-0.80
Normally Consolidated clays	High	0.30-1.5	0.80 - 2.50
Very organic clays and peats	very high	>1.5	>2.50

M_v = Coefficient Of Volume Compressibility; Cc = Coefficient Of Compression

1. By [19]

example with ultimate bearing capacity value (Q_u) by Vesic method (equation 3) of 548 kN/m^2. Q_u by modified Terzaghi's formula (equation 2) for the same problem gives a value of 536 kN/m^2, leading to a difference of 12 kN/m^2. Maintaining the same depth of 0.6 m but changing the width from 0.6 m to 1.2 m with all other parameters remaining the same, gives bearing capacity of 683.52 kN/m^2 by Vesic method and 592.84 kN/m^2 by the modified formula. This represents a reasonable discrepancy of 90.68 kN/m^2.

Another parameter in Vesic's equation (equation 3) that is very difficult to determine is the shear modulus (G).Various values are determined from the direct shear box experiment with different vertical loads. A choice of value of shear modulus with a normal load that approximates the estimated ultimate bearing capacity by the modified equation was used. Uniuyo 1 in Table 4 presents both results for one location. The Q_u obtained by the Vesic equation is far higher than that by the modified formula. Therefore Terzaghi modified equation offers a conservative but reasonable estimate of Ultimate bearing capacity for soils that will fail in local shear. The value given by Terzaghi's modified formula can be taken as the *lower bound value* for bearing capacity for soil that will fail in local shear

3.8 Ultimate Bearing Capacity, Footing Size, and Soil C- ∅ Properties

At the same depth, on the same soil, a square footing has higher ultimate bearing capacity Q_u, than a rectangular footing. For example for a soil with C=23.1 kN/m^2 , $\emptyset = 30.1°$, and $\gamma = 15.21$ kN/m^3, depth of 1.4; a 2.0 m by 2.0 m footing have a Q_u value of 506.6 kN/m^2, while a 2.0 m by 3.0 m footing have a Q_u value of 485.22 kN/m^2. A 3.0 m by 3.0 m still gives a value of 506.6 kN/m^2. Instinctively increasing a footing size to be able to carry more load on a given soil should be done with a check on the Ultimate bearing capacity of the soil which depends strongly on the soil properties C, ∅, and γ. As the example above indicates, the Ultimate bearing capacity take on a limiting value of the soil property with respect to loads placed on it. Assuming a factor of safety of 3, the allowable bearing capacity for the soil is 168.86 kN/m^2. A 3.0 m by 3.0 m footing will sustain a load of 1519.74 kN. Assuming a 25 grade concrete, such a footing based on Eurocode 2 (2004) [20] will be about 50 cm thick to sustain the load against punching shear. The volume of concrete with associated reinforcement for this kind of footing makes it an unusual construction for two and three storey buildings commonly constructed in the study area. Furthermore the settlement associated with the allowable bearing capacity (values closed to 168.86 kN/m^2) and associated compressibility values in Table 7 gives settlement of 2.98 cm to 10.58 cm.

4. CONCLUSION

For the Coastal Plain Sands soils that dominated Uyo metropolis lithology which was covered in this study, the soil within the 0.0 m to 6.0 m in the shallow subsurface are mostly "medium" dense in consistency with 'loosely dense' soil occurring at some location. Due to the consistency of the soils, and the values of angle of shearing resistance that the soils in this locality possess, the soil will fail in local shear thereby reducing the ultimate bearing capacity of the soil.

Values for ultimate bearing capacity determined ranges from 100 kN/m^2 to 571.1 kN/m^2 as determined using $C - \emptyset$ of the soil obtained from laboratory test samples. Associated settlement estimate using the laboratory parameters is between 2.98 cm to 10.58 cm.

Safe bearing pressure ranges from 45 kN/m^2 to 94 kN/m^2 with associated field estimated settlement of between 1.60 cm to 3.89 cm. Settlements estimate using safe bearing pressure are within generally accepted tolerable values; whereas that of allowable bearing capacity are larger. Based on the above, shallow foundation design for the study area should be based on safe bearing pressure.

Terzaghi's modified equation for local shear failure is adequate to estimate the Ultimate bearing capacity of the soils in the study area; Vesic's method may overestimate the bearing capacity.

It is possible to obtain some parameters from direct shear box tests that can be used to reliably estimate settlements for the soil in question. Such parameters as constrained elastic modulus and its inverse (volume compressibility) are obtained from which estimates of settlements can be made. Settlements estimate from this should be compared to that from field values, and as a check is generally smaller.

The soils in the area can be preloaded up to a stress of 400 kPa before placing construction load on it. This will limit settlement to within tolerable limits.

At a particular depth, increasing the size of footing placed on the soils in this study area will not necessarily increase the load carrying capacity of such a footing; the bearing capacity and associated settlement must be evaluated for such situation.

The relationship by Lunne [15] that is used to estimate soil modulus from CPT data in this study results in estimated settlement from field data not widely different to that from laboratory values;, this suggest that it is a reliable relationship to be employed for estimating modulus in the Coastal Plain Sands of Uyo.

COMPETING INTERESTS

Authors have declared that no competing interests exist.

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APPENDIX

Vesic compressibility Equations

Vesic proposed the estimation of the Ultimate bearing capacity from the equation (1) below

$$q_u = C' N_c F_{cs} F_{cd} F_{cc} + q N_q F_{qs} F_{qd} F_{qc} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma c} \quad (1)$$

In this equation, F_{cc}, F_{qc}, and $F_{\gamma c}$ are soil compressibility factors.

The soil compressibility factors were derived by Vesic [4] by analogy to the expansion of cavities. According to that theory, in order to calculate F_{cc}, F_{qc}, and $F_{\gamma c}$, the following steps should be taken:

Step 1. Calculate the *rigidity index,* I_r, of the soil at a depth approximately B/2 below the bottom of the foundation, or

$$I_r = \frac{G_S}{C' + q' \tan \emptyset'} \quad (2)$$

where

G_s = shear modulus of the soil
q´ = effective overburden pressure at a depth D_f + B/2

Step 2. The critical rigidity index, $I_{r(cr)}$, can be expressed as

$$I_{r(cr)} = \frac{1}{2}\left\{exp\left[\left(3.30 - 0.45\frac{B}{L}\right) Cot\left(45 - \frac{\emptyset'}{2}\right)\right]\right\} \quad (3)$$

The variation of $I_{r(cr)}$ with B/L is given in a Table or can be calculated with equation (3)

Step 3. If $I_r \geq I_{r(cr)}$, then

$$F_{cc} = F_{qc} = F_{\gamma c} = 1$$

However, if $I_r < I_{r(cr)}$, then

$$F_{\gamma c} = F_{qc} = \exp\left\{\left(-4.4 + 0.6\frac{B}{L}\right)\tan\phi' + \left[\frac{(3.07\sin\phi')(\log 2I_r)}{1+\sin\phi'}\right]\right\}$$

$$F_{cc} = 0.32 + 0.12\frac{B}{L} + 0.60\log I_r \quad (5)$$

$$F_{cc} = F_{qc} - \frac{1 - F_{qc}}{N_c \tan\emptyset'} \quad (6)$$

Shape and Depth factors used in the computations

Shape Factors by De Beer [21]

$$F_C = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right).$$ (7)

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi'$$ (8)

$$F_C = 1 - 0.4\left(\frac{B}{L}\right)$$ (9)

Depth Factors by Hansen [22]

$$\frac{D_f}{B} \leq 1$$

For $\phi' > 0$

$$F_{cd} = F_{qd} - \frac{1-F_{qd}}{N_c \tan\phi'},$$ (10)

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right)$$ (11)

$$F_{\gamma d} = 1$$ (12)

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