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Geotechnical Investigation for Design and Construction of Civil Infrastructures in Parts of Port Harcourt City of Rivers State, Southern Nigeria.

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ABSTRACT

Before construction activities could begin at any site, engineering geological and geotechnical investigations has to be approved in order to determine the safe bearing capacity of the soil materials and recommend suitable foundation for the structure. In this study, geotechnical investigation for the design and construction of civil infrastructures in parts of Port Harcourt city of Rivers State, Nigeria has been carried out. Fourteen samples of sand and clay from different locations within the Afam Clay Member and Benin Formation were assessed. The Atterberg limit results of the clay samples revealed that the materials are of relatively medium to high compressibility, with the liquid limit (LL) ranging from 28% to 71%, plastic limit (PL) ranging from 10% to 21%, while the plasticity index (PI) ranges from 15 to 54, indicative of medium to low compressive strength. The natural moisture content, with a mean value of 38% was also significantly high, while the unit weight ranged from 15.6KN/m^3 to 18.7KN/m^3, and specific gravity (SG) values range from 2.27 to 2.72. The sand samples had coefficient of uniformity (Cu) and coefficient of curvature (Cc) values ranging from 2.52 to 5.2 and 0.99 to 1.8 respectively, indicating that the sands are poorly graded and are classified as SP. The in-situ standard penetration test (SPT) on the sand samples has N-values ranging from 20 to 28, showing that the sands are medium dense. The clay samples underlying the study area is likely to have medium to low shear strength as suggested by the values of the strength parameters (mean value of angle of internal friction is 6^0 and cohesion is 46Kpa), obtained from the triaxial test. The geotechnical behavior of the materials within the study area shows that the cohesive materials failed some relevant material specifications for most civil infrastructures, having ultimate and safe bearing capacity averaging 410.48KN/m^2 and 136.83KN/m^3 respectively. Thus, they should be avoided as foundation (load bearing) materials during civil constructions, while the cohesion less soil though, of medium dense and poorly graded will serve as better load bearing materials.

Keywords **-** *Triaxial test, angle of internal friction, Standard Penetration Test, Ultimate Load bearing capacity, safe bearing capacity*

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I. INTRODUCTION

The quality of a nation's infrastructure is a critical index of its economic vitality (Tsatsanifos, 2008). To the structural engineer, earth materials (soil and rock) which form a larger portion of the construction materials; basically form the foundation for their structure.Unlike manufactured materials, the engineering properties of soil and rocks are the result of the natural processes that have formed them, and natural or man-made events following their formation and deposition. Variation in the properties of these earth materials coupled with the type of structure resting upon them usually results to a geotechnical failure and geologic hazard when improper or no geotechnical investigation is carried out to determine the engineering properties of the soil.In this work, we have studied the geotechnical properties of the underlying soil (sand and clay) materials within the Port Harcourt city of Rivers State, southern Nigeria, for the design and construction of civil infrastructures.

The investigation intends mainly to look at the clay member within the Benin Formation, as well as the sandy unit and determine their suitability with respect to load bearing, as soft rocks and expansive soils are most often associated with non-durability (Gamble, 1971; Ezeribe, 1994), foundation problems and structural failures (Holtz and Kovacs, 1982; Coduto, 1999; Punmia *et al.*, 2005).

II. GEOLOGICAL SETTING

The study area is located within the Port Harcourt city of Rivers State, southern Nigeria (Fig 1), and falls within the Benin Formation deposited during the tertiary in the Niger Delta basin (Figure 2). The formation of the Niger Delta basin is linked to the development of the Benue Trough as a failed arm of a rift triple junction associated with the separation of Africa and South American plates, and the subsequent opening of the south Atlantic (Allen, 1965; Oomkens, 1974).

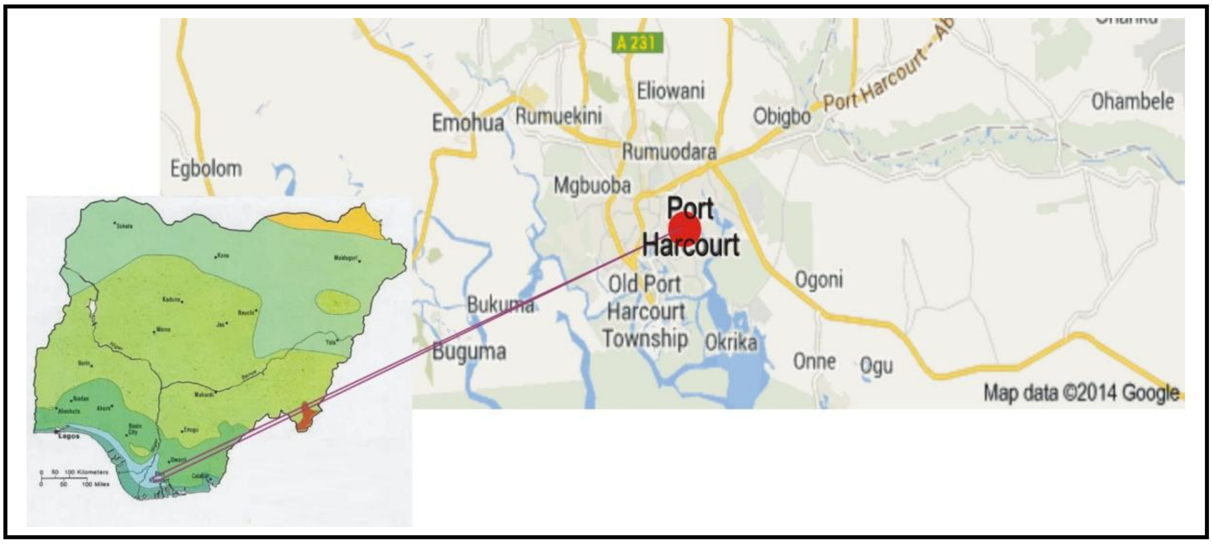

Fig 1: Location map of the study area

Rifting within the Benue Trough started in the Late Jurassic and persisted into the middle Cretaceous (Leiner and De Ruiter, 1977). However, in the region of the Niger Delta, rifting diminished in the Late Cretaceous. The coastal sedimentary basin of Nigeria has been the scene of three depositional cycles (Short and Stauble, 1967). The first began with a marine incursion in the middle Cretaceous and was terminated by a mild folding phase in Santonian time. The second included the growth of a proto – Niger Delta during the Late Cretaceous and ended in a major Paleocene marine transgression. The third cycle, from Eocene to Recent, marked the continuous growth of the main Niger Delta.

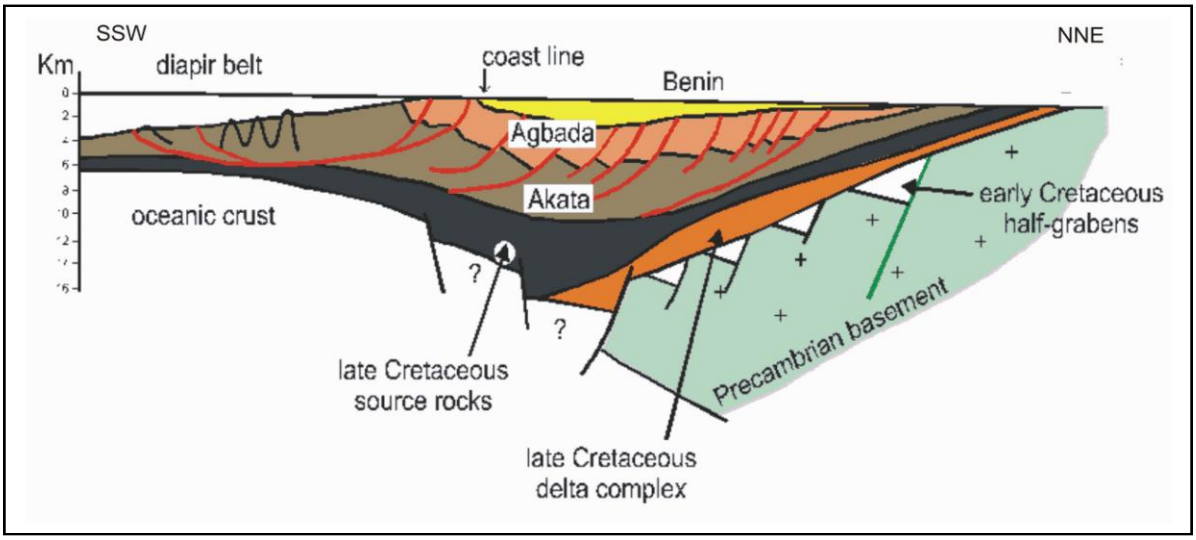

Fig 2: Average cross section through the Niger Delta (*after* Thomas, 1995)

Table 1: Generalized stratigraphy of the Niger Delta Basin (modified from Doust & Omatsola, 1990)

SUBSURFACE			SURFACE EQUIVALENT		
YOUNGEST KNOWN AGE		OLDEST KNOWN AGE	YOUNGEST KNOWN AGE		OLDEST KNOWN AGE
RECENT	**BENIN** Afam Clay Member	OLIGOCENE	PLIO/ PLEISTOCENE	**BENIN FORMATION**	MIOCENE?
RECENT	**AGBADA FORMATION**	EOCENE	MIOCENE EOCENE	**OGWUASHI-ASABA FM AMEKI GROUP**	OLIGOCENE EOCENE
RECENT	**AKATA FORMATION**	EOCENE	L. EOCENE	**IMO SHALE**	PALEOCENE

The Tertiary section of the Niger Delta is divided into three lithostratigrahic units representing prograding depositional facies that are distinguished mostly on the basis of sand and shale ratios (Table 1). The type section of these formations are described in Short and Stauble (1967), and summarized in a variety of papers. The Akata Formation at the base of the delta is of Marine origin and is composed of thick shale sequence, turbidite sand, and minor amounts of clay and silt. It is overlain by the Agbada Formation; a paralic siliciclastic formation, over 3700m thick and represents the actual deltaic portion of the sequence. The Agbada Formation is overlain by the third formation, the Benin Formation, a continental latest Eocene to Recent deposit of alluvial and upper coastal plain sands that are up to 2000m thick (Avbovbo, 1978). The three units extend across the whole delta, and each ranges in age from early Tertiary to Recent. A separate member of the Benin Formation is recognized in the Port Harcourt area; this is the Afam Clay Member, which is interpreted to be an ancient valley fill formed in Miocene sediments (Short and Stauble, 1967).

III. PHYSIOGRAPHY AND CLIMATE

The land surface of River state can be grouped into three main divisions; the fresh water, the mangrove swamps, and the coastal sand ridges. The freshwater zone extends north ward from the mangrove swamp, with the land surface generally under 20m above sea level. The Port Harcourt area falls within the upland with varying heights between 13 to 45m above sea level. The drier upland area of the state covers about sixty one percent, while the riverine area, with a relief of 2m to 5m, covers about thirty nine per cent of the state. The entire topography is characterized by maze of effluents, rivers, lakes, creeks, lagoons and swamps crisscrossing the low lying plains in varying dimension.Drainage within the state is poor, being low lying, with much surface water and a high rainfall. The state is drained by two main river systems, i.e. freshwater systems whose waters originate either outside or wholly within the coastal lowlands, and tidal systems confined largely to the lower half of the state.Port Harcourt area features a tropical monsoon climate with lengthy and heavy rainy seasons (for the remainder of the year excluding the two dry months), and very short dry seasons occurring at the months of December and January, with December on the average being the driest month of the year, having an average rainfall of 20mm. The heaviest precipitation occurs during the month of September with an average of 370mm rain. Temperatures throughout the year in the city are relatively constant, showing little variation throughout the course of the year. Average temperatures are typically between 25^0c – 28^0c.

IV. METHOD OF STUDY

A total of fourteen soil samples (clay and sand) from 3 locations within the Port Harcourt city of Rivers State were collected and tested. Deep boring was done in two locations (Woji and Trans-Amadi) to a depth of 15m using a Percussion boring rig with the aid of augers, clay cutter or shell, while the hand auger was used in boring the 3rd location (Rumuolumeni) to a depth of 6m.Undisturbed samples were collected using the conventional open tube sampler, which measures about 76mm in diameter and 450mm long.The sampling strictly followed standard procedure for soil sampling specified in British Standard Institution (BSI) 1377 (1990).Standard penetration tests (SPT) were performed on non-cohesive soil formation. The test was used to assess the relative density of the soil. In the test, a 50mm diameter spoon sampler was driven 450mm into the soil with a 63.5Kg hammer falling through a height of 760mm. The initial 150mm penetration is the test drive. The number of blows required to effect the remaining 300mm penetration was recorded as the SPT (N) value.

Series of classification and mechanical property tests were carried out on these samples. They include; the Atterberg limit test (Liquid and Plastic limit), specific gravity, unit weight, natural moisture content, particle size analysis, and triaxial shear strength test. All the tests followed standard procedures of testing soils for civil engineering purposes. Specific gravity tests were determined using a 50ml density bottle with a known weight. Oven dried sample and the bottle with the stopper is weighed, the mass of the bottle, soil and water is taken, then the mass of bottle and water is recorded. The ratio of the dry mass of soil to the mass of equal volume water was computed as the specific gravity of the sample. Reference test standard: Euro code 7 - EN 1997-2:2007.

The natural moisture content was determined following the simple method outlined by Akroyd (1957), while the Atterberg limit test was done on soil samples with particle size of less than 0.425mm using the testing procedure defined in ASTM Standard D 4318.
The unit weight test was done on an undisturbed sample using a cylinder of specified height and diameter to obtain the measurement of mass and volume of the soil. While the particle size analysis was carried out using a standard sieve to shake the dry sand samples for several minutes. A plot of the sieve size against % passing was made to determine the grain distribution and grading. The general slope of the distribution curve (Figs 6 &7) was described by the coefficient of uniformity C_u, where $C_u = D_{60}/D_{10}$, and the coefficient of curvature C_c, where $C_c = (D_{30})^2/D_{10}$ x D_{60}.D_{60}, D_{30} and D_{10} are effective particle sizes indicating that 60%, 30% and 10% respectively of the particles (by weight) are smaller than the given effective size. Reference test standard: Euro code 7 - EN 1997-2:2007.

Unconsolidated undrained triaxial shear strengths tests were carried out on the clay specimens (38 mm diameters and 76mm height) using Multiplex 50, ELE International type of triaxial equipment. In this test, the soil specimen was enclosed in a rubber membrane and placed in a triaxial cell filled with water. A cell pressure is applied which simulates the in-situ stress on the specimen. The specimen is then loaded to failure with no drainage from the sample. The test result is then presented with a plot of mohr circle for failure envelopes using various confining pressures and maximum deviator stress at failure. The strength parameters obtained from this plot is then used in the bearing capacity calculation.

V. RESULTS

5.1 Soil Stratigraphy

The information obtained from the boring and soil sampling, as well as the field penetration tests has allowed for the interpretation of the underlying soil stratigraphy within the studied locations. The stratum of the soil encountered across the sites and the borehole logs are shown in Figs 3, 4, & 5.

5.2 Engineering properties of the underlying soils

The investigation has revealed that within the explored depths, the strata succession encountered at the sites are significantly consistent. In general, a layer of soft to firm brown sandy clay and a thick layer of medium dense sand were encountered up to the depth explored.
The relevant engineering parameters of these clays and sands for the investigated locations are shown in Tables 2, 3, and 4 respectively.

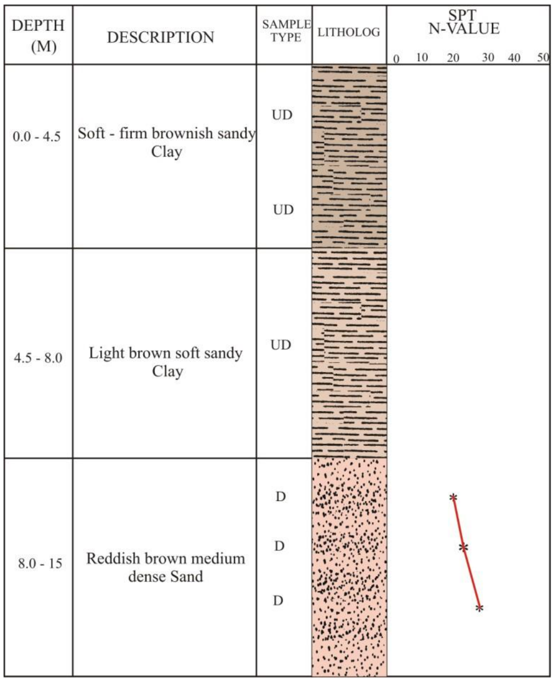

Fig 3: Borehole log at Trans Amadi Location

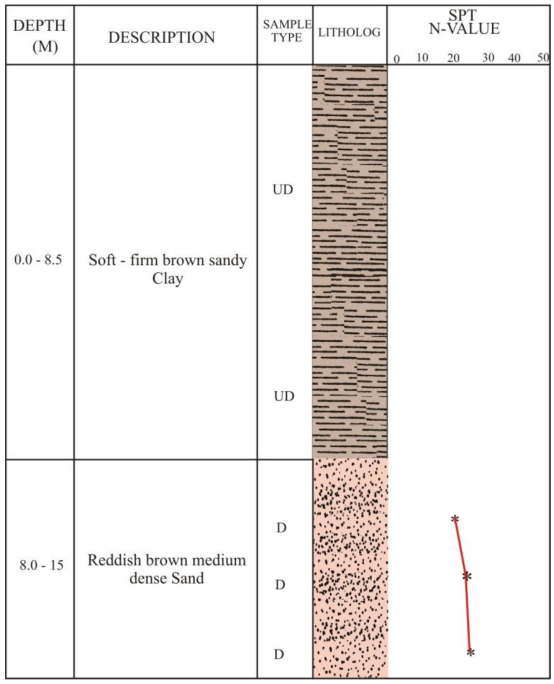

Fig 4: Borehole log at Woji Location

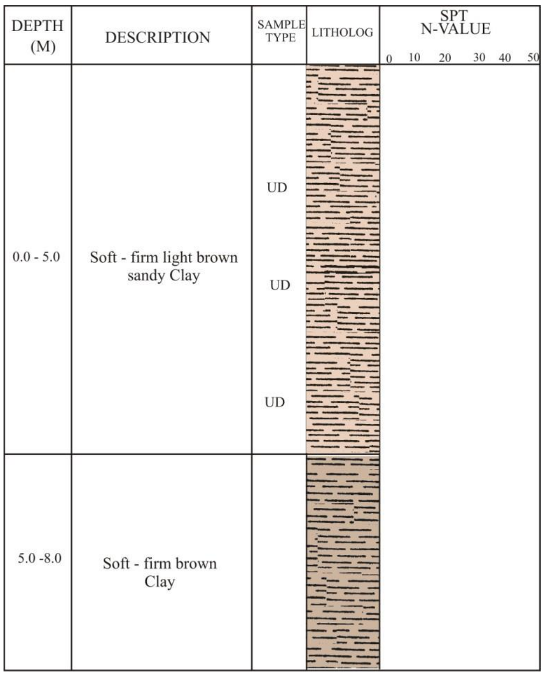

Fig 5: Borehole log at Rumuolumeni Location

Table 2: Engineering properties of the underlying soil at Trans Amadi Location

Engineering Properties	Min	Max	Mean
Natural moisture content (%)	15	25	20
Void Ratio	0.266	0.555	0.411
Liquid Limit (%)	28	46	37
Plastic Limit (%)	10	21	16
Plasticity Index (%)	18	25	22
Dry unit weight (KN/m^3)	12.98	14.95	13.97
Bulk unit weight (KN/m^3)	15.70	18.25	16.98
Undrained Cohesion (KN/m^2)	45	48	46
Angle of internal Friction	6	7	6
Specific Gravity (SG)	2.70	2.74	2.72
USCS Classification	CL	CI	CI

Engineering Properties	Min	Max	Mean
Effective particle size D_{10} (mm)	0.08	0.09	0.09
Effective particle size D_{30} (mm)	0.17	0.23	0.20
Effective particle size D_{50} (mm)	0.18	0.20	0.19
Effective particle size D_{60} (mm)	0.20	0.26	0.23
Coefficient of Uniformity Cu	2.50	2.89	2.69
Coefficient of Curvature Cc	1.80	2.30	2.05
Specific Gravity (SG)	2.65	2.67	2.66
USCS Classification		SP	

Table 3: Engineering properties of the underlying soil at Woji Location

Engineering Properties	Min	Max	Mean
Natural moisture content (%)	35	45	40
Void Ratio	0.377	0.543	0.460
Liquid Limit (%)	43	54	49
Plastic Limit (%)	12	20	16
Plasticity Index (%)	31	34	33
Dry unit weight (KN/m^3)	12.7	14.9	13.8
Bulk unit weight (KN/m^3)	16.3	18.7	17.5
Undrained Cohesion (KN/m^2)	42	48	45
Angle of internal Friction	5	7	6
Specific Gravity (SG)	2.55	2.76	2.66
USCS Classification	CI	CH	CH

Engineering Properties	Min	Max	Mean
Effective particle size D_{10} (mm)	0.08	0.09	0.09
Effective particle size D_{30} (mm)	0.11	0.16	0.14
Effective particle size D_{50} (mm)	0.15	0.21	0.18
Effective particle size D_{60} (mm)	0.17	0.27	0.22
Coefficient of Uniformity Cu	2.13	3.00	2.56
Coefficient of Curvature Cc	0.89	1.05	0.97
Specific Gravity (SG)	2.65	2.70	2.67
USCS Classification		SP	

Table 4: Engineering properties of the underlying soil at Rumuolumeni Location

Engineering Properties	Min	Max	Mean
Natural moisture content (%)	50	60	55
Void Ratio	0.406	0.489	0.448
Liquid Limit (%)	55	71	63
Plastic Limit (%)	10	14	12
Plasticity Index (%)	45	57	51
Dry unit weight (KN/m^3)	9.81	10.8	10.31
Bulk unit weight (KN/m^3)	17.8	19.5	18.65
Undrained Cohesion (KN/m^2)	42	44	43
Angle of internal Friction	5	6	6
Specific Gravity (SG)	2.23	2.45	2.34
USCS Classification	CI	CH	CH

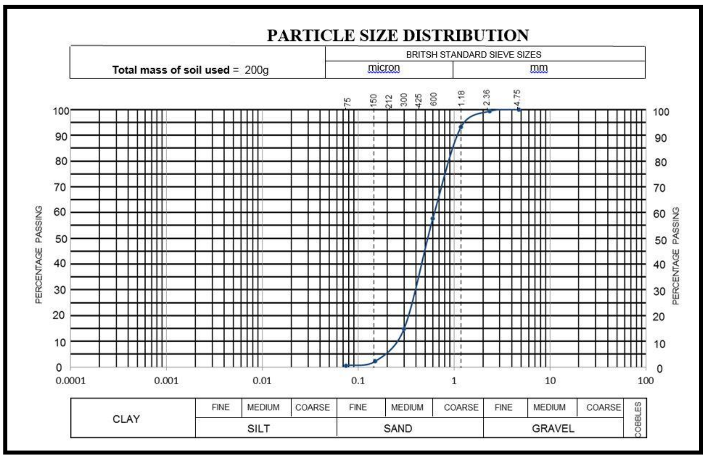

Fig 6: Particle size analysis graph of Trans Amadi Location

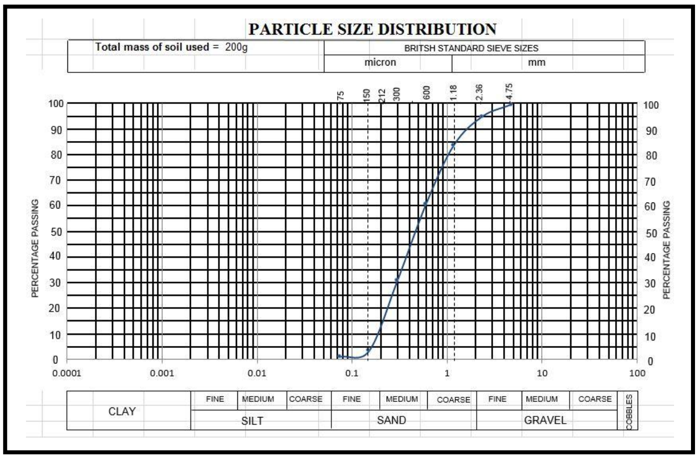

Fig 7: Particle size analysis graph of Woji Location

The bearing capacity calculation for shallow foundations was restricted to the depth of 3m. Static ultimate bearing pressures have been computed using the formula proposed by Terzaghi (1943).

$$q_u = cN_c + qN_q + 0.5\gamma BN_\gamma$$

Where,

q_u = ultimate bearing capacity
c = Undrained cohesion
q = effective overburden pressure ($D\gamma$)
γ = effective unit weight
N_c, N_q, N_γ = Terzaghi's bearing capacity factors

The proposed ultimate bearing capacity and safe bearing capacity at various depths in the investigated locations are shown in Tables 5, 6, and 7.

Table 5: Bearing capacity for shallow foundations (Trans Amadi Location)

Depth (m)	**Width (m)**	**Unit weight 'γ' (kN/m³)**	**Angle of friction (degree)**	**Cohesion 'Cu' (kN/m³)**	**Ultimate Bearing Pressure (kN/m²)**	**Safe Bearing Pressure (kN/m²)**
1.0	0.5 1.0 2.0	18.78	6	46	345.19 345.67 346.60	115.07 115.22 115.53
2.0	0.5 1.0 2.0	18.78	6	46	377.12 377.59 378.53	125.71 125.86 126.17
3.0	0.5 1.0 2.0	18.78	6	46	409.05 409.52 410.57	136.35 136.51 136.82

Table 6: Bearing capacity for shallow foundation (Woji Location)

Depth (m)	**Width (m)**	**Unit weight 'γ' (kN/m³)**	**Angle of friction (degree)**	**Cohesion 'Cu' (kN/m³)**	**Ultimate Bearing Pressure (kN/m²)**	**Safe BearingP ressure (kN/m²)**
1.0	0.5 1.0 2.0	19.60	6	45	339.81 340.31 341.28	113.27 113.43 113.76
2.0	0.5 1.0 2.0	19.60	6	45	373.13 373.62 374.60	124.38 124.54 124.87
3.0	0.5 1.0 2.0	19.60	6	45	406.45 406.94 409.92	135.48 135.65 135.97

Table 7: Bearing capacity for shallow foundations (Rumuolumeni Location)

Depth (m)	**Width (m)**	**Unit weight 'γ' (kN/m^3)**	**Angle of friction φ (degree)**	**Cohesion 'Cu' (kN/m^3)**	**Ultimate BearingPressure (kN/m^2)**	**Safe BearingPressure (kN/m^2)**
1.0	0.5 1.0 2.0	18.65	5	43	309.81 310.27 311.21	103.27 103.42 103.74
2.0	0.5 1.0 2.0	18.65	5	43	324.73 325.19 326.13	108.24 108.39 108.71
3.0	0.5 1.0 2.0	18.65	5	43	339.65 340.11 341.05	113.22 113.37 113.68

VI. INTERPRETATIONS AND DISCUSSIONS

The Atterberg limit analysis of the clay samples show that the clays within the Port Harcourt area are relatively inorganic clays of medium to high plasticity, and will exhibit both low compressive and shear strength, as well as high compressibility. Thus, it is classified as CI and CH according to the Unified Soil Classification System (USCS). The liquid limits ranging from 28% - 71% are relatively high, while the Plasticity Index (PI), which represents the range in the water contents through which the soil is in the plastic state (Seed & Woodward, 1964) are high, having values averaging 54. Sowers and Sowers (1970) noted that PI>31 should be considered high, and indicates high content of expansive clays, most probably montmorillonite and/or illite. (Seed & Woodward, 1964; Sowers & Sowers, 1970) also noted that high plasticity materials are usually susceptible to high compressibility.Highly plastic clays usually have the ability to retain appreciable amount of total moisture in the diffuse double layer, especially by means of absorption (Aghamelu *et al*., 2011). This was clearly seen in the high moisture content of the clays within the area, with a mean value of 38%.The specific gravity (SG) test which serves as a classification test that helps to identify the kind of material and range of its grain sizes, show that the clays are mostly inorganic with subordinate organic clays, with values ranging from 2.27 – 2.72.

The unconsolidated undrained triaxial text conducted on the clay samples to determine their strength parameters [angle of internal friction (φ) and cohesion (c)], has shown that the samples have less ability to resist shearing deformation stresses. This is significantly observed in the low values of cohesion (46kpa) and angle of internal friction (6^0) found within the cohesive materials, which will have its strength reduced with moisture influx, and in turn lead to bearing capacity loss during the life of any engineering project. Punmia *et al*. (2005) noted that clays that are non-consolidated most often record very low values of φ (close to 10^0). The low φ values are attributed to expansive clays as reported by Obiora and Umeji (2004).

The calculated bearing pressure for the underlying soil has shown that the existing cohesive material with an ultimate bearing capacity that ranges from $329.30KN/m^2$ to $410.47KN/m^2$ at 1m and 3m respectively will give a safe bearing pressure of $109.76KN/m^2$ to $136.83KN/m^2$ with reasonable margin of safety, taking into consideration the heterogeneous nature of the soil. For the cohesive soil, a shallow foundation of raft type will constitute low settlement, as long as the maximum allowable bearing pressure is not exceeded.

The medium dense sand occurring at the depth of 8 – 15m have been interpreted to be a poorly graded sand, having coefficient of uniformity (Cu) ranging from 2.52 – 5.2 and coefficient of curvature (Cc) ranging from 0.99 – 1.80. Thus, it is classified as SP according to the Unified Soil Classification System (USCS). It is noted that poorly graded sand will exhibit poor engineering characteristics due to its increased porosity that will result to higher immediate settlement when a load is placed on it. However, the in-situ Standard Penetration Test (SPT) with N - values ranging from 20 – 28 has shown that the sands are medium dense, and thus, will most probably exhibit low immediate settlement and higher load bearing potential.

VII. CONCLUSION

The analysis was carried out to determine the geotechnical properties of the underlying soil in parts of Port Harcourt city of Rivers state, for the design and construction of civil infrastructures. The pattern of soil deposition as revealed from the borehole log indicates a layer of soft – firm sandy Clay with an average thickness of 8m, and a medium dense sand of about 7m thickness.

The classification and strength tests show that the underlying cohesive materials will exhibit low shear and compressive strength, with high compressibility. Highly compressible soils are not usually suitable as load bearing materials. However, with an allowable bearing pressure averaging 136.83KN/m^2, a shallow foundation of raft type will constitute low settlement, provided the maximum allowable bearing pressure is not exceeded.

The non – cohesive soils classed as SP according to the USCS classification, indicative of poorly graded sand. However, the SPT test has revealed the sands as medium dense sand which will amount to low immediate settlement when a load is placed on it.

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