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Geotechnical Investigation Of Sub-Soil For Disaster Risk Reduction Of Building Collapse At Ovade-Ogharefe, South-South Nigeria

Irunkwor, T. C, Nmorsi, D. D, Okobia, C, Atalor, B, Abanjo, N, Molua, C. O, Ngerebara, O.D

Department Of Environmental Management And Toxicology, University Of Delta, Agbor, Delta State, Nigeria
Department Of Geology, University Of Delta, Agbor, Delta State, Nigeria
Department Of Physics, University Of Delta, Agbor, Delta State, Nigeria
Department Of Geology, Rivers State University, Port-Harcourt, Nigeria

Abstract
Geotechnical investigation of sub-soil for disaster risk reduction and its use as support for foundation of civil infrastructures and gas processing plant to prevent building collapse at Ovade-Ogharefe was carried out to determine the safe bearing capacity of the soil materials and to recommend suitable foundation design options. Twelve soil samples were collected from 6 boring locations using the Shell-and-Auger Percussion Rig to depth of 20m where undisturbed and disturbed samples were taken at regular intervals of 1.50m. Two soil zones of Brownish Lateritic Clayey-Sand and Grayish well-graded Sands and Gravels were identified. Results of the Atterberg limit of clay samples revealed that the Liquid Limit varied from 14% to 18%, Plastic Limit varied from 4% to 10.4% and the Plasticity index varied from 5.2% to 14.2%. The coefficient of volume compressibility, coefficient of consolidation, residual cohesion, residual friction angle and standard penetration test on the Brownish Lateritic Clayey-Sands and the Brownish well-graded Sands and Gravels subjected under an overburden pressure of 50Kpa and 400Kpa revealed a subsurface bearing capacity of the upper lateritic clayey sands of 201.284Kpa for isolated footings, 160.76Kpa for continuous strip footings and 122.85Kpa for raft footings to depth of emplacement of 1.50m, 0.75m and 1.50m respectively. The study therefore recommends shallow foundation system which includes isolated footings, continuous strip footings and raft footings respectively for the office complex, residential buildings and gas processing plant.

Keywords: *Allowable Bearing Capacity, Building Collapse, Disaster Risk Management, Foundation Design Option, Geotechnical Investigation, Standard Penetration Test*

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I. Introduction

Building is a structure that is constructed or occupied consisting of a roof and walls designed for specific purposes such as residential, commercial, industrial and institutional usage [1,4,5]. Buildings are intended to provide shelter, accommodation or space for various human activities thereby shaping the built environment and addressing societal needs [2]. Therefore, the safety, serviceability and cost-effectiveness of buildings or civil infrastructures should be given the utmost attention before and during construction. Building collapse is a sudden partially or entirely structural failing of civil infrastructures. Such structural failure occurs when the internal load structural elements fail [3].

While building collapse is a global concern, it is particularly frequent and devastating in developing countries like Nigeria. The frequency and magnitude of building collapses in Nigeria have reached alarming levels in recent years, resulting in considerable loss of lives and properties. The Council of Registered Engineers of Nigeria (COREN) reported 22 cases of building collapse and 33 fatalities between January and July 2024 [4]. According to the Building Collapse Prevention Guild (BCPG), as cited in Ogundeji [5], Nigeria experienced 553 building collapses between 1974 and April 2023 with Lagos State accounting for 59.05% (326 cases) of these incidents. In contrast, States like Zamfara recorded only one collapse in 2018, and others like Taraba, Bayelsa, Gombe and Yobe reported their first collapse cases in 2022 [5].

Over the past 20 years, Nigeria ranked highest in the frequency and severity of building collapses in Africa exemplified by the 21-Storey building collapse in Ikoyi, Lagos on 2nd November, 2021 which claimed 45 lives and seriously injured 10 others [6]. Similarly, notable collapses in 2022 include a three-Storey building in Yaba, Lagos (February 12), the Salvation Ministries church building in Asaba, Delta State (January 11), and the

Deeper Life Bible Church in Badagry, Lagos (April 24) [7-9, 33]. Building collapses also occurred in 2019 and they include the collapse of 3 buildings in Abraka and Asaba, Delta State, and the collapse of a three-storey building in Ita-Faaji, Lagos island that claimed 20 lives [10, 12-14]. Moreover, the Nigerian Building Collapse Prevention Guild (BCPG) shockingly reported that the country has experienced 135 building collapses with about 26 deaths between 2022 and 2024 [11]. These incidents highlight the urgent need for intervention by relevant authorities.

Researchers including [15-18] have attributed building collapses in Nigeria to factors such as poor design, faulty construction, substandard materials, rushed construction and inadequate enforcement of building codes by town planning authorities. Non-adherence to building standards, unexpected load factors not accounted for in building codes, and the engagement of unqualified builders further exacerbate the problem [15, 6]. Additionally, there are no Federal or State regulations in Nigeria mandating consultations with certified professionals for construction projects. Consequently, professional bodies like the Council of Mining Engineers and Geoscientists (COMEG), the Nigeria Association of Engineering Geologists and the Environment (NAEGE), the Nigerian Environmental Society (NES), and the Nigeria Institute of Town Planners (NITP) are unable to exercise adequate oversight. This lack of enforcement fosters the use of substandard materials and increases impunity among building developers.

Preventing of building and infrastructure collapses necessitates thorough investigation of the subsurface soil conditions that support structural foundations. Such investigations help determine the soil's bearing capacity and ensure compliance with geotechnical and civil engineering standards before construction [19]. Variations in the engineering properties of earth materials such as soil and rock and the type of structures built on them, can lead to geotechnical failures and geologic hazards if proper investigations are not conducted before the construction exercise to determine the engineering properties of the soil [20, 21].

Geotechnical investigations also plays a crucial role in disaster risk reduction by providing insights into ground conditions and local geology, assessing risks and developing mitigation measures to enhance infrastructure resilience. This study therefore employed geotechnical method to investigate subsoil engineering properties at Ovade-Ogharefe, Delta State, Nigeria. The findings were used to design suitable foundation systems for office blocks, residential buildings, and a gas processing plant with the aim of preventing infrastructure-related disasters and promote sustainable development by reducing loss of lives, economic damage, and environmental degradation.

II. The Study Area

Regional Geology and Topography

Ovade-Ogharefe with co-ordinates of Latitude 06^0 25' N and Longitude 05^0 43' E (Figure 1) is a community located in Ethiope West Local Government Area of Delta State, Nigeria. Topographically, Ovade-Ogharefe is characterized by relatively flat terrain with admixture of marine and fluvial sediments. The Ethiope River, a flat-floored watercourse traverses the area and drains into the Atlantic Ocean. The region's flood plains are highly susceptible to flooding during the wet season, primarily due to heavy rainfall, a high groundwater table and the flat nature of the valleys. Geologically, the area is within the Dahomey basin and underlain by the *Benin Formation* often referred to as *the Coastal Plain Sands* (Qp) associated with the lower Quaternary period (Pliocene-Pleistocene epoch) and the Aluvium of the upper Quaternary (Recent Sediments) which consists of silty clayey sands, sand and gravels.

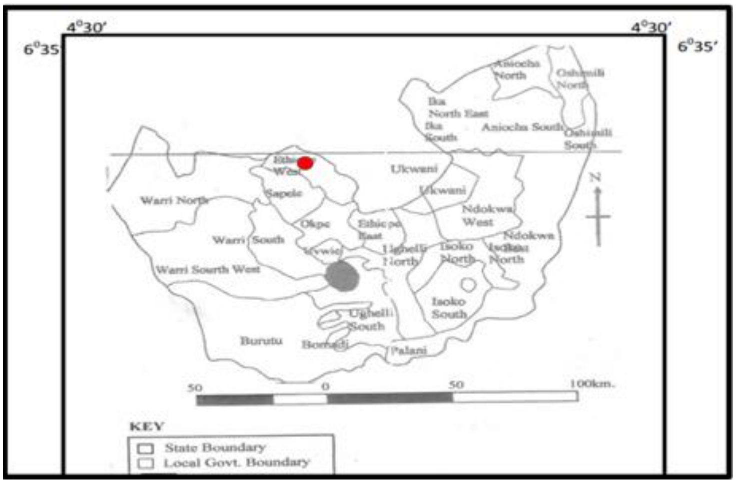

Fig. 1: Delta State map with Study Area in Red Ellipse (modified after Ministry of Land, Survey and Urban Development, Asaba [37]

III. Materials And Methods

Data Acquisition

This study employed the random sampling method and geotechnical data acquisition technique to gain insight into the subsurface soil condition.

Geotechnical Data acquisition

Six locations were randomly selected at the site for sub-soil boring (Figure 2). Boring was conducted at each of the 6 locations to a depth of 20m using the Shell-and-Auger Percussion Rig. Two (2) soil samples were collected at each of the six (6) locations during drilling. Thus, a total of 12 soil samples were collected from the 6 boring locations where undisturbed and disturbed samples were taken at regular sampling intervals of 1.50m during the drilling for visual examination, laboratory testing and classification. Within zones containing cohesive materials such as clays or sandy clays, undisturbed soil samples were obtained with split-spoons and U4-tubes. The water table was not encountered during the drilling indicating that it lies below the final boring depth of 20m.

Standard Penetration Tests (SPT) was carried out at sampling depths where non-cohesive or cohesionless soils (c-ø soils) were encountered. A 50mm diameter split-spoon sampler was driven 450 mm into the soil using a 63.5kg hammer falling through a height of 760mm. The initial 150mm penetration served as a test drive and the number of blows (N) required to drive the remaining 300mm penetration was recorded as the SPT N value. These values were used to assess the bearing capabilities of the subsurface soil layers to withstand foundation loads.

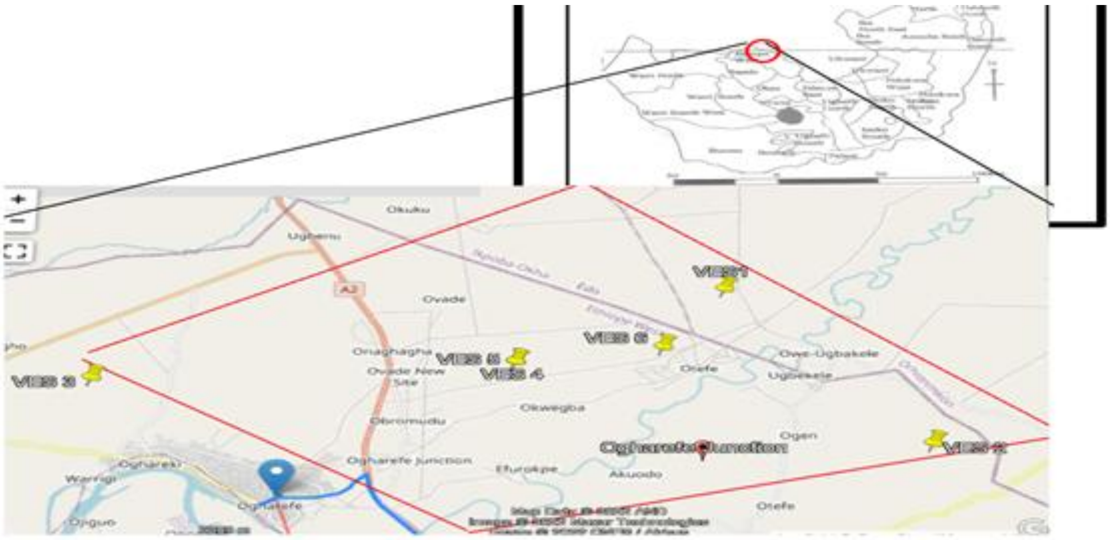

Fig. 2: Location of drilled holes for Sub-Soil Samples Collection in the study Area

Laboratory Testing and Classification

The retrieved soil samples were taken to the laboratory and were tested for Atterberg limit, natural moisture contents, particle size analysis, unit weight, soil strength test for undrained unconsolidated triaxial tests for cohesionless samples and compressibility test applying the one-dimensional oedometer test. The procedures are described as follows:

(i) Atterberg Limit test determined the liquid limit (LL) and the plastic limit (PL) of fine grained soils with particle size less than 0.425mm following the procedures outlined in [22, 23]. The Plasticity index (PI) was calculated as the difference between LL and PL representing the range of water contents in the soil over which the soil exhibits plastic behavior and it is given as:

$$PI = LL - PL \qquad (1)$$

(ii) Moisture content was determined by drying the soil in an oven for about 6 hours at a temperature of 110°C to a constant mass. Thus:

$$\text{Moisture Content} = \text{mass of water in soil}/\text{mass of dried soil} \qquad (2)$$

$$= (\text{Mass of moist soil} - \text{mass of dried soil}) \times 100\%/\text{mass of dried soil} \qquad (3)$$

(iii) Particle size analysis was done using a standard sieve to shake the dry soil samples for several minutes such that while a significant fraction of the soil material passes a 0.075mm sieve, some particles would still be retained in the 0.075mm sieve. This helps to determine the grain distribution and grading. The general slope of the distribution curves could be described by the coefficient of uniformity C_u, and coefficient of curvature C_c

Where:

$C_u = D_{60}/D_{10}$ 4

$C_c = (D_{30})^2/D_{10} \times D_{60}$.. 5

And 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.

(iv) Unit weight was determined by measuring the mass and volume of the undisturbed soil samples using a cylinder of specified dimensions.

(v) Soil strength properties were assessed using undrained-unconsolidated (U-U) triaxial compression tests. Soil specimens (35mm diameter and 110mm height) were prepared from samples obtained with U-4 shebly tubes of 120mm diameter and tested under cell pressures of 50, 100 and 200Kpa respectively. The undrained cohesion (C_u) and undrained friction angle ($ø_U$) were determined as strength indices. Residual cohesion (C) and friction angle (ø) were obtained through direct shear tests on block soil samples.

(vi) The Compressibility (Consolidation) test: The Terzaghi one-dimensional Oedometer consolidation test with a consolidometer was used to evaluate the bearing capacity of cohesive subsurface materials. Two parameters were measured: (a) the coefficient of volume compressibility (M_v) which determines the compressible area that is under a given amount of load; and (b) the coefficient of consolidation (C_v), which represents the likely rate of settlement per annum under the given loading conditions. The tests were performed over a pressure range of 50Kpa to 400Kpa on cohesive soil samples following standard procedures recommended by [24].

Evaluation of Rate of Soil Settlement

The rate of vertical downward movement, shrinking, depression and compression of soil particles from the ground surface arising from the weight of a built structure on the topsoil, otherwise known as settlement was evaluated. This was assessed due to the likely uneven sinks or settlement of the soil that would occur with time and may cause structural deformation in the form of cracks in walls and floor, indicating foundation failure. Thus the likely settlement resulting from loading on the various structures situated on the clayey sand stratum was assessed by considering both the dimensions of the structure and the subsurface lithology beneath the applied foundations. In this context, the total settlement of foundation footings consists of two components: (i) the immediate settlement that occurs during the construction phase and (ii) the long-term settlement that would occur after 90% of consolidation (T90).

The stresses transmitted to the surface of the clayey sands layer (ρ_{Total}) is given as:

ρ_{Total} = Immediate Settlement, ρ_i + Final Settlement, Sc 6

Where: ρ_i was computed from laboratory oedometer data

The Oedometer settlement (ρ_{oed}) at the point of foundation level is given by [32]:

$\rho_i = \rho_{oed} = m_v \cdot \sigma_z \cdot H = m_v \times 0.55\, q_n \times 1.5\, B$ …………………………………………………………… 7

where: m_v = average coefficient of volume compressibility obtainable from laboratory tests on soils from the site

= $0.16 \times 10^{-4} m^2$ / MN, σ_z = average effective vertical stress imposed on the soil by the superstructure=$0.55q_n$,, q_n = imposed load = T (MPa), B = Width of the Production Building, and H = thickness of the compressible layer from bottom of foundation to competent layer.

When the vertical stress distribution pattern is represented by a triangular distribution, the above equation becomes:

$\therefore \quad \rho_i = (0.16 \times 10^{-4}\ m^2/MN)\ (0.55)\ (T\ MPa)\ (1.5 \times B\ m) = 0.0000132(T)(B)m$

[27] was also used to compute the final settlement (Sc) such that:

$Sc = Cc / 1+e_o\ [H_o. Log_{10}\ \{ \sigma_{vo} + \Delta_{\sigma v} \} / \sigma_{vo}]$ ……………………………………………………………… .. 8

Where S_c = final settlement (in cm) of layer of thickness H (m), H= thickness of compressible layer beneath base of foundation = 7.50 m, σ_{vo} = vertical stress in kN/m^2 induced at the center of layer by the net foundation pressure q_n = (17.8 kN/m^3) (3.75m) = 66.75 kPa, Cc= Compression Index ~ 0.009 (w_L – 10) = 0.009(20.4 – 10) = 0.0936,$\Delta\sigma$v = imposed Structural loads on the soil ~ T kPa

$\therefore$ Sc = Cc / 1+ e_o H_o. Log_{10} { σ_{vo} + $\Delta_{\sigma v}$ } / σ_{vo}

= (0.0936 / 1+ 0.8) (7.50m) log_{10} { (66.75 kPa + T kPa) / 66.75kPa}.

= { 0.520 } log_{10} {1 + T / 66.75} cm

S_c = { 0.520} log_{10} {1 + 0.01498T}

The Total settlement at the site (table 7) is therefore given as:

ρ_{total} = ρ_i + S_c …………………………………………………………………………………………… 9

= 0.000132T m + { 0.520} log_{10} {1 + 0.01498T }m

The time period required for either 50% or 90% of the final foundation settlements was computed with the equation:

$t_{(years)}$ = T.d^2/Cv ……………………………………………………………………………………………... 10

Where, d=H (thickness of clay layer measured from foundation level to point where z is small, such as 10 – 20 kPa for drainage in one direction or d=H/2 for drainage at top and bottom of clay stratum) = 3.75 m, C_v=Average of coefficient of consolidation over the range of pressures involved (obtainable either from tri-axial compression or oedometer tests) = 0.76 m^2/yr; T=time factor which for the given condition of loading and drainage at the site corresponds to T_{50} = 0.20 and T_{90}= 0.85.

Evaluation of Soil Liquefaction Potentials

Gas processing plants have a property of vibrating continuously, thus the potential for soil liquefaction was assessed by using the SPT values obtained at the 6 boring locations and the average standard unit weight of 2.76Mg/m^3. Soil dynamics analysis was determined from the oedometer test using Poisson's ratio of between 0.45 and 0.50, shear modulus G of 4.50MPa, and modulus of elasticity E_s of 62,500MN/m^2. The assessment considered three earthquake magnitudes: M= 6.0, 7.5 and 8.25.

The SPT N values, the shear modulus (G), the modulus of Elasticity (E), Poisson's ratio (υ) and earthquake magnitudes (M) values were used to compute and determine the soil liquefaction potential in this study. The raw SPT values were corrected for overburden pressure and hammer energy efficiency by using the relation by [38]:

(N1)60 = NxCN(60ER) 11

And:

CN = {P_0/δv^I}$^{0.5}$ $\leq$ 1.7 12

Where:

P_0 = Atmospheric pressure (≈100Kpa or 100KN/m^2)

δv^I = Effective vertical stress (Kpa). Assume 100Kpa or 100KN/m^2

(N1)60 = the corrected SPT values

CN = Overburden correction factor

ER = Energy ratio (assume 60% for standard safety hammer)

The magnitude scaling factor (MSF) was applied to adjust the earthquake magnitudes of 6.0, 7.5 and 8.25 by using the relation [39]:

MSF = (102.24)(Mw)(2.56) 13

Where:

Mw = the earthquake magnitude

The cyclic stress ratio (CSR) induced by the earthquake of varying magnitudes is given by [38, 40]:

CSR = 0.65(gamax)($\delta v^I\delta v$).rd 14

Where:

Gamax = Peak ground acceleration (PGA). Assume 0.3g for M=7.5 (scaled for M=6.0 and 8.25)

$\delta v\delta v$ = Total vertical stress (assume 150kN/m^2); δv = 200Kpa

$\delta v^I\delta v$ = Effective vertical stress (100kN/m^2); δv^I = 100Kpa

rd = Stress reduction factor (≈ 0.95 for shallow depth such as 5m)

The cyclic resistance ratio (CRR) for the corrected SPT N values (N1)60 was estimated using [38] and [40] correlation:

CRR = exp[14.1(N1)60 + (126(N1)60)^2 – (23.6(N1)60)^3 + (25.4(N1)60)^4 -2.8] 15

The Factor of safety (FS) against liquefaction is given by:

FS = (CRR)/(CSR) 16

Factor of safety (FS) against liquefaction must be greater than unity (i.e FS > 1)
If CRR > CSR (No liquefaction is expected)

IV. Results And Discussion

Engineering Properties of the Soil

The soil profiles within the explored depth of 20m were categorized based on consistency, gradation and strength into 2 distinct soil types/zones; (1) the Brownish Lateritic Clayey Sands (Sc) and (2) the Brownish Well Graded Sands and Gravels (Sw). The fence diagram and distribution of boring points in the site as well as the sub-surface geotechnical lithologic borehole logs at drilling points 1 to 6 are shown respectively in figures 3 and 4 while the geotechnical indices and engineering parameters of the soil are presented in table 1.

The grain size distribution and grading was described by the particle size analysis where the sieve size versus percent of particles of sand passing through the sieve was plotted on a logarithmic scale such that different soils having same degree of uniformity are presented by curves of same shape irrespective of their positions on the particle size distribution plot (Figure 5).

The drainage properties of the soil revealed that the values of coefficient of permeability (k) on the top brownish lateritic clayey sands layer (SC) ranged from 1.75×10^{-8}cm/sec to 1.25×10^{-2}cm/sec (table 2). This result showed that the soils are of both moderate permeability (friable when dry) and very high permeability (low compressibility). The soil consistency result showed that the liquid limit, plastic limit and plasticity index respectively ranged from 14% to 18%, 4% to 10.4%, and 5.6% to 14.2%. The tested soil samples are of good consistency limits indicating low percentage of clay content in the upper brownish lateritic clayey sand layer. [35] adduced that soils with high values of liquid and plastic limits above 20% are considered poor and incompetent as foundation materials. The soils in this study are therefore considered good and competent as foundation materials since the values of liquid and plastic limits are lower than 20% (table 3). Moreover, the plasticity index (PI) of near surface soil samples in the brownish lateritic clayey sand layer are lower than the acceptable standard value of 20% recommended by the Nigeria Ministry of Works and Housing [36] indicating that the soil in this study have good engineering properties and was rated competent (table 3), since the lower the plasticity index of a soil, the higher the soil competence for its use as a foundation support material.

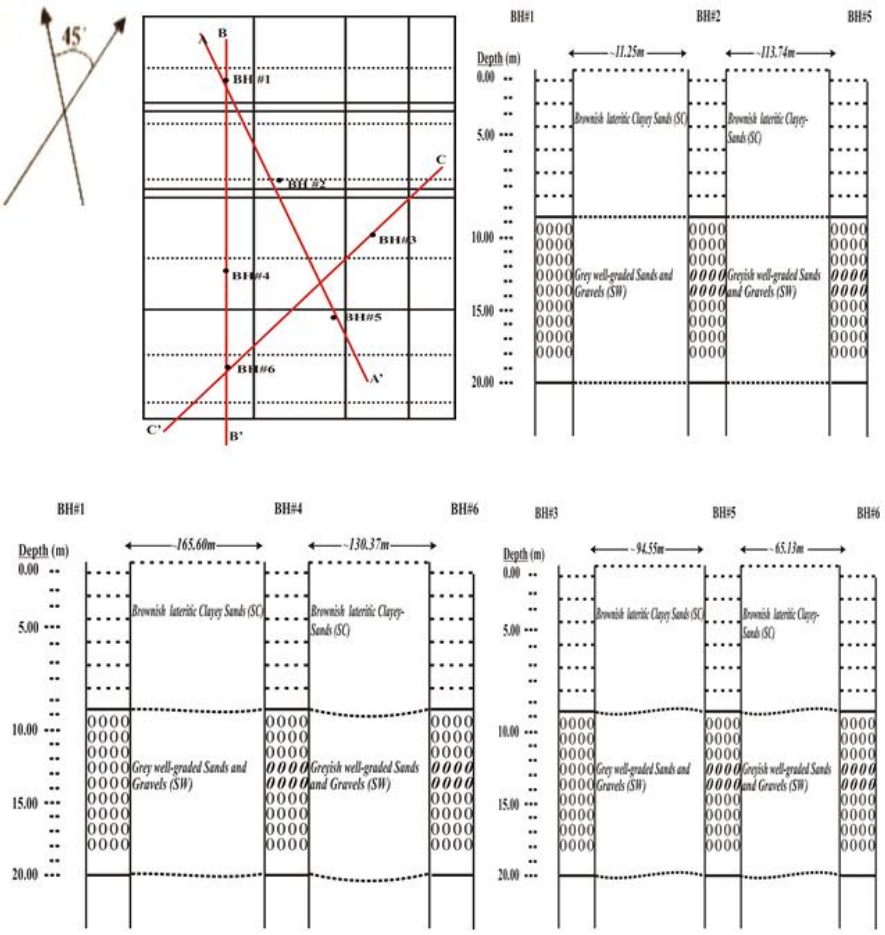

Fig. 3: Fence diagram and distribution of boring points showing Sub-surface disposition of Soil Profile

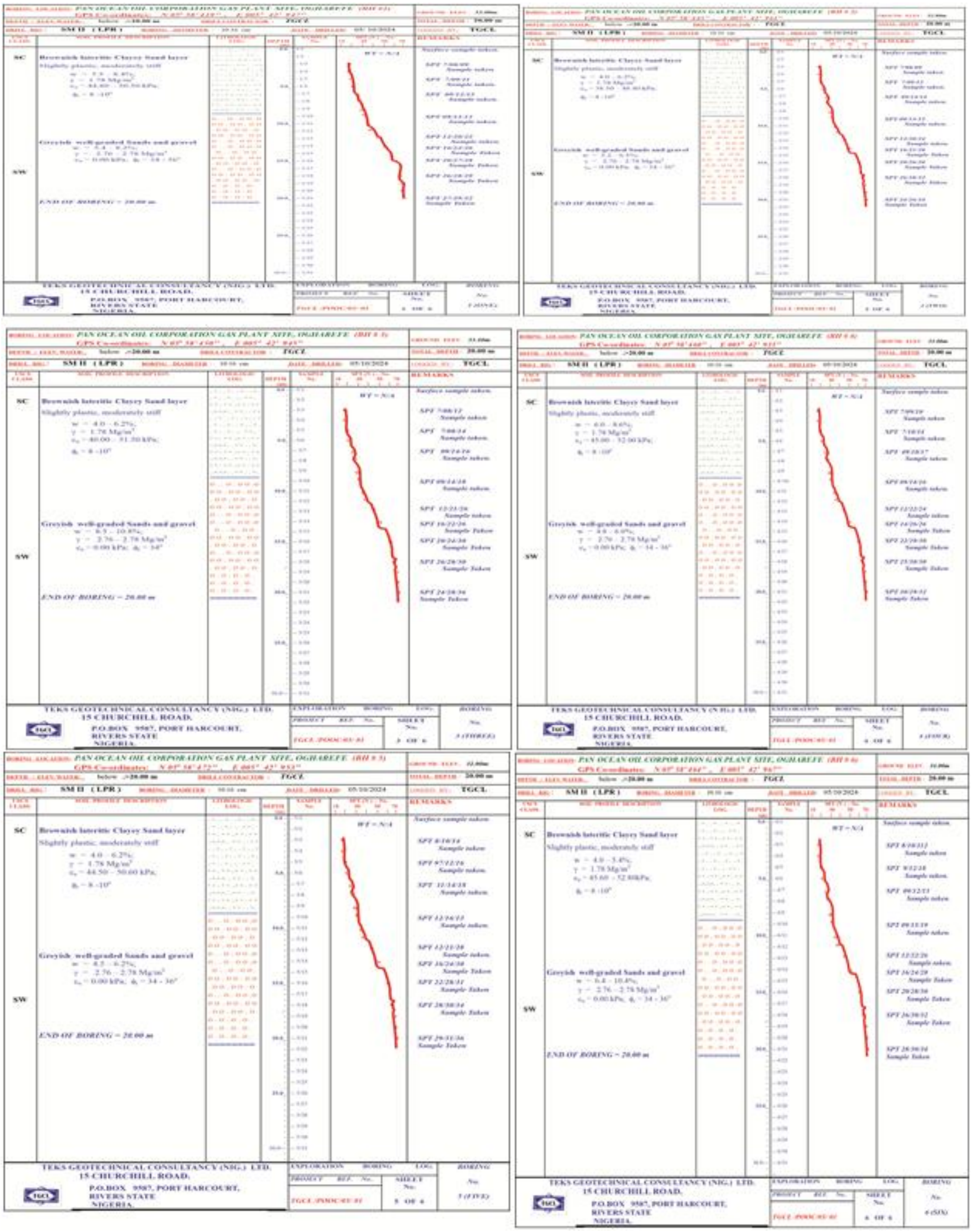

Fig. 4: Sub-surface Geotechnical Lithologic Borehole Logs at drilling Points 1 to 6

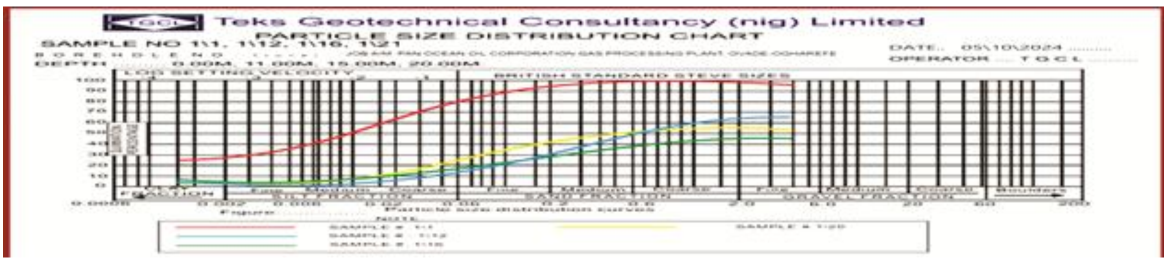

Fig. 5: Particle Size Distribution Curves

Soil Bearing Capacity

The coefficient of volume compressibility (M_v) of the top layer brownish lateritic clay sand (SC) under confining pressures ranging from 50Kpa to 400Kpa was between 0.08m^2/MN and 0.18m^2/MN. The coefficient of consolidation (C_v) for the brownish lateritic clayey sand (SC) ranged from 0.69m^2/year to 0.78m^2/year under an overburden pressure of 50Kpa and from 0.78m^2/year to 0.82m^2/year under an overburden pressure of 400Kpa. The direct shear test results showed that the residual cohesion (C_r) for the brownish lateritic clayey sands (SC) was respectively 38.0Kpa at an overburden pressure of 50Kpa, and 42.50Kpa under an overburden pressure of 400Kpa. However, the residual friction angle ($ø_r$) was found to be 6^o and 8^o under the respective pressure regimes. The Standard Penetration Test (SPT) results for the subsurface materials revealed that the brownish lateritic clayey sands (SC) had SPT N-values of between 16 and 22. The brownish well-graded sands and gravels had SPT N-values of between 26 and greater than 50 (Refusal). These SPT N-values were then converted to Allowable Net Soil Pressures (q_{allow}) or safe bearing capacity (SBC) using the modified relationship by [25]:

q_{allow} = 0.22N(1/F.s) Kpa ………………………………………………………….. 17

q_a = 1/F.s {0.22N(0.1073[1000])}Kpa

where: F.S = Factor of safety = 3.0; and 1Kpa = 1KN/m^2

The result of equation 17 gives the following approximate Allowable Net Soil Pressures (safe bearing capacity) for the various soil layers as:

(i) brownish lateritic clayey sands layer (SC) = 125.898 to 173.110Kpa (125.898 to 173.110KN/m^2)

(ii) brownish well-graded sands and gravels (SW) = 299.01 to 393.43Kpa (299.01 to 393.43KN/m^2)

Since 1Kpa = 1KN/m^2

A close observation in subsoil bearing capacity values revealed that there was an increase in bearing capacity of the soil with depth (table 4) indicating and proving the subsoil competence and its suitability for use as foundation support. Comparing the above calculated values of allowable net soil pressures to the international permissible/accepted values of safe soil bearing capacity in table 5 by [34] revealed that the safe bearing capacity of sub-soil in the study area falls within the international acceptable range of values of safe soil bearing capacity for foundation support such that erected civil infrastructures cannot fail in shear or exceed permissible settlement limit. And the safe soil bearing capacity values by [34] classified the brownish lateritic clayey sands and the brownish well-graded sands/gravel layers in the study area respectively as Loose gravel and dense gravel showing that the soil is competent to support foundations of civil infrastructures. The soil strength and consolidation curve derived from the compressibility test is illustrated in figure 6.

Table 1: Geotechnical Indices and Engineering Parameters of the Sub-soils in the Study Area

Location/ Borehole No	Soil Type	Sam No.	Depth (m)	USC Class	Soil Consistency			Sat. Unit Wt kN/m^3	U – U triaxial Tests		Oedometer Consolidation Tests				AASHTO Compaction		Soundings	Grain size Distribution Pattern (% Passing Sieve Sizes)			
					PL (%)	LL (%)	PI (%)		ϕu (º)	Cu kPa	Mv (m^2 / MN) x 10^{-4} 50.00kPa	Mv (m^2 / MN) x 10^{-4} 400.00kPa	Cv (m^2 / yr.) 50.00kPa	Cv (m^2 / yr.) 400.0kPa	OMC	MDD	SPT (N)	> 4.75mm	4.75Mm	75μ	2μ
B	Brownish lateritic Clayey Sand Layer (SC)	1/11/21/31/4	0.001.002.00	SCSC SCSC SC	5.55.55.55.5	18.817.216.4	13.311.710.9	17.817.817.8	8--8--8---10-	50.5--50.0-4	0.14---0.08	0.18-----0.12-----	0.78---0.69-	0.82---0.78-			-1720-25----	686866656562	565554555452	484646404530	141210080402

H #1		1/51/61/71/81/91/101/11	3.004.005.006.007.008.009.0010.00	SCSC SCSC SCSC	6.87.28.28.48.28.48.0	15.815.214.814.814.214.214.014.0	10.38.47.66.65.87.45.66.0	17.817.817.817.817.817.817.817.8		0.2---44.8-	-----	-	----	----			26-	6664645052	4942404038	2520201010	
	Brownish well-graded sands and gravels layer (SW)	1/121/131/141/151/161/171/181/191/201/21	11.0012.0013.0014.0015.0016.0017.0018.001	SWSWSWSWSWSWSWSWSWSW	5.45.66.56.57.68.28.48.28.28.2	NPNPNPNPNPNPNPNPNPNP	NPNPNPNPNPNPNPNPNPNP	27.627.827.827.827.827.827.627.827.827.8	34-34--34----	0.00-0.00--0.00----	---------	---------	---------	---------			-42---51--61-	48322520080208060402	26222225120804020000	05020000-----	----------

			9.0020.00																		
	Water Table(WT) = >20.0 meters from the ground surface																				
BH #2	Brownish lateritic Clayey Sand Layer (SC)	2/12/22/32/42/52/62/72/82/92/102/11	0.001.002.003.004.005.006.007.008.009.0010.00	SCSC SCSC SCSCSC SCSC SCSC	4.04.64.24.65.45.85.06.86.26.06.2	17.617.416.015.615.214.414.414.014.614.214.2	13.612.811.811.09.88.69.47.28.48.28.0	17.817.817.817.817.817.817.817.817.817.817.8	8--8-8---10-	48.4--48.0-40.2---38.5-	0.12----0.06-----	0.20----0.12------	0.75---0.66------	0.78---0.72------			-21--28----28-	6868666565525654545042	5655545554525952505058	4846464045404530303030	141208080402000000
	Brownish well-graded sands and gravels layer (SW)	2/122/132/142/152/1	11.0012.0013.0014.0	SWSWSWSWSWSWSWSWSWS	6.56.06.05.25.45.55	NPNPNPNPNPNPNPNPNPN	NPNPNPNPNPNPNPNPNPN	27.627.827.827.827.	34-36--36----	0.00-0.00--0.00----	----------	----------	----------	----------			-44--56---60-	38222510080201---	464222252218080402-	15121010862---	00--------

		62/172/182/192/202/21	015.0016.0017.0018.0019.0020.00	W	.65.45.45.8	P	P	827.827.827.827.827.8													
	Water Table(WT) = >20.0 meters from the ground surface																				
BH # 3	Brownish lateritic Clayey Sand Layer (SC)	3/13/23/33/43/53/63/73/83/93/103/11	0.001.002.003.004.005.006.007.008.009.00	SCSC SCSC SCSCSC SCSC SCSC	4.04.54.54.85.05.25.46.25.85.04.8	18.218.017.016.215.115.015.014.214.214.2	14.213.512.511.410.19.89.68.08.49.29.4	17.817.817.817.817.817.817.817.817.817.8	8--8-8--10-	51.5--50.8-40.5 ----40.5	0.14----0.08------	0.18-----0.12------	0.78---0.68------	0.78---0.74------			-2022--30---32-	6866646565525656545238	5452504544424942404048	3836363034323030302828	1210080806040201000

			10.00			14.2		17.8													
	Brownish well-graded sands and gravels layer (SW)	3/123/133/143/153/163/173/183/193/203/21	11.0012.0013.0014.0015.0016.0017.0018.0019.0020.00	SWSWSWSWSWSWSWSWSWSW	4.54.85.25.46.06.87.27.47.47.4	NPNPNPNPNPNPNPNPNPNP	NPNPNPNPNPNPNPNPNPNP	27.627.827.827.827.827.827.827.827.827.8	34-34--36----	0.00-0.00--0.00----	----------	----------	----------	----------			47-48--54---64	3822221006020101--	363222252218080402-	1816141210951--	00--------
	Water Table(WT) = >20.0 meters from the ground surface																				
BH #4	Brownish lateritic Clayey Sand Layer (SC)	4/14/24/34/44/54/6	0.001.002.003.004.	SCSC SCSC SCSCSC S	4.04.54.54.85.05.2	18.218.017.016.215	14.213.512.511.410	17.817.817.817.817	8--8-8---10-	51.5--50.8-40.5 --	0.14----0.08------	0.18----0.12------	0.78--0.68------	0.78--0.74------			-2022--30---32-	686664656552565654	545250454442494240	383636303432303030	121008080604020100

		4/74/84/94/104/11	005.006.007.008.009.0010.00	CSC SCSC	6.06.25.86.04.8	.115.015.014.214.214.214.2	.19.89.08.08.48.29.4	.817.817.817.817.817.817.8		--40.5								5238	4048	2828	0
	Brownish well-graded sands and gravels layer (SW)	4/124/134/144/154/164/174/184/194/204/21	11.0012.0013.0014.0015.0016.0017.0018.0019.0020	SWSWSWSWSWSWSWSWSWSW	4.54.85.25.46.06.87.27.47.47.4	NPNPNPNPNPNPNPNPNPNP	NPNPNPNPNPNPNPNPNPNP	27.627.827.827.827.827.827.827.827.827.8	34-34--36----	0.00-0.00--0.00----	----------	----------	----------	----------			47-48--54---64	382221006020101--	36322252218080402-	1816141210951--	00--------

			.00																		
	Water Table(WT) = >20.0 meters from the ground surface																				
BH #5	Brownish lateritic Clayey Sand Layer (SC)	5/15/25/35/45/55/65/75/85/95/105/11	0.001.002.003.004.005.006.007.008.009.0010.00	SCSC SCSC SCSCSC SCSC SCSC	4.04.54.54.85.05.25.45.45.45.44.8	18.218.017.016.215.115.015.014.214.214.214.2	14.213.512.511.410.19.89.68.88.88.89.4	17.817.817.817.817.817.817.817.817.817.817.8	8--8-8---10-	52.8--50.2-46.8 ----45.6	0.12----0.18------	0.30----0.34-----	0.60---0.72------	0.88---0.76------			-1722--34--34-	5854525254525854525636	4442404544424942303038	3836363034323030302828	1210080806040201000
	Brownish well-graded sands and gravels layer (SW)	5/125/135/145/155/165/175	11.0012.0013.0014.0015.00	SWSWSWSWSWSWSWSWSWSW	6.46.86.27.410.010.210.41	NPNPNPNPNPNPNPNPNPNP	NPNPNPNPNPNPNPNPNPNP	27.627.827.827.827.827.827.82	34-34--36----	0.00-0.00--0.00----	----------	----------	----------	----------			48-52--58---67	342221006020101--	363422252218080402-	1816141210951--	00-------

		/185/195/205/21	16.0017.0018.0019.0020.00		0.210.210.4			7.827.827.8													
	Water Table(WT) = >20.0 meters from the ground surface																				
BH # 6	Brownish lateritic Clayey Sand Layer (SC)	6/16/26/36/46/56/66/76/86/96/106/11	0.001.002.003.004.005.006.007.008.009.0010.00	SCSC SCSC SCSCSCSC SCSC SCSC	4.04.54.54.85.05.25.45.45.44.8	18.218.017.016.215.115.015.014.214.214.2	14.213.512.511.410.19.89.68.88.88.89.4	17.817.817.817.817.817.817.817.817.817.817.8	8--8-8---10-	52.8--50.2-46.8 ----45.6	0.14----0.16------	0.38----0.34------	0.60---0.72------	0.88---0.76------			-2230--25---34-	5854525254525854525636	4442404544424942303038	38363630343230302828	2422201814121010654
	Brownish well-graded	6/1	11.	SWS	6.4	NPN	NPN	27.	34-	0.0	---	---	---	---			48-	342	363	181	42-

	sands and gravels layer (SW)	26/136/146/156/166/176/186/196/206/21	0012.0013.0014.0015.0016.0017.0018.0019.0020.00	WSWSWSWSWSWSWSWSW	6.86.27.410.010.210.410.210.210.4	PNPNPNPNPNPNPNPNP	PNPNPNPNPNPNPNPNP	627.827.827.827.827.827.827.827.827.8	34--36----	0-0.00--0.00----	-------	-------	-------	-------			52--58---64	2221006020101--	422252218080402-	6141210951--	---
	Water Table(WT) = >20.0 meters from the ground surface																				

Table 2: Consolidation, Bearing and Drainage characteristics of the Soil

Soil Type	Depth Range (thickness) m.	SPT N-Values	Bearing Strength C / ϕ	Coefficient of Compressibility (M_v) m^2/MN	Coefficient of Consolidation (C_v) m^2/yr	Coefficient of Permeability (K) cm/sec	Remarks
Brownish lateritic Clayey Sand (SC)	0.0 – 9.00 *(9.00)*	16 - 22	40.00 - 45.52kPa / 6º – 10º	0.08 - 0.34	0.48 - 0.76	1.75×10^{-8}	Moderately impermeable but friable when dry; likely to have high water absorbency
Brownish well-graded Sands /gravel (SW)	9.0 - 20.00 *(>11.00)*	26 to >50	kPa & 34º – 36º	-	-	1.25×10^{-2}	Very High permeability; very low compressibility

Table 3: Competence Rating of the Brownish Lateritic Clayey Sub-soil Layer Using Plasticity Index (PI) Standard

Location/ Borehole No	**Soil Type**	**Sample No.**	**Depth (m)**	**Soil Consistency**			**Acceptable Standard**		**Soil Competence**
				PL (%)	**LL (%)**	**PI (%)**	LL and PL [35]	PI FMWH [36]	

BH#1	Brownish lateritic Clayey Sand Layer (SC)	1/1	0.00	5.5	18.8	13.3	**20%**	**20%**	Competent
		1/2	1.00	5.5	17.2	11.7			
		1/3	2.00	5.5	16.4	10.9			
		1/4	3.00	5.5	15.8	10.3			
		1/5	4.00	6.8	15.2	8.4			
		1/6	5.00	7.2	14.8	7.6			
		1/7	6.00	8.2	14.8	6.6			
		1/8	7.00	8.4	14.2	5.8			
		1/9	8.00	8.2	14.2	7.4			
		1/10	9.00	8.4	14.0	5.6			
		1/11	10.00	8.0	14.0	6.0			
BH #2	Brownish lateritic Clayey Sand Layer (SC)	2/1	0.00	4.0	17.6	13.6	**20%**	**20%**	Competent
		2/2	1.00	4.6	17.4	12.8			
		2/3	2.00	4.2	16.0	11.8			
		2/4	3.00	4.6	15.6	11.0			
		2/5	4.00	5.4	15.2	9.8			
		2/6	5.00	5.8	14.4	8.6			
		2/7	6.00	5.0	14.4	9.4			
		2/8	7.00	6.8	14.0	7.2			
		2/9	8.00	6.2	14.6	8.4			
		2/10	9.00	6.0	14.2	8.2			
		2/11	10.00	6.2	14.2	8.0			
BH #3	Brownish lateritic Clayey Sand Layer (SC)	3/1	0.00	4.0	18.2	14.2	**20%**	**20%**	Competent
		3/2	1.00	4.5	18.0	13.5			
		3/3	2.00	4.5	17.0	12.5			
		3/4	3.00	4.8	16.2	11.4			
		3/5	4.00	5.0	15.1	10.1			
		3/6	5.00	5.2	15.0	9.8			
		3/7	6.00	5.4	15.0	9.6			
		3/8	7.00	6.2	14.2	8.0			
		3/9	8.00	5.8	14.2	8.4			
		3/10	9.00	5.0	14.2	9.2			
		3/11	10.00	4.8	14.2	9.4			
BH #4	Brownish lateritic Clayey Sand Layer (SC)	4/1	0.00	4.0	18.2	14.2	**20%**	**20%**	Competent
		4/2	1.00	4.5	18.0	13.5			
		4/3	2.00	4.5	17.0	12.5			
		4/4	3.00	4.8	16.2	11.4			
		4/5	4.00	5.0	15.1	10.1			
		4/6	5.00	5.2	15.0	9.8			
		4/7	6.00	6.0	15.0	9.0			
		4/8	7.00	6.2	14.2	8.0			
		4/9	8.00	5.8	14.2	8.4			
		4/10	9.00	6.0	14.2	8.2			
		4/11	10.00	4.8	14.2	9.4			
BH #5	Brownish lateritic Clayey Sand Layer (SC)	5/1	0.00	4.0	18.2	14.2	**20%**	**20%**	Competent
		5/2	1.00	4.5	18.0	13.5			
		5/3	2.00	4.5	17.0	12.5			
		5/4	3.00	4.8	16.2	11.4			
		5/5	4.00	5.0	15.1	10.1			
		5/6	5.00	5.2	15.0	9.8			
		5/7	6.00	5.4	15.0	9.6			
		5/8	7.00	5.4	14.2	8.8			
		5/9	8.00	5.4	14.2	8.8			
		5/10	9.00	5.4	14.2	8.8			
		5/11	10.00	4.8	14.2	9.4			
BH #6	Brownish lateritic Clayey Sand Layer (SC)	6/1	0.00	4.0	18.2	14.2	**20%**	**20%**	Competent
		6/2	1.00	4.5	18.0	13.5			
		6/3	2.00	4.5	17.0	12.5			
		6/4	3.00	4.8	16.2	11.4			
		6/5	4.00	5.0	15.1	10.1			
		6/6	5.00	5.2	15.0	9.8			
		6/7	6.00	5.4	15.0	9.6			
		6/8	7.00	5.4	14.2	8.8			
		6/9	8.00	5.4	14.2	8.8			
		6/10	9.00	5.4	14.2	8.8			
		6/11	10.00	4.8	14.2	9.4			

Table 4: Sensitivity Analysis of Bearing Capacities of soils for Isolated and Raft Foundations

S/No.	Foundation Depth (m)	Bearing Capacity Values for Foundation Types [kPa]		Remarks
		Foundation Types		
		Isolated Footings	RAFT FOUNDATION	

		Analytical Methods adopted					
			Meyerhof's (1974)	Bowles' (1988)	Terzaghi & Peck (1967)	Brinch Hansen's (1968)	
1.	0.50	192.38	76.55	82.37	92.01	161.83	There is an increase of bearing capacity with depth except the Meyerhof's Method
2.	1.00	196.83	76.55	85.00	99.43	168.83	
3.	1.50	201.28	76.55	87.63	106.84	175.83	
4.	2.00	205.73	76.55	90.26	114.26	182.83	
5.	2.50	210.18	76.55	92.89	121.68	189.83	

Table 5: Typical Soil Bearing Capacity Values [34]

Soil Type	Safe Bearing Capacity Value (Kpa) (or KN/m^2)
Soft Clay	<75
Firm Clay	75 - 100
Loose Gravel	<200
Dense Gravel	200 - 600

Rate of Settlement and Consolidation

The result of soil settlement and consolidation rate showed that the time period that is required for 50% and 90% of final settlement of the brownish lateritic clayey-sand (SC) layer would be respectively 3.70 years and 15.72 years. These are represented in tables 6 and 7. Thus, the settlement expected to occur during the construction phase (ie immediate settlement) is about 0.0000132T (m) while the long-term settlement value expected to take place long after the construction phase is about $0.000132T + \{(0.520) \log_{10}(1+0.01498T)\}$m where T is the dead weight of the plant.

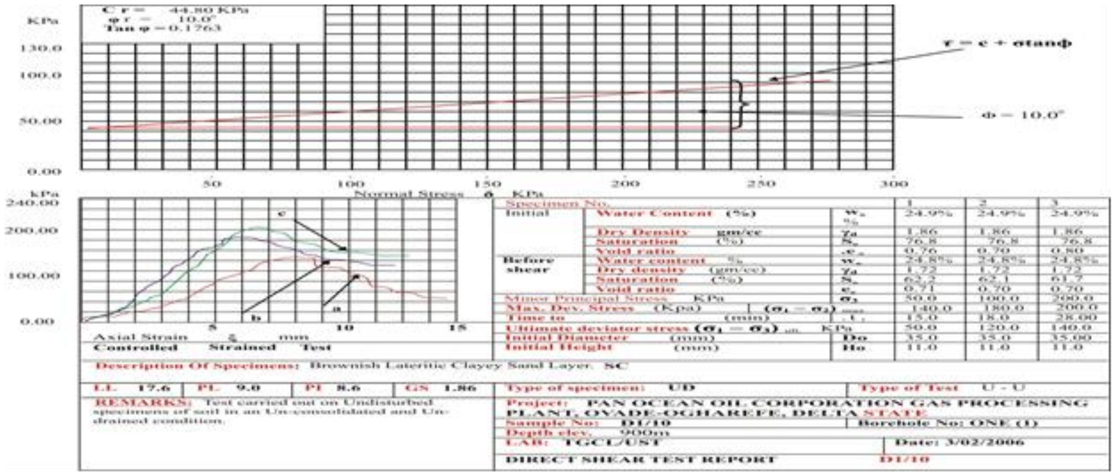

Fig. 6: Shear Strength and Consolidation Curve

Table 6: Computed Settlements in the Study Area

Site	Computed Settlements (cm)			Remarks
	Immediate settlement(ρ_i)	Long-term Settlement (ΔH_f)	Total Settlement (ρ_{total})	
Proposed Gas Processing Plant, Ovade-Ogharefe	0.000132T m	$\{0.520\} \log_{10} \{1 + 0.01498T\}$m	$0.000132T + \{0.520\} \log_{10} \{1 + 0.01498T\}$m	Compressible layer beneath foundation level = 7.50 m

Table 7: Rates of Settlements in the Study Area for the Gas Processing Plant

Project Site	Rates of Settlement (years)		Remarks
	T_{50}	T_{90}	
Proposed Ovade Gas Processing Plant	3.700	15.728	Over a compressive layer of 7.50m beneath foundation level

Soil Liquefaction Potentials

The potential for soil liquefaction in this study was analyzed with liquefaction potential criteria (table 8) and the results indicated that there was no possibility of liquefaction occurring at the site within the three (3) earthquake magnitudes M= 6.0, 7.5 and 8.25 (table 10) except for earthquake magnitudes M=6.0 (N=17) and M=7.5 (N=21, 22, 25) that respectively have marginal, high and moderate Liquefaction potential risk. These sub-soil layers with marginal, high and moderate liquefaction potential risks were due to low N-value (17, 21, 22) and high CSR while the subsurface layers with no possibility of soil liquefaction can be explained by the fact that the water table is more than 20m below the ground surface as liquefaction typically occurs when water table is close to the surface.

The results further showed that for earthquake magnitude 6.0, only N=17 was marginally safe (FS=1.18) while all other SPT values (N≥20) have FS>1.5 therefore no liquefaction risk. For earthquake magnitude (M) 7.5, N=25 was marginally safe at FS=1.0, N≤ 24 has high to moderate liquefaction risk at FS< 1.0 and safe liquefaction of the soil for N≥ 28 at FS> 1.2. Soil liquefaction potential at earthquake magnitude (M) 8.25 indicated that N≤ 28 has high to moderate risk at FS< 1.2 while N=17 (FS1.10) and N≥ 30 (FS> 1.29) are respectively safe. However, N= 20-25 have safer liquefaction potential (FS>1.65). Higher magnitudes reduce FS, but all SPT values N≥ 17 are safe for M=8.25 in the analysis. It can therefore be inferred that higher earthquake magnitudes increase liquefaction risk; and higher SPT N-values significantly increases the cyclic resistance ratio (CRR) and factor of safety (FS). SPT values ≥ 30 are generally safe even for M= 8.25 hence liquefaction risk decreases with increasing SPT N-values.

Table 8: Liquefaction Potential Criteria [38, 40]

Factor of Safety (FS)	Liquefaction Risk
< 1.0	High Liquefaction Risk
1.0≤ FS ≤ 1.2	Marginal Liquefaction Risk
> 1.2	No Liquefaction

Table 9: Magnitude Scaling Factor and Cyclic Stress Ratio for Each Earthquake Magnitude [39]

Magnitude (M)	Magnitude Scaling Factor (MSF)	Cyclic Stress Ratio (CSR)
6.0	1.5	0.17
7.5	2.5	0.28
8.25	0.89	0.31

Foundation Design Options

Based on the subsurface soil types, lithological configuration and depth to the water table, shallow foundations are recommended for the infrastructures and gas processing plant. These foundation options includes: (i) the isolated footings, (ii) the continuous strip footings, and (iii) the raft footings.

Isolated Footings: Isolated footings should be used for the office buildings. The average bearing capacity for the isolated footings ranged from 192.38kpa to 210.18kpa (table 11). These values falls in the range of loose gravel/dense gravel (table 5). Key design considerations include (i) the depth of the footings (D_f) and (ii) the width or breadth of the footing (B). The depth of the foundation is measured from the ground surface to the base of the foundation. The recommended depth for the isolated footings under the office buildings is 1.50m. The width or breadth of the footing is the horizontal dimension in the x-axis of the foundation level which is recommended to be 5.0m. The length of the base for the isolated footings defined as the horizontal dimension in the z-axis is also taken as 5.0m (Figure 7). The Net Ultimate bearing capacity for isolated footings according to [26] is given by:

q_{nf} = $1.3[\,{}_cN_c + p_o\,(N_q - 1\,) + 0.4\,\gamma\,B\,N_\gamma\,]$………………………………………………………… .. 18

where: c = cohesion of soil at site, ~ *50 kPa*.

N_c = Terzaghi Bearing Capacity factor with respect to cohesion (table 13)

N_q = Terzaghi Bearing Capacity factor with respect to surcharge (table 13)

N_γ = Terzaghi Bearing Capacity factor with respect to unit weight (table 13)

B = width or breadth of the Isolated footing = B meters *(assumed)*

γ = unit weight of the soil materials at site. = 17.8 kN/m^3

p_o = effective pressure of overburden soil at foundation level, = $\gamma' D_f$. = (17.8 kN/m^3) (1.50m) = <u>26.70kPa</u>

1.3 = shape factor of footing with respect to cohesion,

At a depth of 1.50m (table 11) we have the following soil properties: .c = 50.00 kPa, ϕ = 10.0°, N_γ = 0.50,

γ = 17.8 kN/m^3, N_c = 8.40, N_q = 2.50. Assuming a Factor of Safety (F.S) = 3.0 and that B/L ~ 1.00 = 5/5 we have:

$\therefore q_{allow}$ = 1/3 {(1.3)(50.00) (8.4) + (26.70 kPa) (2.50 – 1) + (0.4) (17.8 kN/m3) (B m) (0.50) }

= 1/3 { 1.3 x 420} + { 40.05} + { 3.56 B}]

= <u>195.35 + 1.1867 B kPa</u>

Hence, the Net Ultimate Bearing Capacity for the Isolated footing of B meter square for office buildings at the site is found to be about 195.35 + 1.1867 B kPa.

Table 10: Soil Liquefaction Potential Analysis in the Study Area

Layer	Depth (m)	SPT (N)	(N1)60	CRR (M=6.0)	CRR (M=7.5)	CRR (M=8.25)	CSR (M=6.0)	CSR (M=7.5)	CSR (M=8.25)	FS (M=6.0)	FS (M=7.5)	FS (M=8.25)	Liquefaction Risk Potential (based on FS Criteria)		
													M=6.0	M=7.5	M=8.25
Brownish Lateritic Clayey Sand (SC)	0-8	17	17	0.20	0.12	0.34	0.17	0.28	0.31	1.18	0.43	0.39	Marginal	High	None
		20	20	0.30	0.18	0.51	0.17	0.28	0.31	1.76	0.64	0.58	None	High	None
		21	21	0.33	0.20	0.56	0.17	0.28	0.31	1.94	0.71	0.65	None	Moderate	None
		22	22	0.37	0.22	0.62	0.17	0.28	0.31	2.18	0.79	0.71	None	Moderate	None
		25	25	0.47	0.28	0.79	0.17	0.28	0.31	2.76	1.00	0.90	None	Marginal	None
	8-10	28	28	0.58	0.35	0.98	0.17	0.28	0.31	3.41	1.25	1.13	None	None	None
		30	30	0.67	0.40	1.12	0.17	0.28	0.31	3.94	1.43	1.29	None	None	None
		32	32	0.75	0.45	1.26	0.17	0.28	0.31	4.41	1.61	1.45	None	None	None
		34	34	0.83	0.50	1.41	0.17	0.28	0.31	4.88	1.79	1.61	None	None	None
		39	39	1.05	0.63	1.77	0.17	0.28	0.31	6.18	2.25	2.03	None	None	None
Brownish Well-graded Sands and gravel (SW)	10-14	42	42	1.20	0.72	2.02	0.17	0.28	0.31	7.06	2.57	2.32	None	None	None
		44	44	1.27	0.76	2.14	0.17	0.28	0.31	7.47	2.71	2.45	None	None	None
		47	47	1.42	0.85	2.39	0.17	0.28	0.31	8.35	3.04	2.74	None	None	None
		48	48	1.47	0.88	2.47	0.17	0.28	0.31	8.65	3.14	2.84	None	None	None
		51	51	1.64	0.98	2.75	0.17	0.28	0.31	9.59	3.50	3.16	None	None	None
		52	52	1.70	1.02	2.87	0.17	0.28	0.31	10.00	3.64	3.29	None	None	None
	14-20	54	54	1.82	1.09	3.06	0.17	0.28	0.31	10.71	3.89	3.52	None	None	None
		56	56	1.94	1.16	3.26	0.17	0.28	0.31	11.35	4.14	3.74	None	None	None
		58	58	2.05	1.23	3.46	0.17	0.28	0.31	12.06	4.39	3.97	None	None	None
		60	60	2.17	1.30	3.65	0.17	0.28	0.31	12.76	4.64	4.19	None	None	None
		61	61	2.24	1.34	3.77	0.17	0.28	0.31	13.12	4.79	4.32	None	None	None
		64	64	2.44	1.46	4.10	0.17	0.28	0.31	14.29	5.21	4.71	None	None	None
		67	67	2.65	1.59	4.47	0.17	0.28	0.31	15.59	5.68	5.13	None	None	None

Table 11: Final Average Values of Bearing Capacity for Isolated and Raft Foundations

S/No	Foundation Depth (m)	Final Average Bearing Capacity Values [kPa]		Remarks
		Foundation Types		
		Isolated Footing	Raft Foundations	
1.	0.50	192.38	103.20	The values of the Final Average Bearing Capacity for the Raft Foundations are the averages for the Four types of Foundation designs.
2.	1.00	196.83	107.45	
3.	1.50	201.28	111.72	
4.	2.00	205.73	115.98	
5.	2.50	210.18	120.24	

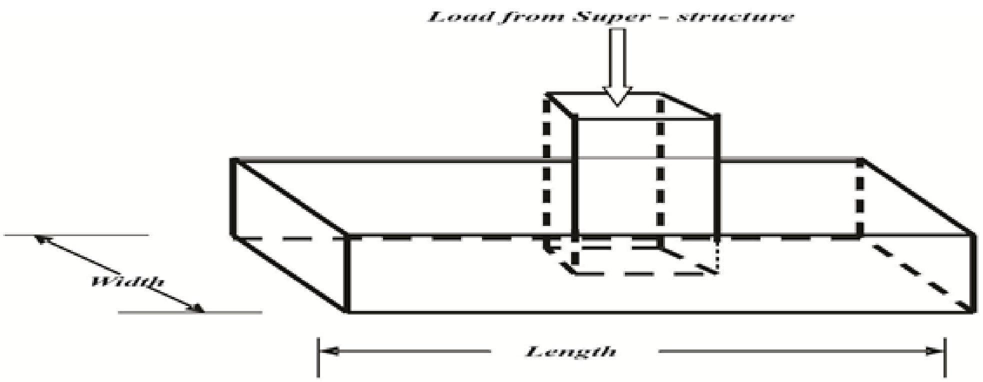

Fig. 7: Schematic of dimensions of an Isolated Footing

The Continuous Strip Footings: Continuous strip footings are recommended for the residential buildings as foundation. The soil bearing capacity for the continuous strip footing is 160.76kpa (table 12) which is in the range of loose gravel classification by [34] (table 5). Key design considerations include:

(i) the depth of the footing (D_f) which should be 0.75m below the ground level. This depth was occasioned from the boring and sounding records obtained during the drilling exercise. The 0.75m depth corresponds to the lateritic clayey sands (SC) layer, which therefore implies that approximately 0.75m of this overburden would be removed or excavated during the foundation construction phase (figure 8); (ii) the width of base of the foundation footing represented as B metres; and (iii) the length of the continuous strip footings represented as L metres for the residential buildings.

The Net Ultimate Bearing capacity for continuous strip footings was evaluated by the equation provided by [26]:

$q_{nf} \quad = \quad {}_cN_c + p_o\,(N_q - 1) + 1/2\,\gamma B\,N_\gamma$ ……………………………………………………………… 19

Where the above parameters are as defined in equation 18 and table 13.

At a foundation depth of 0.75m and assuming a Factor of Safety (F.S) of 3.0, we have

$\therefore\ q_{allow} = 1/3\ \{(50.00)\ (8.4) + (26.70\ \text{kPa})\ (2.50 - 1) + (0.5)\ (17.8\ \text{kN/m3})\ (B)\ (0.50)\ \}$

$= \ 1/3\ \{\{420\} + \{40.05\} + \{4.45\ B\}\}$

$= \ \underline{153.35 + 1.483B\ \ \text{kPa}}$

Thus, the Net Ultimate Bearing Capacity for Continuous Strip Footing with a width of B meters for residential buildings at the site was found to be approximately 153.35 + 1.483B kPa (compared to $q_{(allow)}$ = 195.35 + 1.1867 B kPa for the Isolated Footing of the office buildings).

Settlement considerations based on these equations are limited to 25.4 mm. A Factor of safety of 3.0 was applied to account for any unexpected high settlement values that may likely arise at this site.

Table 12: Bearing Capacity Values for the Infrastructures in the Study Area for the Gas Processing Plant

SPT N-value	Bearing Capacity Computational Methods (KPa)						Field Methods (kPa)	Average Values (kPa)
	Foundation Type Options						SPT	
	Isolated Footings	Continuous Strip Footings	RAFT FOUNDATIONS					
			Meyerhof (1963)	Bowles (1985)	Terzaghi & Peck (1943)	Hansen (1968)		
	Depth of Foundation (m)							
	1.50m	0.75m	1.50m					
17	201.284	160.76	76.55	87.63	107.44	208.86	133.77	139.47
OK		OK	OK	OK	OK	High	OK	Acceptable

Table 13: Values of Terzaghi Bearing Capacity Factors

Depth (m)	ϕ (degrees)	C (Kpa)	N_c	N_γ	N_q
0.50	0	0	5.14	0	1.00
1.00	**5**	25	**6.5**	**0.10**	**1.60**
1.50	10	**50**	8.40	0.50	2.50
2.00	15	75	11.00	1.40	4.00
2.50	20	100	14.8	3.50	6.40
3.00	25	125	20.70	8.10	10.70
3.50	30	150	30.00	18.10	18.40
4.00	35	175	46.00	41.10	33.30

4.50	40	200	75.30	100.00	64.20
5.00	45	225	134.00	254.00	135.00

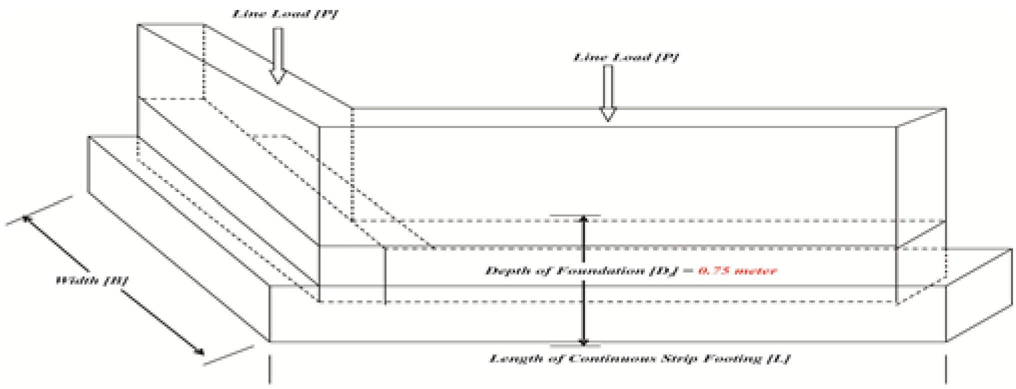

Fig. 8: Schematic of dimensions of a Continuous Strip Footing

The Raft Footings: Raft footings are recommended for the gas plant buildings. The soil bearing capacity of the raft footing ranged from 103.20kpa to 120.40kpa (table 9) and these values falls in the range of loose gravel according to [34] classification (table 5). The design considerations include: (i) the depth of the foundation footing (Df) below ground level, (ii) the width of base of the raft footings (B), and (iii) the length (L) of the raft footings (figures 9 & 10). With reference to the boring and sounding records, the raft footings should be located at a depth of 1.70m where 1.50m should be situated below the ground level and 0.20m above the ground level. This depth corresponds to the brownish lateritic clayey sands (SC) layer and implies that approximately 1.50m of this overburden would be excavated or removed during the foundation construction phase. The width of the raft footings was estimated to be B = (y+2x) metres, where y represents the width of the building's base and x is an additional 1.00m to the building's base width. The length of the raft footing is the same as the length of the gas processing plant building.

The bearing capacities of the soils at the various boring sites for the design of rigid raft footings were computed using several classical methods such as (i) the SPT method, (ii) Terzaghi's method, (iii) Meyerhof's method, (iv) Bowles method, and (v) Brinch Hansen's method as presented in table 4.

(i) The allowable net soil pressure for the design of rigid raft based on SPT field data may with sufficient accuracy be taken as:

q_a (tons / sq. ft) = 0.22 N ……………………………………………………………………………….. 20

where: N = SPT value (corrected for Water Table effects)

At a depth of 1.50 meters during boring, an average value of SPT N-value of 17 was obtained such that:

q_a (tons / sq. ft) = 0.22 (17) = 3.74 tons / sq. ft = (3.74) (0.1073) = 0.4013 MPa = 401.302 KPa.

q_a (tons / sq. ft) = 1/FS (0.22 N) ……………………………………………………………………………….. .. 21

For a Safety Factor of 3.0, the allowable bearing capacity becomes:

Allowable Bearing Capacity = 1/3 (401.302kpa) = 133.77 kPa

This value of bearing capacity can be conveniently used for a depth range of 1.50 - 2.50 meters.

(ii) Using the [27] based on laboratory results from soil testing, we have:

$q_u = q_c / F.S = 1/F.S \{\{ (1\text{-}0.2\ B/L)\ \gamma\ B/L.N_\gamma \} + \{ (1 + 0.20\ B/L)\ c\ N_c\} + \{ (\gamma D_f\ N_q)\}\}$……………. .. .. .22

where: B = width of Raft Foundation = 5.00m; L = Length of Raft Foundation = 10.00m (assumed)

γ = unit weight of soil at foundation level; N_γ, N_c, N_q = Terzaghi bearing Capacity Factors (table 13)

$q_u = q_c / F.S = 1/3 \{ \{ (0.80 \times 0.50)\ 17.8\text{kPa} \times 0.50 \} + \{ (1.20 \times 0.50)\ 50.00\text{kPa} \times 8.40 + \{(17.8\text{kPa})\ (1.50\text{m})\ (2.50)$

= 1/3 { 322.31 } = 107.44 kPa.

(iii) Applying the equation/formula of [28] at depth of 1.50m and SPT N-value of 17 minimum at the field, we have:

$q_u\ (\text{kN/m}^2) = 12N(B+0.305/B)^2$ ………………………………………………………………………………. 23

$q_u\ (\text{kN/m}^2) = (12 \times 17)\ \{ (5.00 + 0.305) / 5.00 \}^2 = 229.647\ \text{kN/m}^2$ [where B ~ 5.00m, assumed]

q_u (kN/m^2) = 229.647 kPa

Applying a factor of safety of 3.0 gives,

$q_{Allow} = 1/3(229.647) = 76.549$ kN/m^2
(iv) Using [29] at a depth of 1.50m, minimum SPT N-value of 17 we have:
$q_u = 12.5N(B+0.305/B)^2\ Kd$………………………………………………………………………………. 24
where:
$Kd = 1+ 0.33(D/B) \leq 1.33$ ……………………………………………………………………………… 25
$= 1+ 0.33(1.50/5.00) = 1.099$
$\therefore q_u = 12.5 (17) \{ (5.00 + 0.305) / 5.00 \}^2 (1.099) = 262.898$ kPa
For a Factor of Safety of 3.0, .$q_{(allow)} = q_u$ / F.S = 262.898 / 3.0 = 87.6326 KPa.
(v) The Ultimate Capacity according to [30] was computed thus:
$Q_u = -c\ Cot.\phi + (\Delta q + c\ Cot.\phi)\ N_q\ s_q\ d_q + 0.5\ B\ N\gamma\ s\gamma\ d\gamma$……………………………………….. .. 26
.$q_f = cN_c\ s_c d_c i_c b_c g_c + p_o\ N_q\ s_q d_q i_q b_q g_q + 0.50\ \gamma\ B\ N_\gamma\ s_\gamma\ d_\gamma\ i_\gamma\ b_\gamma\ g_\gamma$ ………………………………………….. .. 27
where, γ = density of soil below foundation level, B = width of foundation, c = undrained cohesion of soil
$p_o = [\gamma_{soil} - \gamma_{water}]\ D_f = [17.80 – 1.00]$

The effective pressure of overburden soil at foundation level is denoted as $\mathbf{D_f}$, which is the depth of the foundation and is a variable. The bearing capacity factors $\mathbf{N_\gamma}$, $\mathbf{N_q}$, and $\mathbf{N_c}$ correspond to different aspects of the foundation design. Additional factors include:
$\mathbf{s_\gamma}$, $\mathbf{s_q}$ and $\mathbf{s_c}$ are shape factors
$\mathbf{d_\gamma}$, $\mathbf{d_q}$, and $\mathbf{d_c}$ are depth factors
$\mathbf{i_\gamma}$, $\mathbf{i_q}$, and $\mathbf{i_c}$ are load inclination factors
$\mathbf{b_\gamma}$, $\mathbf{b_q}$, and $\mathbf{b_c}$ are base inclination factors; and
$\mathbf{g_\gamma}$, $\mathbf{g_q}$, and $\mathbf{g_c}$ are ground surface inclination factors.

For a horizontal ground surface with a horizontal footing base, all inclination factors (ground, base and load) are set at unity. Assuming a change in pressure (Δq) of 200 kPa, the relevant factors and soil conditions can be substituted into equation 26 to calculate the bearing capacity which gives as follows;

$q_u = \{ -50.00 \text{ kPa} [\text{Cot } 10^o]\} + \{ (200 \text{ kPa} + 50.00\text{kPa} (\text{Cot } 10^o)) (2.50)(1)(1)\} + \{ 0.50(5.00)(0.50)(1)(1)\}$
$= 626.586$ kPa
$q_{(allowable)} = qu / F.S = \underline{208.862}$ KPa

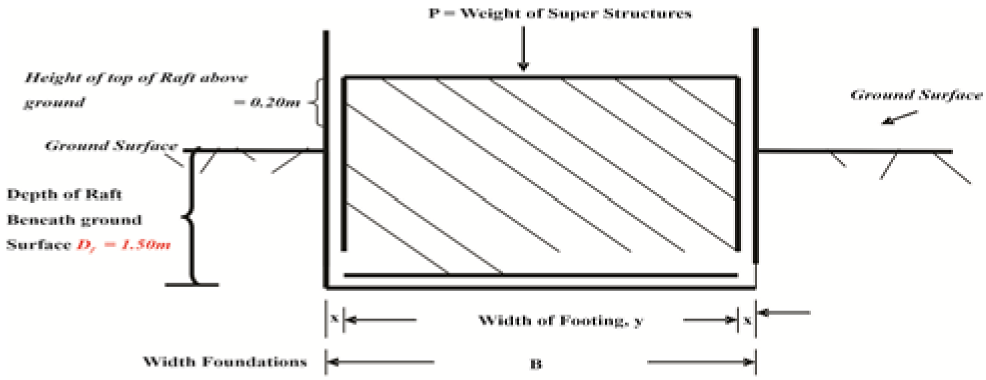

Fig. 9: Schematic Representation of the Gas Processing Plant Building on Raft Foundation

The bearing capacity values for the raft at a foundation depth of 1.50m calculated using various classical geotechnical methods as shown in table 4 revealed that they fall below the upper limits for bearing capacity which ranged from 380kPa to 470kPa cautioned by [31]. From table 12 it is also evident that the range of useable soil bearing capacity values for construction based on a foundation depth of 1.50m and a base width (B) of 5.0m is between 76.55kPa and 208.86kPa with an average value of 139.47kPa.

Given that the soil profiles are quite homogeneous, a sensitivity analysis of the soil bearing capacity for both isolated and Raft Footings was conducted for foundation depths of 0.50m, 1.00m, 1.50m, 2.00m and 2.50m (table 4). The net pressure on footing with backfill goes with the assumption that top of Raft footing is above ground surface and it is illustrated in figure 10.

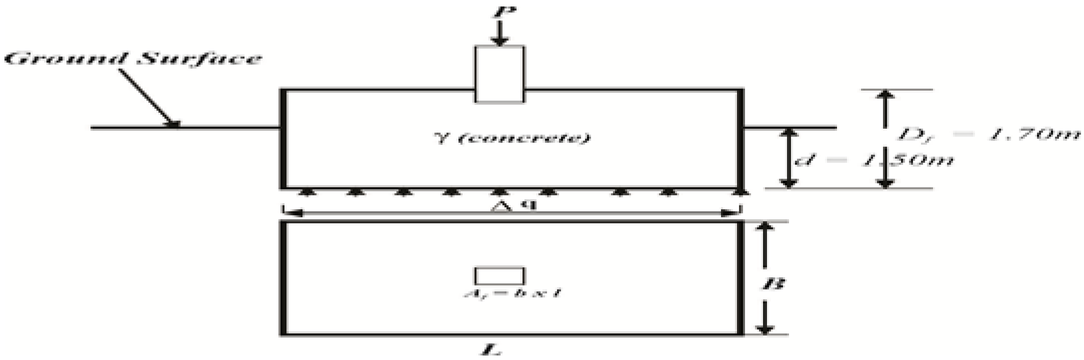

Fig. 10: Net Pressure on footing with backfill (assuming top of Raft footing above ground surface)

V. Conclusion

The study investigated the geotechnical properties of the underlying soil at Ovade-Ogharefe for the design and construction of a gas processing plant and other civil infrastructures in order to prevent building failure and collapse. Two primary soil profiles were identified: the brownish lateritic clayey sand layer (Sc) and the grayish well-graded sands and gravels (Sw). Various geotechnical engineering properties of each sub soil were analyzed, leading to the recommendation of shallow foundation options for the office complex, residential buildings and the gas processing plant. The bearing capacity for isolated footings was found to be approximately 195.35+1.1867B(kPa) or 201.284kPa with a width of 5m. For continuous strip footings, the bearing capacity was about 153.35+1.483B(kPa) which resulted to 160.76kPa with a depth of 5m. The bearing capacity for raft footings ranged from 76.55kpa to 208.86kPa with an average of 139.47kPa. The study also found that 50% of the settlement would occur about 3.70 years after construction, while 90% of the settlement would occur after about 15.72 years following the completion of the gas plant. Given that the gas processing plant will be vibrating continuously, soil dynamics analysis which included Poisson's ratio (0.45 to 0.50), Shear Modulus G of 4.50 MPa and Modulus of Elasticity E_s of 62,500MN/m^2 was run. The result showed that there was no possibility of liquefaction occurring at the site within the three (3) earthquake magnitudes M= 6.0, 7.5 and 8.25 except for earthquake magnitudes M=6.0 (N=17) and M=7.5 (N=21, 22, 25) that respectively have marginal, high and moderate Liquefaction potential risk due to low N-value and high CSR. It is recommended that sub-soil layers of high and moderate susceptibility to potential liquefaction risk be improved by either compaction or stone columns, and the depths for isolated footings, continuous strip footings and the raft footings be respectively 1.50m, 0.75m and 1.50m to align with the safe bearing capacity of the upper lateritic clayey sands. To prevent building failure and collapse it is crucial to incorporate geotechnical investigations into disaster risk management planning. This will help reduce the impact of disasters on communities, enhance resilience and enable engineers to select the most appropriate foundation designs. Geotechnical investigations provide valuable information about soil conditions, building loads, external forces to maintain structural integrity and ensure the long-term safety of buildings.

List of abbreviations

SPT	Standard Penetration Test
LL	Liquid Limit
PL	Plastic Limit
SC	Brownish lateritic Clayey Sand
Sw	Brownish well graded sands and gravels
Kpa	Kilo pascal
Mpa	Mega pascal

Declarations:

Availability of data and materials

All data supporting this research are included in the article. However, it can be further provided on demand from the corresponding author.

Competing interests

The authors declare that they have no competing interests.

Ethics, Consent to Participate, and Consent to Publish declarations

Not applicable

Ethics declaration
Not applicable

Ethics and Consent to Publish declarations
Not applicable

Clinical trial number
Not applicable

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Authors' contributions

TC developed the research framework and prepared the manuscript. C, DD and N performed the experimental setup and analyzed the results. CO, OD and BA further analyzed the results and improved on the theoretical aspects of this paper. All authors read and approved the final manuscript and agree to be accountable for all aspects of the work in ensuring that questions related to the accuracy or integrity of any part of the work are appropriately investigated and resolved.

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References

[1] Ching, F. D. K. (2014). Building Construction Illustrated. John Wiley And Sons.
[2] International Code Council (ICC). (2018). 2018 International Code Council Performance Code For Buildings And Facilities. Retrieved Online From Https://Www.Scribd.Com. (Accessed: 2nd October, 2024).
[3] Staff W. L. (2020). Hazards: Infrastructure Failure-Building Collapse In Hard Rock Hotel, Https://Ready.Nola.Gov/Hazard-Mitigation/...(Accessed 10 November 2024).
[4] News Agency Of Nigeria (2024). Nigeria Records 22 Building Collapses, 33 Deaths In 7 Months-COREN. Business Day, 18 July 2024, Retrieved Online From Https://Businessday.Ng/News/Article/Ni...(Accessed 10 November, 2024)
[5] Ogundeji J. (2023). 553 Buildings Collapsed In 49 Years-BCPG. Punch, 13 August 2023, Retrieved Online From Https://Punchng.Com. (Accessed 10 August, 2024).
[6] Okunola O. H. (2022). Quantifying Frequent Building Collapse And Disaster Risk Reduction In Nigeria. Retrieved Online From Https://Www.Brookings.Edu/Blog/Africainfocus (Accessed 16 July, 2024).
[7] Eniola D. (2022). Casualty Figure Unknown As Another Building Collapses In Lagos. Guardian, 12 February 2022, Retrieved Online From Https://Www.Guardian.Ng/News/Building (Accessed 16 July 2024)
[8] Obiezu T. (2022). Building Collapse In Southern Nigeria Kills 3, Including 2 Children. VOA News, 12 January 2022. Retrieved Online From Https://Www.Voanews.Com (Accessed 16 July, 2024)
[9] Afeez H, Godfrey G. (2022). Another Building Collapses In Lagos. Punch, 13 February 2022, Retrieved Online From Https://Punchng.Com. (Accessed 10 May 2024).
[10] Ihua-Maduenyi M. (2020). Nigeria Recorded 43 Building Collapse Cases In 2019. Punch, 27 January 2020, Retrieved Online From Https://Punchng.Com. (Accessed 1 July 2024).
[11] THE POINTER Editorial (2024). Increased Cases Of Building Collapse In Nigeria. The Pointer, 27 September 2024, Retrieved Online From Https://Www.Thepointernewsonline.Com. (Accessed: 12 November 2024)
[12] Orusi K. (2019). One Dies, Others Injured In Collapsed Building In Delta State University. The Nigerian Voice, 18 September 2019, Retrieved Online From Https://Www.Thenigerianvoice.Com. (Accessed 1 July, 2024).
[13] Osuyi P. (2019). Four Die As Hotel Collapses In Delta. The Sun, 21 July 2019, Retrieved Online From Https://Www.Sunnewsonline.Com. (Accessed 1 July, 2024).
[14] Ya'U Madobi M. (2021). Building Collapse In Nigeria: Time To Stem The Tide. The Cable News, 10 November 2021, Retrieved Online From Https://Www.Thecable.Ng-Collapse (Accessed 16 July, 2024)
[15] Awoyera P. O, Alfa J, Odetoyan A, Akinwunmi I. I. (2021). Building Collapse In Nigeria During Recent
[16] Years-Causes, Effects And Way Forward. IOP Conference Series: Materials Science And Engineering, 2021.
[17] Fagbenle O. I. And Oluwunmi A. O. (2010). Building Failure And Collapse In Nigeria: The Influence Of The Informal Sector. J Sustainable Developments, 3(4): 268-276.
[18] Folagbade S. O. (2009). Case Studies Of Building Collapse In Nigeria: Proceedings Of A Workshop On Building Collapse-Causes, Prevention And Remedies, The Nigerian Institute Of Building, Ondo State Chapter, 23-24 October 2009.
[19] Badejo E. (2009). Engineers, Others Urge Multi-Disciplinary Approach To Curb Building Collapse Of Buildings: Proceedings Of The National Conference On Effective Contract Management In The Construction Industry, The Nigerian Institute Of Building, 22-23 August 2009, Pp. 188-200.
[20] Youdeowei P. O. (1995). Report On Sub-Soil Investigations For Foundation Of Blocks Of Residential Buildings, GRA. Phase 3, Port-Harcourt, Bimkol Engineering Nigeria Limited, Port-Harcourt, Nigeria, June 1995
[21] Youdeowei P. O. And Nwankwoala H. O. (2013). Suitability Of Soils As Bearing Media At A Freshwater Swamp Terrain In The Niger Delta. Journal Of Geology And Mining Research, 5(3): 58-64.
[22] Ezenwaka K. C, Ugboaja A, Ahaneku C. V And Ede T. A. (2014). Geotechnical Investigation For Design And Construction Of Civil Infrastructures In Parts Of Port-Harcourt City Of Rivers State, Southern Nigeria. The International Journal Of Engineering And Sciences, 3(8): 74-82.

[23] ASTM D 423 (1986). Recommended Practice For Investigation And Sampling Soil And Rock For Engineering Purposes. In Annal Book Of ASTM Standards D423.
[24] AASHTO T-89 (2004). Standard Method Of Test For Determining Soil And Rocks For Engineering Purposes.
[25] B.S 1377, Part 2 (1990). British Standard Methods Of Test For Soils For Civil Engineering Purposes. British Standard Institute (BSI): London. Pp. 8-200
[26] Peck R. B, Hansen W. E And Thornburn T. H (1974). Foundation Engineering. 2nd Edition. New York: John Wiley & Sons Incorporation, P.514.
[27] Tomlinson M.J. (1980). Foundation Design And Construction. 4th Edition. London: Pitman Advanced Publishing Programme, P793.
[28] Terzaghi K. (1943). Theoretical Soil Mechanics In Engineering Practice. 2nd Edition. New York: John Wiley, Pp. 271 – 276.
[29] Meyerhof G. G. (1963). Some Recent Research On Bearing Capacity Of Foundations. Canadian Geotechnical Journal. 1: 16 – 26.
[30] Bowles J.E. (1985). Physical And Geotechnical Properties Of Soils. 2nd Edition. Mcgraw-Hill Book Company
[31] Hansen, J. B. (1968). A Revised Extended Formula For Bearing Capacity. Danish Geotechnical Institute Bulletin, No. 28.
[32] Bowles, J.E. (1977). Foundation Analysis And Design. 2nd Edition. New York: Mcgraw-Hill Book Company, P.750.
[33] Skempton A. W. And Macdonald H. H. (1956). The Allowable Settlement Of Buildings Of Civil Engineers,
[34] Proceedings Of Civil Engineers, 3(5): 727 – 784.
[35] Abubakar M. (2024). Collapse After Collapse-Why Lagos Building Keep Crashing Down. BBC News, Lagos, 15 September 2024. Retrieved Online From Https://Www.Bbc.Com/News/Articles/Cx... (Accessed 12 November 2024).
[36] Lee A. (2024). Bearing Capacity Of Soil: Types And Calculations. Tensar International Limited. Retrieved Online From Https://Www.Tensar.Co.Uk. (Accessed 27 January, 2025).
[37] Fajana A.O, Olaseeni O.G, Bamidele O.E And Olabode O.P (2016). Geophysical And Geotechnical Investigation For Post Foundation Studies, Faculty Of Social Sciences And Humanities, Federal University Oye Ekiti. FUOYE Journal Of Engineering And Technology, 1(1): 62-66
[38] Federal Ministry Of Works And Housing (FMWH) (1972). Highway Manual Part 1: Road Design. Federal Ministry Of Works And Housing, Lagos.
[39] Delta State Ministry Of Lands, Survey And Urban Development, Asaba. Retrieved Online From Https://Www.Landsdeltastate.Gov.Ng (Accessed 12 November, 2024).
[40] Seed H.B, Tokimatsu K, Harder L.F And Chung R.M (1985). Influence Of SPT Procedures In Soil Liquefaction Resistance Evaluations. Journal Of Geotechnical Engineering, 111(12): 1425-1445. Https://Doi.Org/10.1061/(ASCE)0733-9410(1985)111:12(1425).
[41] Idriss I.M And Boulanger R.W (2008). Soil Liquefaction During Earthquakes. Earthquake Engineering Research Institute (EERI), MNO-12. California: Oakland. ISBN: 978-1-932884-19-4.
[42] Youd T.L, Idriss I.M, Andrus R.D, Arango I, Castro G, Christian J.T, Dobry R, Finn W.D.L, Harder L.F, Hynes M.E, Ishihara K, Koester J.P, Liao S.S.C, Marcuson W.F, Martin G.R, Mitchell J.K, Moriwaki Y, Power M.S, Robertson P.K, Seed R.B And Stokoe K.H (2001). Liquefaction Resistance Of Soils: Summary Report From The 1996 NCEER And 1998 NCEER/NSF Workshops On Evaluation Of Liquefaction Resistance Of Soils. Journal Of Geotechnical And Geoenvironmental Engineering, 127(10): 817-833. DOI: [10.1061/(ASCE)1090-0241 (2001)127:10(817)] (Https://Doi.Org/10.1061/(ASCE) 1090-0241 (2001) 127: 10(817).