Original Article

GEOTECHNICAL INSIGHTS ON SHALLOW FOUNDATIONS IN SEDIMENTARY TERRAINS: FINDINGS FROM BONNY ISLAND

Samuel C. Ebikeme[1] and Engr. Juliet A. Tamuno[2]

[1]Department of Industrial Safety and Environmental Engineering Technology[1] Federal Polytechnic of Oil and Gas, Bonny Island, Rivers, Nigeria
[2]Department of Petroleum and Gas Engineering Technology, Federal Polytechnic of Oil and Gas, Bonny Island, Rivers, Nigeria
DOI: https://doi.org/10.5281/zenodo.13789832

Abstract: This study investigated the Geotechnical assessment of shallow foundation in sedimentary deposits. The samples were taken by drilling with the aid of percussion drilling rig to the maximum depth of 2m, and were further subjected to various laboratory analysis. According to the findings, the lithographic sequence consists of fine sand to dense sand. The soil has a moisture content of 17.5-18.8%, the particle size distribution analysis showed poorly graded sand. Values of the undrained cohesion is 0 kPa showing cohesionless nature of the soil and angle of internal friction are between 32º to 33º indicating good shear strength properties. Bearing capacity for the homogenous subgrade ranged from the top soil to 1 m depth below the top soil, 218 kN/m2 to 753 kN/m2, and to 2m depth below 1m it ranges from 427 kN/m2 to 1064 kN/m2, this suggested very adequate strength for load bearing structures. Based on the assessment, a shallow foundation with a depth of 1or 1.5 meter is recommended within the allowable bearing pressure stated above. A shallow foundation with a spread footing or mat foundation is suitable for this site, the spread footing can transfer the load to the required depth, while the mat foundation can distribute the load over a larger area, reducing the pressure on the soil.

Keywords: Foundation, Geotechnical, Sedimentary, Sub-soil, Bonny

1. INTRODUCTION

The soil strata within the Niger Delta region are naturally made up of some sedimentary deposits that sometimes affect the foundation of the Civil engineering structures over the years. The southern part lies higher and contains typically freshwater swamp forest, in some places mixed with mangrove, open swamp, dry land rainforest or dune slack (EAU, 1992). However, the depth of a building's foundation varies depending on the nature and size of the structure to be constructed. This is indispensable in Bonny Island and its environs which have challenging terrain with 70% of the total land area, 214.52 m^2 suffering tidal flooding and land subsidence (NLNG, 2005). In the world at large, soil assessment is a vital process in determining the information needed on the soil for the design of the foundation of structures such as buildings, roads, bridges, dams, etc.; not adhering to soil investigation report strictly can sometimes lead to structural failure in major and minor civil engineering projects. A properly designed foundation transfers the load throughout the soil without overstressing the soil (Das, 2010). As development plans are introduced, the expansion of physical infrastructure to support the growing population and economic activities becomes essential. This growth requires a comprehensive understanding and addressing of geotechnical challenges, especially those associated with sedimentary deposits in Bonny Island and its surrounding areas. These deposits can create considerable engineering obstacles for shallow foundations within the area. Most soils/rocks are usually assumed to be very good foundation materials but their heterogeneous nature caused by the existence of faults, voids, cracks, fractures or joints filled with fluids or organic matter can reduce their bearing capacity (Alaminiokuma et al, 2018).

The investigation employed Percussion drilling and Cone Penetration Testing (CPT) to evaluate the soil's bearing capacity and determine the optimal design for a secure and stable building foundation. This research focuses on a geotechnical analysis, particularly of shallow foundations, as they are the preferred choice for construction on Bonny Island.

2.0 DESCRIPTION OF THE STUDY AREA

The study site is located within the premises of the FPOG, Bonny Island, Rivers State, in the South South region of Nigeria. This location can be accessed only by water and air transportation due to its unique terrain. Geologically, the site is part of the Niger Delta sedimentary basin, a region known for its complex sedimentary structures and rich natural resources (Allen, 1965). Based on the geographic coordinates system, the study site is approximately 4° 25′ 56.796″ N 7° 11′ 34.477″ E, 4° 25′ 59.791″ N 7° 11′ 39.896″ E, 4° 25′ 47.785″ N 7° 11′ 40.827″ E, and 4° 25′ 47.871″ N 7° 11′ 33.312″ E for latitude and longitude respectively. The sedimentary deposits in this location primarily consist of yellowish gray dense sands. The sands are generally well-sorted and are inter bedded with silty and clayey layers, reflecting changes in the energy conditions of the depositional environment over time.

Bonny Island is positioned within the beach ridge onshore geomorphic sub-environment of the Niger Delta. This area is characterized by a series of sandy ridges formed parallel to the coastline, which are elevated above the surrounding deltaic plain. These ridges are significant as they influence the local hydrology and vegetation patterns (Short & Stauble, 1967). The terrain's constant exposure to such forces results in a continually evolving landscape, influencing both soil composition and stability.

.

Original Article

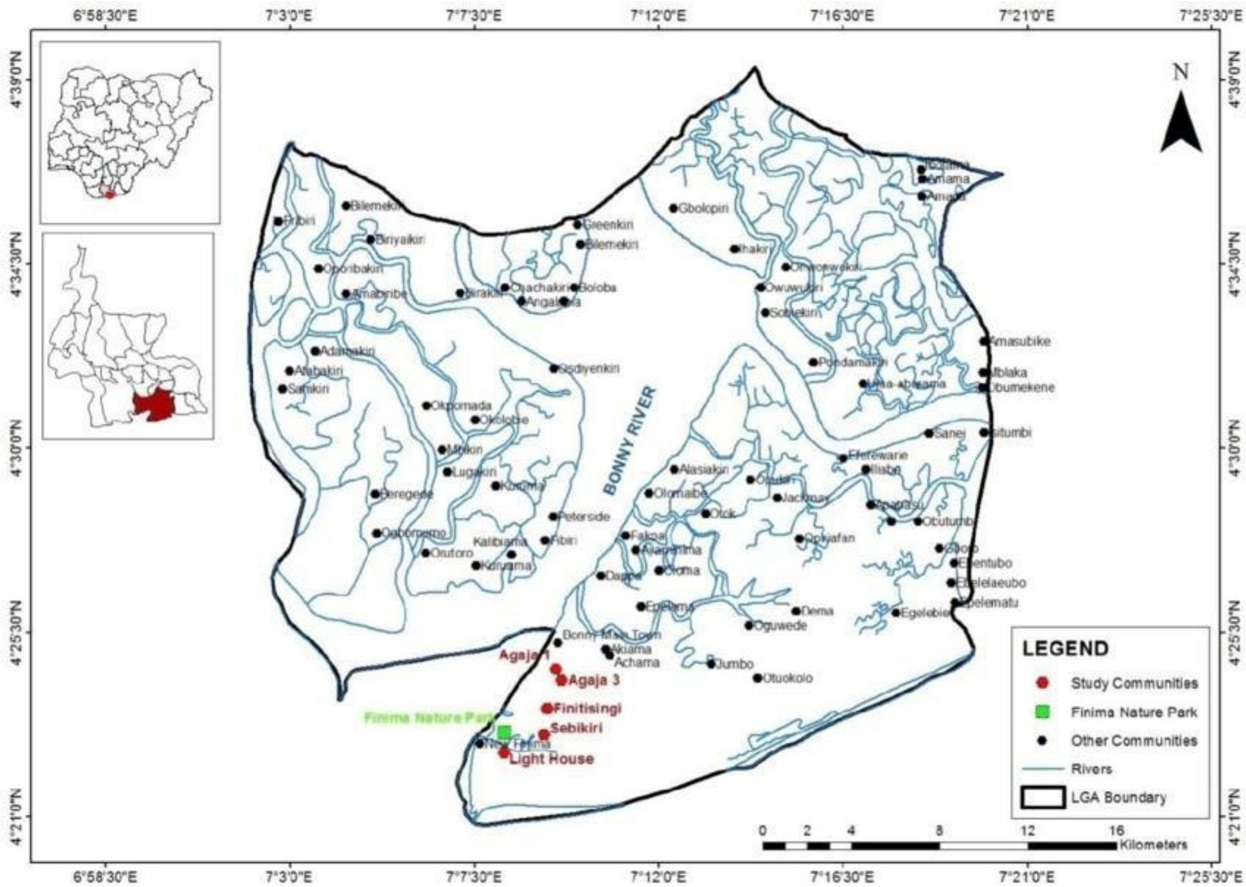


Fig 1: Map of Bonny Local Government Area

3.0 MATERIALS AND METHOD

Before commencing the fieldwork, a comprehensive reconnaissance survey was carried out to evaluate the entire area thoroughly. This preliminary step was crucial as it ensured the collection of adequate data necessary for developing a detailed study plan. The initial phase of the work involved a meticulous review of the topography and geology maps of the location, which provided a foundational understanding of the area's physical characteristics.

Once the fieldwork began, soil samples were collected at designated drilling points. For this specific study, samples were taken from top to a maximum depth of 2 meters at a single location. This precise depth was chosen to provide relevant data for the analysis. At the end of the drilling session, a rigorous cleaning of all equipment used was performed. This procedure was vital to prevent the introduction of any extraneous materials that could contaminate subsequent samples during further drilling operations. Throughout the fieldwork activities, meticulous documentation was maintained. Relevant data, including dates, coordinates, elevations, distances, locations, boreholes, and depth intervals, were recorded in detail. This documentation process ensured that all significant information was captured accurately and could be referred to during the analysis phase. After field activities concluded each day, the recorded data were carefully transcribed into a datasheet for organization and future reference. The collected soil specimens were then transported to the soil laboratory for a comprehensive analysis. This step was essential to determine the various properties and characteristics of the soil samples. The laboratory analysis provides critical insights into the soil composition, which is necessary for the study's objectives

3.1 TESTS FOR CLASSIFICATION

The soil specimens were subjected to classification tests that strictly adhered to all pertinent geotechnical engineering standards, including the British Standards (BS 1377, 1990). To validate and enhance the field identification process, a series of laboratory classification tests were conducted on numerous soil samples. These tests included natural moisture content, Atterberg limits, grain size distribution, hydrometer analysis, specific gravity, and permeability.

The formula for Water content, Atterberg Limit, Void ratio, Porosity, Unit weight, Specific gravity in equation (1-6) were evaluated from the method of (Bowles, 1997) and Permeability in equation (7) as investigated by (Kozeny, 1927; Carman, 1939)

Moisture Content = Mass of water x 100 Dry soil mass (Ms) (1)

Coefficient of uniformity (Cu) = D_{60}/D_{10} (2)

Coefficient of concavity Cc = D_{30}^2

D60 ⃞ D10

Permeability (Allen Hazen's Formula) K = C * $D10^2$ Specific Gravity (Gs) = Mass (3) of soil particles

Mass of equal volume of water (4)

3.2 The Bearing Capacity Formula

The findings from the Shear box test were evaluated for unit weight, undrained cohesion, and internal friction angle. Terzaghi's equation (6) (1943) and Buisman (1935; 1940) was employed to calculate the ultimate bearing capacity of the foundation subgrades. To ensure the total settlement in the study area remained within acceptable limits, a safety factor of 3.0 was applied, using equation (7) to determine the allowable bearing capacities of the soil. The premise of the formula is that the footing foundation has infinite length (is continuous) and the load is vertical and concentric with the footing center line, the soil is homogeneous, and the ground surface is horizontal.

$qu = cNC + \gamma 1DfNq + 0.5\gamma 1BN\gamma$ (6) where, qult = ultimate unit resistance of the footing c' = effective cohesion intercept

B = footing width Df = depth of foundation

$\gamma 1$ = average effective unit weight of the soil below the foundation

Nc, Nq, $N\gamma$ = non-dimensional bearing capacity factors c = cohesion of the soil (ksf) (kPa)

q = total surcharge at the base of the footing = $q_{appl} + \gamma_a D_f$ (ksf) (kPa) q_{appl} = applied surcharge (ksf) (kPa)

γ = unit weight of the overburden material above the base of the footing causing the surcharge pressure (kcf) (kN/m3)

D_f = depth of embedment (ft) (m)

γ = unit weight of the soil under the footing (kcf) (kN/m3)

Bf = footing width, i.e., least lateral dimension of the footing (ft) (m)

Nq = bearing capacity factor for the "surcharge" term (dimensionless) = $e^{\pi \tan \phi} \tan^2 (45^0 + \phi/2)$

Nc = bearing capacity factor for the "cohesion" term (dimensionless)

= (Nq – 1) cot ϕ for $\phi > 0°$ = 2 + π = 5.14 for $\phi = 0°$

$N\gamma$ = bearing capacity factor for the "weight" term (dimensionless)
= 2 (Nq + 1) tan (φ)
qa = qu / F.S (7)
Where qa = Allowable Bearing Pressure qu = Ultimate Bearing Capacity
F.S = Factor of safety
Allowable net soil pressures (8) qu = 0.22 N (0.1073) X 1000 kN/m2

$$Cu = \frac{q_c - \square^1_{vo}}{N_k} \qquad (9)$$

Where, qc - □'$_{vo}$ = net cone resistance
N_k = Cone factor

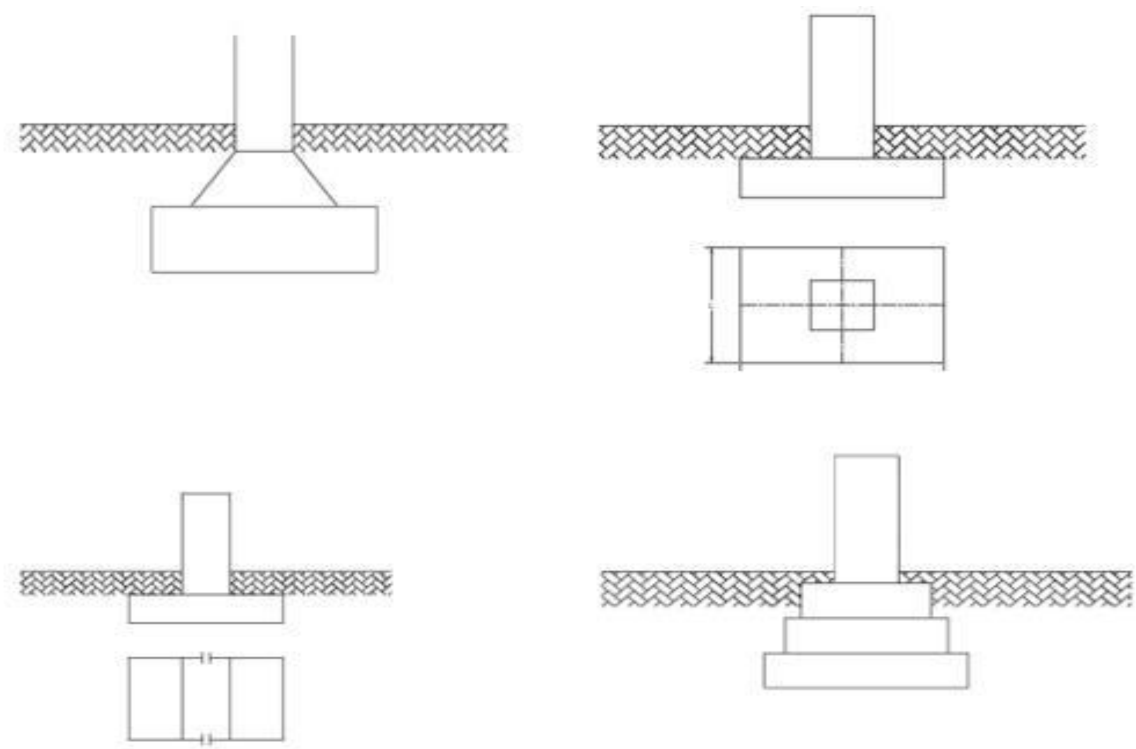


Fig 2: Typical shallow foundation

4.0 RESULTS AND DISCUSSION (a) Sub-Soil Lithostratigraphy

The lithographic sequence shown in Figure 3 illustrates a stratigraphy characteristic of soil engineering. At the surface, there is a layer of very light gray fine sand, which transitions into yellowish gray fine sand at a depth of 1 meter. At a depth of 2 meters, the sequence culminates in yellowish gray dense sand.

Original Article

● DISTURBED SAMPLE
▼ SPT SOUNDING

Method of Drilling: Percussion
Static water Level :1.3m
Date:03/06/ 2024

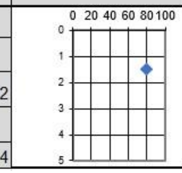
SOIL PROFILE				TEST RESULTS								
DEPTH (m)	DESCRIPTION	STRATA PLOT	SAMPLE	SPT-N	Moisture	Unit Weight	Undrained Cohesion	Internal Friction	Specific Gravity	SPT (N) VALUES (blows/0.3m)		
				Blows	Cont (%)	Wt (kN/m3)	(kN/m^2)	Deg ^{0}C				
Top	Very Light Gray Fine Sand		●									
1	Yellowish Gray Medium Sand		●		17.5	16.6	0	32	2.62		SPT(N)	Depth(m)
1.5	Yellowish Gray Dense Sand		▼	14/30/50							80	1.5
2	Yellowish Gray Dense Sand		●		18.8	16.6	0	33	2.64			

Fig 3: Borehole log

(b) Moisture Content:

At 1 meter depth, the fine sand has a moisture content of 17.5% and at 2 meters depth, the dense sand has a moisture content of 18.8%. The moisture content values indicate a moderate to high water content in the soil, with a slight increase in moisture content with depth. The fine sand at 1 meter has a relatively lower moisture content compared to the dense sand at 2 meters. This could be due to the difference in soil density and particle size, with the denser sand at 2 meters retaining more water.

(c) Particle Size Distribution

At 1 meter depth, the Coefficient of Uniformity (Cu) is 1.6666 and Coefficient of Curvature (Cc) is 0.864. At 2 meters depth, the Coefficient of Uniformity (Cu) is 1.6470 and Coefficient of Curvature (Cc) is 0.8403. The particle size analysis results indicate that the soil at both depths is well-graded, with Cu values close to the ideal range of 1-3. This suggests a relatively uniform distribution of particle sizes, which is beneficial for shallow foundation design. The Cc values are also within the acceptable range (0.5-1.5), indicating a smooth and continuous particle size distribution curve. The slight variations in Cu and Cc values between the two depths may be due to the change in soil density i.e fine sand to dense sand. However, the overall similarity in values suggests that the soil's particle size distribution is relatively consistent with depth.

USCS Soil Classification: N/A
AASHTO Soil Classification: N/A
Weight of Dry Sample (g): 150

Sieve Number	Diameter (mm)	Soil Retained [g]	Cumulative weight retained [g]	Soil Retained (%)	Soil Passing (%)
3/8-in	9.5				
4	4.75				
8	2.36	0	0	0.0	100.0
16	1.18	1	1	0.7	99.3
30	0.6	3.8	4.8	2.5	96.8
50	0.3	19.4	24.2	12.9	83.9
100	0.15	103.9	128.1	69.3	14.6
200	0.075	20.1	148.2	13.4	1.2
Pan		0	148.2	0.0	0.0

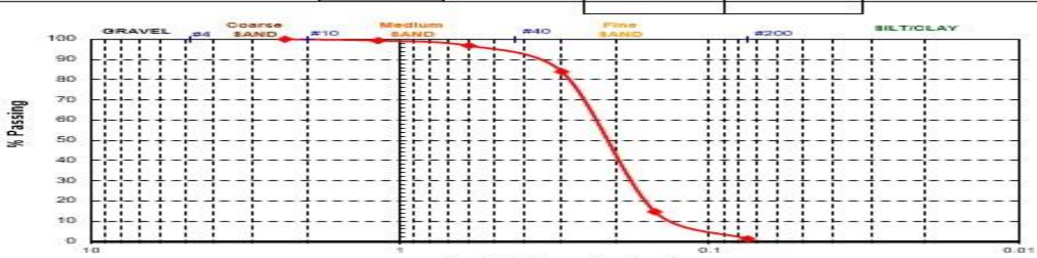


Fig 4: Grain Size Analysis for 1 meter

Original Article

USCS Soil Classification: N/A					
AASHTO Soil Classification: N/A					
Weight of Dry Sample (g): 300					
Sieve Number	Diameter (mm)	Soil Retained [g]	Cumulative weight retained [g]	Soil Retained (%)	Soil Passing (%)
3/8-in	9.5				
4	4.75				100.0
8	2.36	1.0	1	0.3	99.7
16	1.18	2.6	3.6	0.9	98.8
30	0.6	20.9	24.5	7.0	91.8
50	0.3	78.4	102.9	26.1	65.7
100	0.15	174.1	277	58.0	7.7
200	0.075	11.8	288.8	3.9	3.7
Pan		0	288.8	0.0	0.0

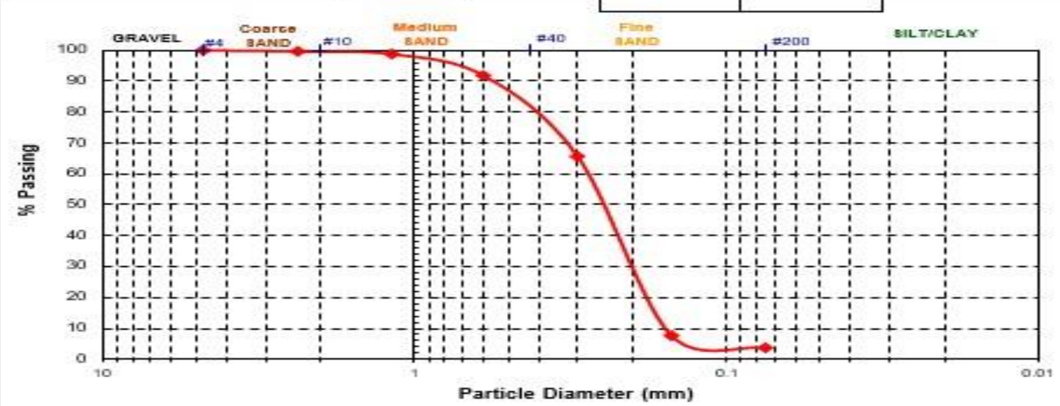


Fig 5: Grain Size Analysis for 2 meter **(d) Permeability:**

The coefficient of permeability (k) values obtained from the soil samples at 1 meter (2.25 x 10^-6 m/s) and 2 meters (2.89 x 10^-6 m/s) indicate a moderate to high permeability, which is typical for sandy soils. The increase in permeability with depth (from 2.25 to 2.89 x 10^-6 m/s) suggests a slight increase in the soil's ability to allow water to flow through it. This could be attributed to the change from fine sand to dense sand, which may have a more uniform pore structure, allowing for easier water flow. **(e) Specific Gravity:**

The specific gravity values obtained from the soil samples at 1 meter (2.62) and 2 meters (2.64) are relatively close, indicating a consistent soil composition with depth. The slight increase in specific gravity with depth suggests a slight increase in density, which is consistent with the change from fine sand to dense sand. The specific gravity values are within the typical range for sand soils, which is between 2.60 and 2.70. This suggests that the soil has a moderate to high density, which is suitable for shallow foundation design.

Original Article

SPECIFIC GRAVITY DETERMINATION (FINE AGGREGATE) Standard: BS 812 -2				
BH NO:	1	Depth(m):	1.0	
Observations				
Items	Bottle Number		1	2
1	Mass of bottle + soil + water (M_3)	(g)	106.20	106.10
2	Mass of bottle + soil (M_2)	(g)	76.50	76.50
3	Mass of bottle full of water (M_4)	(g)	75.30	75.30
4	Mass of bottle (M_1)	(g)	26.60	26.60
5	Mass of water used (M_3 - M_2)	(g)	29.70	29.60
6	Mass of soil used (M_2 -M_1)	(g)	49.90	49.90
7	Vol. of soil (M_4 -M_1) - (M_3 - M_2)	(ml)	19.00	19.10
8	$Gs = \frac{(M_2 - M_1)}{(M_4 - M_1) - (M_3 - M_2)}$		2.63	2.61
	AVERAGE		2.62	

Fig 6: Specific Gravity for 1 meter

SPECIFIC GRAVITY DETERMINATION (FINE AGGREGATE) Standard: BS 812 -2				
BH NO:	1	Depth(m):	2.0	
Observations				
Items	Bottle Number		1	2
1	Mass of bottle + soil + water (M_3)	(g)	106.80	106.70
2	Mass of bottle + soil (M_2)	(g)	77.20	77.20
3	Mass of bottle full of water (M_4)	(g)	75.30	75.30
4	Mass of bottle (M_1)	(g)	26.60	26.60
5	Mass of water used (M_3 - M_2)	(g)	29.60	29.50
6	Mass of soil used (M_2 -M_1)	(g)	50.60	50.60
7	Vol. of soil (M_4 -M_1) - (M_3 - M_2)	(ml)	19.10	19.20
8	$Gs = \frac{(M_2 - M_1)}{(M_4 - M_1) - (M_3 - M_2)}$		2.65	2.64
	AVERAGE		2.64	

Fig 7: Specific Gravity for 2 meter

(f) Shear box test

At 1 meter depth, the angle of Internal Friction (φ) is 32° and Cohesion (c) is 0 kPa. At 2 meters depth, the angle of Internal Friction (φ) is 33° and Cohesion (c) is 0 kPa. The shear box test results indicate that the soil at both depths is a cohesionless soil (c = 0 kPa), which is typical for sandy soils. The angle of internal friction (φ) values is relatively high, indicating good shear strength properties. The slight increase in φ from 32° to 33° with depth may be due to the increase in soil density from fine sand to dense sand. This suggests that the soil's shear strength improves with depth. The cohesionless nature of the soil indicates that the soil's shear strength is primarily frictional, relying on the angle of internal friction (φ) to resist shear stresses. These results suggest that the soil has suitable shear strength properties for shallow foundation design.

Original Article

Test and sample details	
Standard: EN ISO 17892-10:2018	
Bore Hole No: 1	Depth (m): 1.0
Specimen Size: 60mm x 60mm x 25mm	Sample Description: Fine sand
Type of Test: Consolidated Drained Shear box	Bulk Unit Weight (KN/m^3): 19.5
Moisture Content (%): 17.5	Dry Unit Weight (KN/m^3): 16.6

	TEST 1	TEST 2	TEST 3
Wt of Ring + Sample (g)	311.6	311.9	311.8
Wt of Ring (g)	133.2	133.2	133.2
Wt of Sample (g)	178.4	178.7	178.6
Normal Stress (KN/m^2)	100	200	300
Shear Stress Failure (KN/m^2)	67.2	132.0	183.3

Friction Angle (Φ)	32°
Cohesion, C (KN/m^2)	0

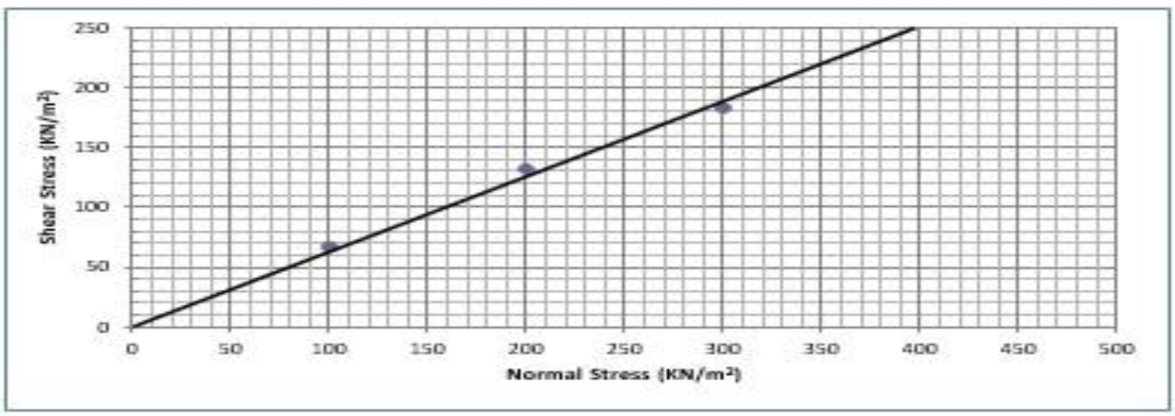


Fig 8: Shear Box for 1 meter

Test and sample details	
Standard: EN ISO 17892-10:2018	
Bore Hole No: 1	Depth (m): 2.0
Specimen Size : 60mm x 60mm x 25mm	Sample Description: Fine sand
Type of Test: Consolidated Drained Shear box	Builk Unit Weight (KN/m^3): 19.8
Moisture Content(%): 18.8	Dry Unit Weight (KN/m^3): 16.6

	TEST 1	TEST 2	TEST 3
Wt of Ring + Sample (g)	314.6	314.3	314.5
Wt of Ring (g)	133.2	133.2	133.2
Wt of Sample (g)	181.4	181.1	181.3
Normal Stress (KN/m^2)	100	200	300
Shear Stress Failure (KN/m^2)	67.2	135.7	195.6

Friction Angle (Φ)	33°
Cohesion, C (KN/m^2)	0

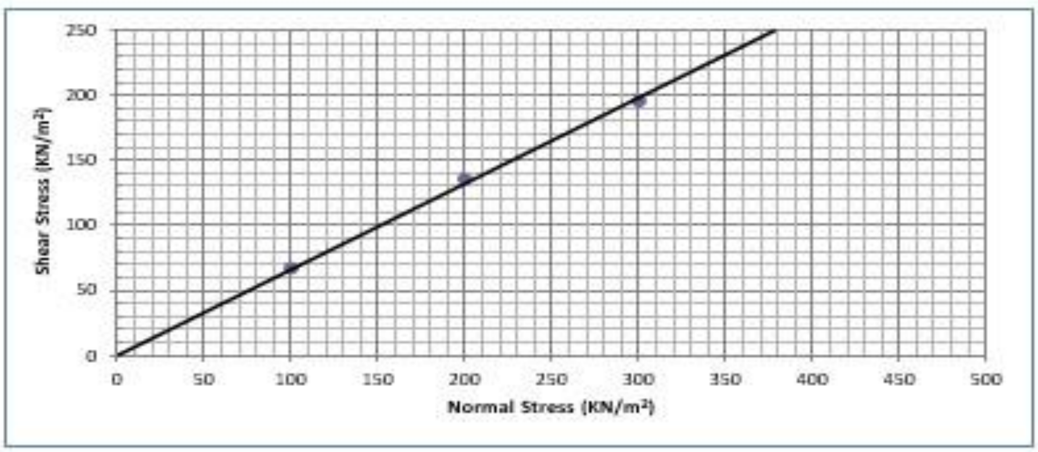


Fig 9: Shear Box for 2 meter

Table 1: Geotechnical Results Summary

S/N	Geotechnical Properties	1 Meter	2 Meter	Average
1	Moisture Content (%)	17.5	18.8	18.15
2	Internal friction angle friction (φ)	32	33	32.5
3	Undrained cohesion (kPa)	0	0	0
4	Specific Gravity	2.62	2.64	2.63
5	Unit weight (KN/m^2)	16.6	16.6	16.6
6	Coefficient of Uniformity (Cu)	1.6666	1.6470	1.6568
7	Coefficient of Curvature (Cc)	0.864	0.8403	1.7043
8	Coefficient of permeability cm/sec	2.25	2.89	2.57
9	Static Water Level(m)	1.3		

Table 2: Ultimate and Allowable bearing pressure from equation

DEPTH (m)	FOUNDATION WIDTH (m)	ULTIMATE BEARING PRESSURE (kPa)			SAFE BEARING PRESSURE (kPa)		
		L/B = 1.0	L/B = 1.5	L/B = 5	L/B = 1.0	L/B = 1.5	L/B = 5
1	1	653	668	689	218	223	230
	1.5	742	765	796	247	255	265
	2	832	862	903	277	287	301
	2.5	921	958	1010	307	319	337
	5	1367	1441	1545	456	480	515
	10	2259	2407	2615	753	802	871
2	1	1283	1301	1326	428	434	442
	1.5	1389	1416	1453	463	472	484
	2	1495	1531	1580	499	510	527
	2.5	1602	1646	1707	534	549	569
	5	2132	2220	2342	711	740	781
	10	3192	3369	3616	1064	1123	1205

(g) Bearing Capacity Computations

Bearing capacity is an important parameter for assessing the strength of soils to resist shear failure when loads are imposed. To verify and make comparison, average bearing capacity of the in-situ soil was computed using laboratory results obtained from direct shear test of disturbed samples and in-situ dynamic penetration test. This was computed using equation 6 & 7

SPT-N Values Bearing Capacity

These values of SPT N-values when converted to allowable net soil pressures are according to the method of Peck, Hanson and Thornburn (1974) employing the modified relationship as shown in Equation 8.

Table 2: Ultimate and Allowable Bearing Pressures from Equation

Table 3: Ultimate and Allowable Bearing Pressures SPT-N Values

Depth of foundation, Df (m)	Average corrected SPT N-Value	Ultimate Bearing Capacity qu (kN/m2)	Allowable Bearing Capacity qa (kN/m2)
1.5	80	1888.5	629.5

Original Article

Bearing Capacity Based on CPT

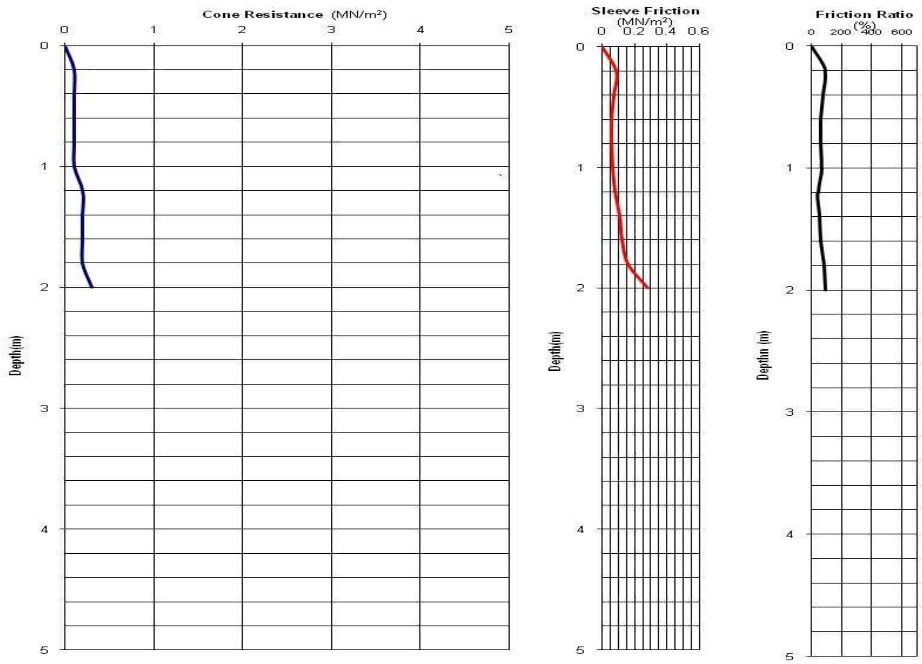


Fig 10: CPT Logs

Undrained strength values were also derived directly from Cone Penetration Tests results based on the well-established equation 9.

Safety factors of about 3.0 are applied to the ultimate bearing pressures to obtain the maximum safe pressures of the soil.

Table 4: Ultimate and Allowable Bearing Capacities from Cone Penetration Test

S/N	Depth (m)	Ultimate Bearing Capacity (Kpa)	Safe Bearing Capacity (Kpa)
1	0.20	27.5	9.2
2	0.40	26.4	8.8
3	0.60	25.4	8.5
4	0.80	25.8	8.6
5	1.00	25.1	8.4
6	1.20	52.9	17.6
7	1.40	52.2	17.4
8	1.60	57.3	19.1
9	1.80	56.5	18.8
10	2.00	87.4	29.1

5.0 CONCLUSION

The geotechnical assessment of the shallow foundation in sedimentary deposits reveals a layered soil profile with medium sand at 1 meter and dense sand at 2 meters. The allowable bearing pressures calculated using a factor of safety of 3.0 indicate suitable values for shallow foundation design. The soil properties and behavior, including moisture content, angle of internal friction, and unit weight, suggest a relatively stable soil condition. Bearing capacity for the homogenous subgrade ranged from the top soil to 1 m depth below the top soil, 218 kN/m^2 to 753 kN/m^2, and to 2m depth below 1m it ranges from 427 kN/m^2 to 1064 kN/m^2. The CPT testing reveals a sandy soil profile with the highest ultimate bearing capacity of 87.4 kPa and a safe bearing capacity of 29.1 kPa at 1 meter. This indicates a strong foundation soil that can support the weight of a building.

6.0 RECOMMEDATION

The study shows that buildings foundation in the study location should be within 1 m or 1.5m at shallow level, since it falls at the allowable bearing pressure in the soil strata. Furthermore, foundation type such as spread footing or mat foundation is advisable for the site.

REFERENCES

Alaminiokuma, G. I., & Omigie, J. I. (2018). Geotechnical properties of soil for civil engineering constructions in Nyuene, Bonny Island, Nigeria. Canadian Journal of Pure and Applied Science, 12(3), 4593-4600.

Allen, J. R. L. (1965). Late Quaternary Niger Delta and adjacent areas: Sedimentary environments and lithofacies. AAPG Bulletin, 49(5), 547-600.

Amadi, A. N., Eze, C. J., Igwe, C. O., Okunlola, I. A., & Okoye, N. O. (2012). Architect's and geologist's view on the causes of building failures in Nigeria. Modern Applied Science, 6(6), 31-38.

ASTM. (1979). Annual book of American Society for Testing and Materials standards (Vol. 1289). ASTM Tech. Publ.

Bowles, J. E. (2012). Engineering properties of soils and their measurements (4th ed.). McGraw Hill Education (India) Private Limited.

British Standards Institution. (1990). Methods of test for soils for civil engineering purposes (B.S. 1377: Part 2).

Das, B. M. (2010). Principles of foundation engineering (7th ed.). Cengage Learning.

Doust, H., & Omatsola, E. (1990). Niger Delta. In J. D. Edwards & P. A. Santogrossi (Eds.), Divergent/passive margin basins (AAPG Memoir 48, pp. 239-248).

EAU (Environmental Advisory Unit). (1992). Hydrological study of the LNG development areas on Bonny Island.

Hospers, J. (1965). Gravity field and structure of the Niger Delta, Nigeria, West Africa. Geological Society of America Bulletin, 76(4), 407-422.

Ilori, O. A., & Anusa, A. A. (2020). Geotechnical characterization of the subgrade on some rural roads in Cross River State, Nigeria and the influence of geology. Journal of Engineering Research and Reports, 10(3), 27-30.

Joint Departments of the Army and Air Force, USA. (1983). Soils and geology procedures for foundation design of buildings and other structures (except hydraulic structures) (Technical Manual TM 5-818-1/AFM 88-3, Chapter 7).

Mallo, S. J., & Umbugadu, A. A. (2012). Geotechnical study of the properties of soils: A case study of Nassarawa-Eggon town and environs, Northern Nigeria. CJ Earth Science, 7(1), 40-47.

NLNG. (2005). Environmental impact assessment for the Nigeria LNG six project Bonny Island (Vols. 1 and 2). Ecosphere Nigeria and Babsal & Company.

Okali, D. U. U., & Igboanugo, A. B. I. (1991). Vegetation of the beach ridges of the Niger Delta. Journal of Tropical Ecology, 7(2), 155-162.

Oyediran, A., & Durojaiye, H. F. (2011). Variability in the geotechnical properties of some residual clay soils from southwestern Nigeria. International Journal of Scientific and Engineering Research, 2(9), 1-6.

Raj, P. P. (2012). Soil mechanics and foundation engineering. Dorling Kindersley (India) Pvt. Ltd. Short, K. C., & Stauble, A. J. (1967). Outline of the geology of the Niger Delta. AAPG Bulletin, 51(5), 761-779.

Terzaghi, K. (1943). Theoretical soil mechanics. Wiley.

Tuncer, E. R., & Lohnes, R. A. (1977). An engineering classification for basalt-derived lateritic soils. Engineering Geology, 4, 319-339.

Weber, K. J., & Daukoru, E. M. (1975). Petroleum geology of the Niger Delta. Proceedings of the Ninth World Petroleum Congress, 2, 209-221