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Foundation Design Consideration In Marginal Lands Of South-Western Niger Delta (Nigeria).

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Abstract
As the world's population continues to increase andrural urban migration on the rise, there is increasedpressure on land in most urban cities. This pressure within the oil-rich Niger Delta region of Nigeria hasled to the use of marginal lands for development.This study involves a pre-construction investigation to characterise the subsurface lithology and recommend an appropriate foundation design for a one-storey building in a marginal land of the south-western Niger Delta. The shell and auger boring method was used to collect disturbed and undisturbed samples from three (3) geotechnicalboreholes at 1.5 m interval each. The Standard Penetration Tests (SPT) were carried out at 16m, 18m and 20m depth in the granular sediments to assess the in-situ densities. The soil investigation results revealed three (3) subsurface lithology within the limit of the boring. It showed a dark-grey peat, Pt at 0 – 4.5 m, soft-grey organic clay, OL at 4.5 – 12 m and a grey-sand, SM at 12 – 20 m which is where the boring stopped. The OL has an undrained shear strength between 21.17 – 22.52 KN/m^2, coefficient of permeability between 1.2 x 10-6 - 1.85 x 10-8 cm/sec, coefficient of compressibility between 24.34 – 25.28 m^2/MN, and the coefficient of consolidation 0.94 – 1.45 m2/yr indicating low permeability and moderately to low compressibility. SM are fine to medium-grained, medium to loose dense silty-sands. Standard penetration tests (SPT) values within this layer range from 4.5 – 17, the bulk unit weight ranges from 11.96 – 14.2 g/cm3. The ultimate bearing capacity (qu) and allowable bearing capacity (qa) vary from 149 – 170.99 KN/m2 and 49.67 – 56.99 KN/m2 respectively at 5 m and 221.62 – 257.61 KN/m2 and 73.87 – 85.87 KN/m2 respectively at 10 m. The settlement rate reveals 0.072- and 0.313-years duration to achieve 50% and 90% post-construction settlement, respectively. In conclusion, a reinforced concrete steel casing pile foundation at a depth of 20 m is recommended.

Keywords: *Southwestern Niger Delta, Marginal Lands, Pile Foundation Design, Building Load, Vertical Stress, Bearing Capacities, Total Settlement, Rate of Settlement*

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I. Introduction

Deltaic areas are characterised as complex terrain due to widespread swampy soils, dense vegetations, and interconnected rivers. These rivers transport large quantities of sediment, which are often deposited rapidly. The Sediments are mainly under consolidated because consolidation lags behind sedimentation. The Niger Delta region of Nigeria serves as the centre of field logistics and operations of many companies engaged in exploration and production in the downstream sector of the oil industry. Consequent to the flat nature and the dense criss-cross network of rivers, extensive portions of the landmass are seasonally flooded leaving more than 80% of this landmass to be classified as marginal land. The term "marginal" is not supported by either a precise definition or research to determine which lands fall into this category. It is most commonly followed by 'degraded' lands, and other widely used terms such as 'abandoned', 'idle', 'pasture', 'surplus agricultural land'. According to Rutledge (1970), a marginal land is one which is unsuitable for development in its original condition. The huge demand for residential space due to population increase in several cities of the Niger Delta region of Nigeria have necessitated the need for reclamation of marginal coastal lands which comprise mainly of swampy soils (Abam, 1993). Hence, the need for extensive infrastructural development to support this urbanization. To prevent adverse environmental impact or structural failure in such marginal lands, detailed sub-soil geotechnical investigation is required to properly design and construct civil engineering structures.

The recent rise in cases of building collapse in Nigeria'scoastal cities has drawn attention to the importance of pre-designgeotechnical investigations forsustainable infrastructural development, especially in marginal lands. Marginal landsare unsuitable for development in its original condition (Routledge, 1970). Amadi (2009) concluded that collapses of building in Nigeria were attributed to one of or a combination of the absence of geotechnical investigation, under design, improper investigation, improper supervision, and poorquality construction. With the increasing building collapse in Nigeria, it is now necessary in the Engineering and construction industry for soil investigation studies to be conducted for proper foundation design. Foundations safely sustain and transmit to the ground, the combined dead, imposed and wind loads does not cause any settlement or other movement which would cause damage or instability to any part of the building (Chudley, 1998). The most common types of foundation usually used in water-logged areas where the soil is fragile and has a very low bearing capacity are the raft and pile foundations. Though commonly used, the design and construction of the raft and pile foundations in water-logged areas is not without its setback which have raised much concern and proven difficult to implement (Jha, 2009). This current study is situated in the south-western Niger Delta marginal lands for a proposed one-storey building duplex on a 281.49m^2 landmass with the coastal area of Bayelsa state. The coastal zone, which comprises the beach ridges and mangrove swamps, is underlain by an alternating sequence of sand and clay with a frequent occurrence of clay within 10 m below the ground surface (Nwankwoala, 2016). Because of the shallow depth of the compressible clays to the surface, the influence of imposed loads results in consolidation settlement. This, in addition to other factors, contributes to the failure of civil engineering structures (Youdeowei, 2013 and Amadi et al. 2012). Such engineering failures necessitate theurgent need to evaluate the subsurface lithologies and examine the geotechnical characteristics and bearing capacities of the sub-soils in the area. Therefore, this study aims to evaluate the suitability of sub-soil conditions and proffer recommendations of the appropriate foundation type and design for a one storey duplex on the marginal land of Nembe in the Niger Delta.

2.1 DESCRIPTION OF STUDY AREA

The study area, Ogbolomabiri, Nembe Bayelsa (Fig. 2.1), with latitudes 4°32'20.184" & 4°32'2.154" N and longitudes 6°23'36.054" & 6°23" 25.068 E lies within the Niger Delta region of Nigeria. Deposits are geologically young, ranging from the Eocene to the recent Pliocene, composed of sediments characteristic of several depositional environments. These include river mouth bar, delta front platform, delta slope and open shelf sediments. The river mouth bar sediments generally consist of coarse-grained sands which extends out in shallow water depths before merging with the sands and clays of the sub-horizontal delta front platform. The area constitutes an extensive plain exposed to periodical inundation by flooding when the rivers and creeks overflow their banks. A prominent feature of the rivers and creeks is the natural levees on both banks which has vast areas of back-swamps and lagoons/lakes behind them with negligible surface flow (Youdeowei and Nwankwoala, 2010). Although various types of morphological units and depositional environments have been recognised in the area (coastal flats, ancient/modern sea, river and lagoonal beaches, sand bars/flats, flood plains, seasonally flooded depressions, swamps, ancient creeks and river channels), the area can be sub-divided into five major geomorphological units, namely: (i) active/abandoned coastal beaches (ii) saltwater, mangrove swamps (iii) freshwater swamps, back swamps, deltaic plain alluvium and meander belt (iv) dry deltaic plain with abundant freshwater swamps (Sombreiro- Warri deltaic plain) and (v) dry flat land and plain.

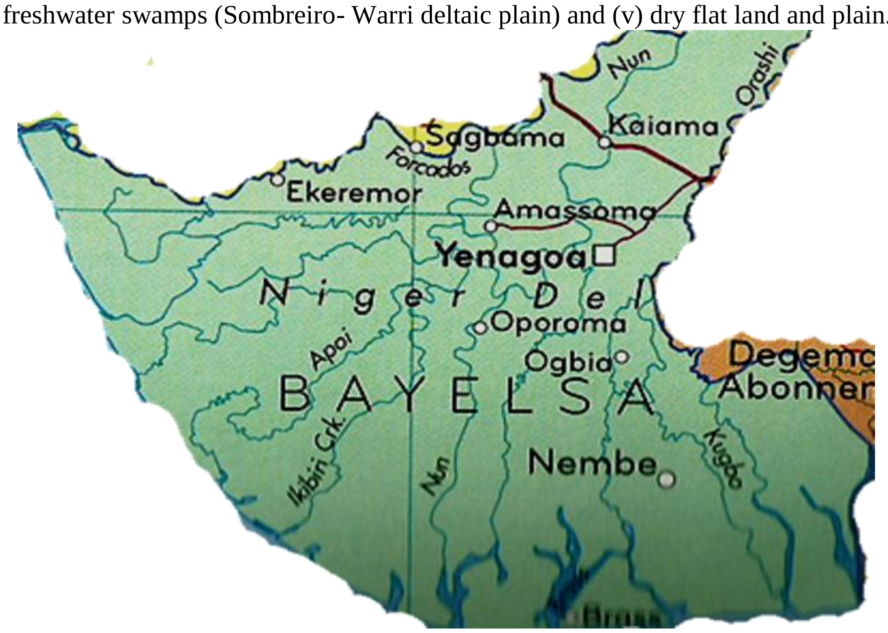

Fig. 2.1. The Satellite map showing the south-western Niger Delta (Bayelsa– Nigeria)

Fig. 2.2Satellite map of the project site

The study area which is situated in a developing area of Ogbolomabiri, Bayelsa, Nigeria (figure 2.2) comprises essentially low-lying areas overlain by Peaty soil (between 0m-4m above sea level). The general topography of the proposed project site is submerged by the saltwater of the mangrove swamp of the Niger Delta (Fig. 2.3). The vegetation in the area consists mainly of mangrove, shrubs and other secondary vegetal growths (Fig. 2.4).

Figure 2.3.Typical Topography at the study area.Figure 2.4.Typical Vegetation Pattern at the study area

III. Methodology

3.1 Field investigation

Field and laboratory investigation like Soil sampling, measurement of water table and standard penetration testing were carried out in three (3) geotechnical boreholes. The boreholes were drilled by the shell and auger cable percussive drilling method, using a hand rig (Fig. 2.3). The hand rig is fitted with a free fall auger which was lifted to about 1.0 m above ground level and allowed free-fall under gravity to advance the boring. As the auger falls, it cuts through the soil such that the cut soil material is retained inside it through a clerk. The auger is then brought to the surface where the soil retained is bailed out. Representative undisturbed and disturbed samples were taken at regular intervals of 1.5 m depth and also when a change in soil type was observed. The samples were systematically described in each stratum in terms of its visual and laboratory analysis. Standard Penetration Tests (SPT) were carried out at regular intervals of depth in the granular sediments to assess the in-situ densities. In this test, the number of blows required to drive the standard sampling spoon 300 m penetration after the initial sitting drive was recorded as the SPT (N) value.

3.2 Laboratory analysis

Detailed laboratory investigations were carried out on representative undisturbed and disturbed samples obtained from the boreholes for the classification and other tests. These tests included natural moisture content, Atterberg limits, unit weights determination, grain-size distribution analysis, unconsolidated-undrained triaxial and shear strength test. All tests were carried out in accordance with BS 1377 (1990).

3.2.1 Data Analysis
3.2.1.1 Bearing capacity

The ultimate bearing capacity, qu, for the foundation was determined using theTerzaghi(1943) bearing capacity formulae as stated below;

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

c = unit cohesion, B = width of footing, Df = depth of foundation, N_γ, N_c, *and* N_q are bearing capacity factors (Table 1), which are functions of angle of friction.

Table 1. Values of Terzaghi Bearing Capacity Factors

$\phi(^0)$	N_c	N_γ	N_q
0	5.14	0	1
5	6.5	0.1	1.6
10	**8.4**	**0.5**	**2.5**
15	11	1.4	4
20	14.8	3.5	6.4
25	20.7	8.1	10.7
30	30	18.1	18.4
35	46	41.1	33.3
40	75.3	100	64.2
45	134	254	135

At a depth of between 1.00 - 20.00 meters at this site, we have the following soil properties: .c = 11.0 kPa, ϕ= 2.0^o, N_γ = 0.1, γ = 12.9 kN/m^2, Nc =6.3, Nq =1.2. Assuming a Factor of Safety(F.S)= 3.0. Also assuming that B/L ~ 0.42

The ultimate (qu) and allowable (qa) bearing capacities ofthe subgrades were determined by applying a factorof safety of 3.0. Due to the reduction in bearing capacity caused by water table within the limit of influence, Terzaghi water table correction factors were applied (eqn. 1) to determine bearing capacity to accountfor its effect.A re-assessment of thebearing capacity at the specified depth of the foundation of1.0-20.0 m below the ground surfaceusingthe following analysis,Terzaghi's, Meyerhof's and Bowles method.

Pile foundation

Pile bearing capacity analysis was achieved using the methods of Peck, Hanson, Thomburn (1974), Terzaghi (1960) and Berazanteu (1961). The general equation for the total load on pile, Q can be expressed as:
Q_{ult} (KN) = Q_b + Q_f……………(eqn 2)
Where: Q_{ult} (KN) = Ultimate Bearing Capacity of Soil, Q_b(KN) = Base Resistance offered by the Soil,Q_f(KN) = Shaft Resistance offered by the Shear Stress between the Soil and Shaft
For the ultimate base resistance in sand
Q_b(KN) = A_{base} + p (Nq – 1)………………(eqn 3)
Q_f(KN) = $A_{shaft.}$ K_s.P_{av} tan□……………….(eqn 4)
For the allowable bearing capacity of the soil
Q_{allow}(KN) = $\frac{Q_b + Q_f}{F.S}$………………….(eqn 5)
F.S = Factor of Safety
Q_{allow}(KN) = Allowable Bearing Capacity of Soil

3.2.1.2 Settlement Analysis

The likely settlements that may arise as a result of loading on the various structures should be computed considering the dimensions of the structure and the subsurface lithology beneath the applied raft foundations. And the final settlement of raft foundation is the total of immediate settlement during the construction phase and the long-term settlement after T90; 90% of consolidation. For this study, Oedometer consolidation tests (ASTM, 1997) were conducted on the undisturbed samples retrieved from the field. These samples were testedin a 75mm diameter x 20mm high ring over a pressure range of 40kPa – 320kPa and the data was analysed using Taylor square root of time fitting method (Taylor, 1948) to derive the consolidation indices. The pre-consolidation pressures were determined from void ratio vs log of pressure curves using Casagrande's method of construction (Cassagrande, 1948). To test the design against excessive settlement of footings, the total settlement and rates of settlement for 50% and 90% were evaluated. The total settlement was determined using eqn. (6) for over-consolidated clays applying the criteria $P0 < Pc < (P0 + \Delta Pav)$ (Das, (1999), while 50% and 90% rates of settlement were calculated using eqns. (8) and (9) respectively. Terzaghi's (1943) classical equation for settlement is given as:

$$Sc = Cc / 1+e_o [H_o. \text{Log}10 \{ p_o + Dp \} / p_o]. \quad \ldots \quad \ldots \quad (6)$$

Where: Sc = final settlement (in cm) of layer of thickness H (m), H = thickness of compressible layer beneath base of foundation = 3.0 m, po = initial overburden pressure = unit weight x depth of thickness = 52m, Cc = Consolidation coefficient = 0.21, p = imposed Structural loads on the soil~ 120 kPa, eo = initial void ratio = 0.840, The time period required for either 50% or 90% of the final foundation settlements can be computed using the relationship:

$$T_{(years)} = T.d^2 / C_v \ldots\ldots\ldots\ldots(7)$$

$$T_{50} = 0.197.d^2 / C_v \ldots\ldots\ldots\ldots(8)$$

$$T_{90} = 0.85.d^2 / C_v \qquad \ldots\ldots\ldots(9)$$

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) = 3.0 m, Cv = Average of coefficient of consolidation. Over the range of pressures involved (obtainable either from tri-axial compression or oedometer tests). = 24.342 m2 / yr, T = time factor which for the given condition of loading and drainage at the project site corresponds to T 50 = 0.197 and T 90= 0.85.

IV. Results And Discussion

4.1 Results

The summary of the results of soil properties for the three boreholes are presented in Table 2 while stratigraphic correlation obtained is presented in Figure 3.1. Three (3) distinct soil layers were encountered at the project site within the limit of the borings (0-20m) and comprised the following: Dark grey peat (Pt), soft grey, Organic clay (OL), greyish silty-Sand (SM).

Geotechnical properties

The first layer encountered during the boring is the dark grey peat which extended from the depth of 0 - 4.5m with a high moisture content. The second layer underlying the study area was the greyish organic clay encountered between 3 – 12 m. The results revealed that the saturated unit weight of this layer ranged between 11.1 – 14.4 kN/m3, the liquid limit (LL) from178.2 – 205.8/, plasticity index (PI) from 76.4 – 105.8, while the plastic limit (Wn) ranges between 157.8 – 195.5% classifying it as high plasticity and high compressibility organic clays (OL) under the unified soil classification scheme. Also, the result of the undrained shear strength of this layer varies from 21.17 – 22.52 KN/m2, the coefficient of permeability (K) range between 1.2 x 10-6 - 1.85 x 10-8 cm/sec, the coefficient of compressibility (Mv) ranges from 24.34 – 25.28 m2/MN, and the coefficient of consolidation (Cv) 0.94 – 1.45 m2/yr indicating low permeability and moderately to low compressibility. Underlying the soft greyish organic clay layer is a sand layer that extends from 10.5 m across the study area to beyond 20m, where boring terminated. These sands are greyish, loose to medium density, fine to medium grain, and non-plastic silty-sands. The standard penetration tests (SPT) values within this third layer ranges from 4.5 – 17 and the bulk unit weight ranges from 11.96 – 14.2 g/cm3. Under partially saturated conditions, the ultimate bearing capacity (qu) and allowable bearing capacity (qa) vary from 149 – 170.99 KN/m2 and 49.67 – 56.99 KN/m2 respectively at 5 m depth and 221.62 – 257.61 KN/m2 and 73.87 – 85.87 KN/m2 respectively at 10 m depth. Furthermore, the qu and qa vary from 294.21 – 344.23 KN/m2 and 98.07 – 114.74 KN/m2 respectively at 15 m and 366.80 – 430.85 KN/m2 and 122.26 – 143.61 KN/m2 respectively at 20m.

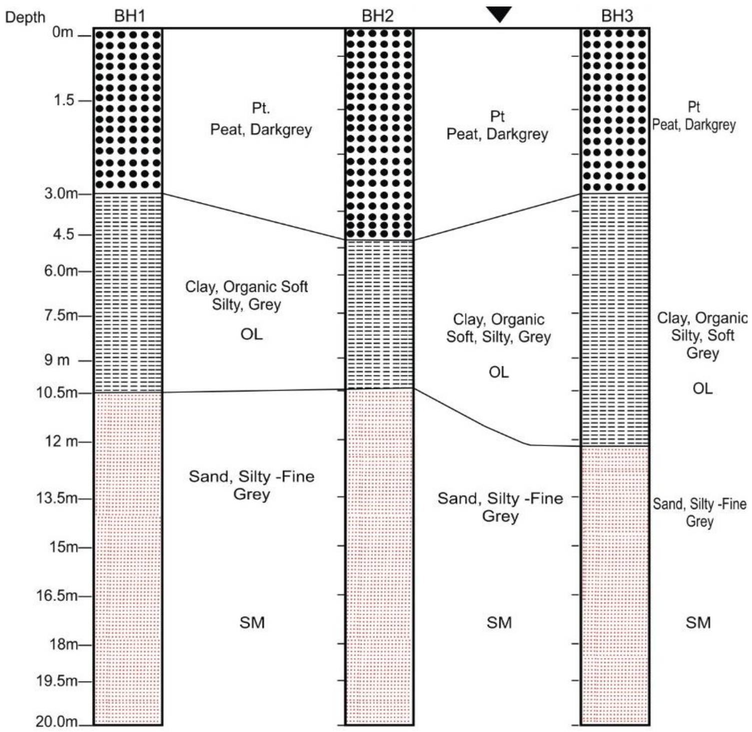

Figure 3.1Stratigraphic Correlation of Subsoil's within the study area

Table 2. Summary of geotechnical properties of the sub-soils at the study area

Soil Type	Sample Depth (m)	Wn (%)	LL (%)	PI (%)	γ_{sat}kN/m3	SPT (N)	> 4.75 (mm)	4.75 (mm)	75μ	2μ
Dark grey Peat (Pt)	1/0	115.8 – 125.8	NP	NP	NP	-	-	-	-	-
	1/4.5	108.5 – 120.5	NP	NP	NP					
Soft greyish organic clay, (OL)	1/9	195.5	178.2	76.4	11.1	-	-	-	-	-
	1/10.5	157.8	205.8	105.8	14.4 – 14.4					
Greyish silty-sand (SM)	1/18	70	-	-	-	7	15	40	35	79
	1/20	81.5	-	-	-	10	20	38	47	85

USC, unified soil classification system; Wn, natural moisture content; LL, liquid limit; PI, plastic index; M_v, coefficient of compressibility; C_v, coefficient of consolidation; SPT, standard penetration test; $\gamma_{sat,}$ saturated unit weight; τ, shear strength; K, coefficient of permeability

Soil bearing capacity.

The bearing capacity values of the subsurface materials at the proposed project site were evaluated using field triaxial test and SPT values. It is observed that the peat which is dark grey and soft (Pt) has a thickness of 3.5m, while the underlain soft grey organic clay (OL) has a thickness of 7m and, sandy, silt-fine, grey which underlies the OL layer about 10m thick.During the boring, the entire area was submerged with water. The values of coefficients of permeability (k) obtained during the consolidation tests on the LightGray Silty Sands (SM) indicated that these materials are of low permeability (Table 3), while that obtained for the organic, soft, silty-clay (OL) indicated that these materials are of moderately low permeability.

Table 3: Consolidation and drainage Characteristics of soils at the study area

SOIL TYPE	Depth (m)	Shear Strength kN/m^2	Coefficient of Compressibility (M_v) m^2/MN	Coefficient of Consolidation (C_v) m^2/yr	Coefficient of Permeability (K) cm/sec
Peat, dark grey, soft. (Pt)	0.0-4.5	-			-
Organic clay, soft, grey (OL)	4.5-9.0	21.2	24.34	0.94	$1.20\text{x}10^{-6}$
		22.5	25.28	1.45	$1.85\text{ x}10^{-8}$

The settlement considerations are under the inherent use of these equations limited to 25.4 mm. The use of the factor of safety of 3.0 takes care of any unexpected high settlement values that may likely be obtained for this site. It could be observed within the study area that the range of soil bearing capacity values useable based on afoundation depth ($D_{f)}$ of20.00 metersfor a B value of 9.6 meters are 122.26 – 143 kPa. The above values of bearing capacity are based on the proven field methods using the Triaxial and SPT results techniques (Peck, Hansen and Thornburn, 1974). Furthermore, based on the empirical methods by Terzaghi and Peck (1967), it could be observed that the range of soil bearing capacity values useable for the structure, based on afoundation depth (D_f) of2.00metersfor a B value= 9.60 m, is between 51.32 to 76.39 $Kn/m^2$with an average of 62.06 KN/m^2 (Table 4)

Table 4. Summary of bearing capacity of deep foundation for triaxial test and computational methods at foundation levels of 20 m

BH	**Depth (m)**	**φ**	**C_U (KN/m^2)**	**N_c**	**N_q**	**N_γ**	**Bulk unit weight (γ)**	**Q_u (KN/m^2)**	**F.S**	**Q_a (KN/m^2)**
BH1	5	2	10.0	6.3	1.22	0.04	12.9	155.29	3	51.32
	10	2	10.0	6.3	1.22	0.04	12.9	233.98	3	77.99
	15	2	10.0	6.3	1.22	0.04	12.9	312.67	3	104.22
	20	2	10.0	6.3	1.22	0.04	12.9	391.36	3	130.47
BH2	5	2	11.0	6.3	1.22	0.04	14.2	170.99	3	56.99
	10	2	11.0	6.3	1.22	0.04	14.2	257.61	3	85.87
	15	2	11.0	6.3	1.22	0.04	14.2	344.23	3	114.74
	20	2	11.0	6.3	1.22	0.04	14.2	430.85	3	143.61
BH3	5	2	10.0	6.3	1.22	0.04	11.9	149.03	3	49.67
	10	2	10.0	6.3	1.22	0.04	11.9	221.62	3	73.87
	15	2	10.0	6.3	1.22	0.04	11.9	294.21	3	98.07
	20	2	10.0	6.3	1.22	0.04	11.9	366.80	3	122.26

Table 5. Summary of results of soil Bearing capacity for the study site

BH	DEPTH (m)	Pile Diameter 10” (0.254mm)	A_{shaft}	A_{base}	Q_b (KN)	Q_f KN)	Q_{wt} (KN)	F.S	Q_{allow} (KN)
BH1	10.5-20	0.254	15.96	0.050	6.87	9.64	16.51	3	5.50
BH1	10.5-20	0.254	15.96	0.050	6.80	9.56	16.36	3	5.45
BH2	12-20	0.254	15.96	0.050	3.26	4.55	7.81	3	2.60

Table 6. Bearing Capacity of Pile in soil based on SPT value after Meyerhof (1956) for bored piles.

BH	Shear stress KN/M^2	T= c+σn tanφK N/M^2	SPT NVALU E	SPT NVALUE CORRECTED	qu (KN)	F.S	qa (KN)	Pile Diameter
BH 1	320	21.2	7.5	11.25	89.3	3	267.92	356
BH 2	330	22.5	4.5	9.7	68.57	3	205.72	356
BH 3	320	21.2	17	16	154.9 6	3	464.9	356

qu, ultimate bearing capacity, qa, allowable bearing capacity, f.s, factor of safety

$Qu = 133NA_b + 0.67\ NA_s$……………………………(eqn 10)

Where: Qu = ultimate total Load in KN
N = Average Corrected SPT Value below Pile Tip = 7.5
N"= Corrected Average SPT Value along the pile shaft = 11.25
A_b = Bulk Area of Pile in m^2 = $0.356m^2$

Implication for foundation design and construction

- Based on the geotechnical investigation, it is recommended that the proposed structure should employ the use of pile foundation as foundation system.
- The pile foundation with raft on it should be borne at depths of 20 m comprising 1 m raft thickness below the ground surface and 1 m above the ground surface as this will necessitate the removal of about 2 m of the overburden during the foundation construction phase.
- The length and diameter of the pile on raft foundation in all cases should be taken as (L= 20meters, where: D = Diameter of the base of the Pile is 10" 0.254m).
- The diameter of the pile should be 10 inches (0.254m) range of values obtained for the allowable bearing capacity for the recommended rafting on pile footings for the structure based on afoundation depth (D_f) of2 mfor a diameter value = 0.254 m. Following the Meyerhof (1974), Bowles (1988), Terzaghi& Peck (1967) and Brinch Hansen (1968) method for determining bearing capacity, the average allowable bearing capacity of soil (Qa), base resistance (Qb) and shaft resistance (Qf) are 4.51 kN, 5.64 kN, and 7.91 kN respectively. This value of pile bearing capacity can be conveniently used for the deep bearing silty sands (SM) at the project site.
- The computed settlement data for the project sites indicates that the total settlement os soil within the project sites is estimated to be about S= 0.177 cm. This is the settlement is expected to take place during and after the construction phase of the building.
- The computed settlement data for the project sites indicates that the long-term Settlement value for the structures at the site is estimated to be about {0.114} 3log {3.07} cm. This is the settlement expected to occur along after the construction phase of the various zones at the project sites.
- About 50% of the settlements must have taken place about 0.072 years after construction, while 90% of the settlement will occur after about 0.313 years after the completion of the project (Table 7).
- The general topography of the proposed project site is relatively flat and submerged in water
- Artificial drainage resulting from runoff and similar operations cannot be ruled out and may result in foundation structures. However, measures should be taken to prevent flooding and erosion by using sand-filling to a height of at least 0.50m and interlocking bricks.

Table 7 Summary of results of Settlement Analysis

BH	Dept (m)	Consolidation coefficient (CC) 0.009(LL-10)	Initial Void Ratio (e^o)	Thickness of Sample (m) H	Initial Overburden Pressure (p^o) kN/m^2	Weight of initial Building to be Built($P\Delta$) Kn/m^2	T_{50}% (m^2/yr)	T_{90}% (m^2/yr)	Total Settlements (cm)
1	1-3	0.21	0.840	3	52	120	0.072	0.313	0.177
	5	0.21	0.84	5	52	120	0.202	0.870	1.706

V. Summary And Conclusion

This study revealed a near-surface stratigraphy of soft dark grey peat (Pt) of a 4.5 meters thickness, underlined by a greyish soft organic clay (OL) which is 7 meters thick and underlain by a light grey silty-sand about 10 meters thick which extends to 20 m. The entire area was submerged by water during the boring exercises; hence, measurements should be taken to prevent flooding and erosion by using sand-filling to a height of at least 0.5 m and the use of interlocking bricks. The values of coefficients of permeability (k) obtained during consolidation tests on the silty-sands (SM) indicate that these materials are of low permeability values, while that obtained for the organic clay (OL) indicate that these materials are of moderately low permeability. The nature and anticipated load of the structure notwithstanding, the superstructure must be supported using pile and raft foundation founded on the silty sand. However, the plastic clay beneath the peat will undergo consolidation along with the compression and creep that will result from loading the loose sand beneath it. Therefore, adequate consideration should be taken of this settlement during the design and construction of the foundation

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