Subsurface Engineering Geological Investigation and Prediction of Axial Pile Capacities for the Design and Construction of Deep Foundations in the Calabar River Channel, Nigeria

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Abstract

Pile foundation was proposed to support a 444.8kN loading jetty in the shoreline of the river channel. To determine the pile bearing capacities, engineering geological investigation involving the boring of 6 geotechnical boreholes to a depth of 30m, sampling, in situ standard penetration test and laboratory analysis were carried out. Foundation subsoil profile depicted low shear strength, intermediate plasticity (CI) inorganic clays to high plasticity organic clays (CH) of soft to medium consistency with high compressibility, and medium to high swell potential. 3.7% of the specimens were of intermediate plasticity while 96.3% were of high plasticity. 3.7% were of medium swell potential, 7.4% of high swell potential while 88.9% were of very high potential class. On the basis of consistency limits, 48.15% of the clays are Ca - montmorilonite, 29.63% Illite while 14.81% were Kaolinitic clays. The sand layers were of medium density, medium to coarse grained size fractions occurs to 30.0m with clay interval ranging from 13.0 to 17.5m in thickness. The top clay layer's pile capacity varies from 29kN at 1.5m increasing linearly to 288kN at 15.0m in sand for the 406mm diameter tubular steel pile. The clay layer within a depth range of 16.5m and 18.0m recorded a decrease in pile capacity. The trend of variation in pile capacity was similar for all pile diameters (508mm-900mm). The largest pile diameter (900mm) depicted a pile capacity of 76kN at 1.6m depth and 3311kN at 30m. Pile underreamed bulbs are recommended for placement in the stable, medium to coarse grained, medium density and stiff to very stiff sand layers and pile installation should commence from the centre towards the edges. Pile interval should be determined to limit drainage effects.

Keywords: engineering geological investigation, pile foundation, expansive clays, under-reaming

Introduction

Foundation engineering design is traditionally carried out against bearing capacity failure and excessive settlement which are directly related to the shear strength of the soil at the foundation. The limiting shear stress at which shear failure may occur and hence the resistance to failure consists of structural resistance due to particle interlocking, frictional resistance to movement between individual soil particles at the material contacts and cohesion or adhesion between particle surfaces (Punmia et al. 1994). Cohesion is the shearing strength at zero normal stress on the failure plane both of which depend on the geological history, state of saturation, permeability characteristics and drainage conditions (Murthy 2012).

Bell (2004) noted that foundation design is concerned primarily with ensuring that movements of footings are within tolerable limits without adverse effect on the functionality and performance of the structure. The tolerable limit is the maximum settlement of 25mm for the foundation footings under allowable load on the basis of shear failure and settlement. Hence, the design of a foundation structure requires an understanding of the local geological and groundwater conditions and,

more particularly, an appreciation of the various types of ground movement that can occur. It is designed to (1)bear the structural loads without exceeding permissible stress, (2) transmit and distribute the loads uniformly to the subsurface soils, (3) be on stable subgrade to control shrinkage (4) provide stability without overturning and (5) be capable of reducing stress against soil movement. These design objectives rely on the soil's cohesive strength, friction angle and the surcharge at the embedment depth. Other important factors for consideration include water table, thickness of clay layers and drainage conditions (Look 2007). Deep foundations are those whose depths are much greater than the widths of the structure (Teme 2017, Das 1999). Pile foundations are considered when design loads are high, subsurface strata are weak and collapsible, load must be transferred to deep, firm stratum, reduction of differential settlement of irregular structures with uneven load distribution mechanism is necessary, foundation substratum portends erosion and scour, to resist horizontal forces due to earthquakes and wind. Piles are advantageous because of their load transfer mechanism along the pile length and as point load at the base. They are both end bearing and frictional resistant because they distribute load at the base and shaft respectively. The choice of pile type is determined by

ground and local conditions; and working load which is a function of pile size and grade (Look 2007).

The bearing capacity of pile foundations can be determined using data from full scale pile load testing which are very costly to perform. Bell, (2004) stated that pile foundations must be capable of sustaining the load with an adequate factor of safety, without allowing settlement detrimental to the structure to occur and the ultimate bearing capacity may be taken as that load which causes the head of the pile to settle 10% of the pile diameter. Dynamic analysis of wave equations using pile driving analyzer; effective or total stress field analysis of static soil properties and direct or indirect in situ testing are other methods of estimating pile capacities. Pile bearing capacities are often estimated from in situ test data as complementary static and dynamic analysis. Omer and Delpak (2007) maintain that data required for predicting the load carrying capacity and settlement of piles are difficult and costly to interpret from routine site investigations and field tests.

Pre-design and construction engineering geological investigations aims to assure adequate consideration of all geological factors affecting the location, design, construction, operation and maintenance of engineering works into the design and construction as results derived therefrom are design tools and should be incorporated (Attewell and Farmer 1976, West 1999). The stability and performance of any infrastructural project depends on the subgrade strength, consolidation settlement, swelling and expansivity which Abija et al (2019a) observed guide in the choice of foundation for the superstructure's load. Foundation failures which has been adduced as one of the greatest causes of building collapse, is caused by inadequate knowledge of the ground conditions (Abam, 2017) necessitating exploratory probing into the subsurface conditions beneath a foundation, testing and appropriate interpretation of geotechnical test results for structural design. Inadequate exploratory investigation and wrong interpretation of results would amounts the unanticipated ground response different from the design predictions (Abija 2018). Abam (2017) noted that the correct interpretation of results is remains an academic exercise in Engineering Geology and Civil engineering classrooms but often a neglected topic in practical applications thus a major problem facing design engineers is on the choice of values of soil properties. This is exacerbated by the uncertainty which characterize geotechnical parameters due inherent soil inhomogeneity. Das (1999) maintained that judicious evaluation of site conditions and soil properties must be incorporated into foundation engineering in consideration of idealized soil mechanics theories which require knowledge of the geological conditions under which the soils and or rocks were deposited. Soil moisture effects on shear strength reduction, amount of clay, compressibility and swelling characteristics, permeability and true cohesion which control subgrade stability, all depend on the provenance. The mode of consolidation guides the choice of parameters for prediction of settlement.

The design and construction of foundations for coastal and offshore engineering structures such as jetties, bridges, offshore platforms, offshore waste repositories and subsea manifolds is fraught with challenges due to inherent weak foundation subgrades starting from the mudline, groundwater effects on bearing capacity, shear and compacted strength of the subsurface layers and clay characteristics at and below the seabed. Coastal infrastructures are subjected to flood wave effects and erosion, unconsolidated, soft sediments being most amenable. The performance of the foundation for any coastal structure depends largely on the depth to which suitably bearing soils can be found (Bell 2004). The investigations which must meet minimum requirements are carried out to a minimum depth of 30m and maximum depth defined by the anticipated design load, bathymetry and the ground condition. Investigations require special equipment and expertise in deriving the engineering characteristics of the foundation subgrades along the underlying subsurface soil profile. Barges and suspended rafts with the capacity to support the drilling rig and personnel are often deployed and the use of static standard piezocone penetration tests to refusal depth supplementary to the standard penetration test carried out in the geotechnical boreholes. The standard penetration test is a static test conducted in situ in the geotechnical borehole during the boring process and has remained the most commonly used method for the determination of pile bearing capacities irrespective of the uncertainties in interpretation of results and repeatability inherent in the energy of the pile hammers. The use of the SPT - N in determination of pile capacity in the industry is very well entrenched and this is achieved by direct application of SPT Numbers or indirect use of friction angle and cohesion estimated from measurements.

The proposed project is a loading jetty with a design load capacity of 444.8kN and a sheet pile wall for protection of the shoreline. This study was carried out to characterize the subsoil profile, recommend most suitable foundation, estimate the bearing capacity, predict structural settlement and recommend foundation design.

Study Location

The river is tidal with a tidal range varying along the shoreline (Figure 1) and relatively low waves. It has an area of 406km² and is located between latitudes 040 56"N and 050 4"N and longitudes 080 15"E and 080 24"E. The climate is tropical equatorial with sunshine being high throughout the year and maximum between January and May while minimum occurs in July and September. Temperatures range on average between 26 and 27 °C during the dry months of to March; and about 24 °C during wet months of June and September. Daily temperatures oscillate between 31.7 °C and 23 °C in dry season highest average values of humidity reach 90 in August as against an average minimum of 74 % in February. Rainfall is most intense (>3500 mm) between April and October, the values being 5 - 7 times higher than in November to March (500 mm). The heavy rainfall tends to accelerate runoff volume and rate thereby resulting in flooding and environmental degradation in the city. The study area is adjacent to the Great Kwa and Calabar Rivers and creeks of the Cross River (Abija et al. 2019b, Abija, 2019).

The geotectonic evolution and regime is the same as that of the Niger Delta and Benue trough, Nigeria (Abija et al; 2019b). The Calabar flank is a hinge line bordering the East-South-East limit of the Niger Delta basin. Tectonically, Cretaceous fracture zones controlled basin evolution during the triple junction rifting and opening of the south Atlantic which palaeo-indicators include trenches and ridges (Abija, 2019). These fracture zone ridges subdivide the margin into individual basins and forms the boundary faults of the Cretaceous Benue -Abakaliki trough that cuts far into the West African shield. The Benue trough, an aulacogen of the triple junction rift system started opening in the Late Jurassic and persisted into the Middle Cretaceous (Lehner and De Ruiter, 1977) diminishing in the Niger delta in the Late Cretaceous. The Niger Delta basin evolved through triple junction rifting, opening of the continent and extension of the fracture zones into the Gulf of Guinea during the Cretaceous. The development of the Niger Delta resulted from the formation of the Benue trough as a failed arm of a rift triple junction associated with the separation of the African and South American continent and subsequent opening of the South Atlantic (Evamy et al., 1978). Most parts of Calabar are overlain by a veneer of consolidated and unconsolidated coastal plain sands of the Benin Formation which overlies the Nkporo shale (Edet and Nyong; 1993).



Fig. 1: Map of the study area showing the project site

These Pleistocene continental sands, sandstones and gravels are friable and of fresh water origin forming excellent aquifer properties with occasional intercalation of shales. The Benin Formation forms the regional aquifer of the Niger Delta basin and it grade into various types of quaternary alluvial deposits comprising mainly of recent deltaic sands on the surface (Etu-Efeotor and Akpokodje; 1990; Abam; 2016; Abija and Abam; 2018). Adjoining Calabar to the north, are the Calabar Flank, a cretaceous sedimentary unit and the Precambrian basement complex of the Oban massif. The coastal pain sands of the Benin Formation are overlain by Quaternary deposits of about 40-50 m thick. Hydrogeologically, the main water-bearing unit in the area is the coastal plain sand aquifer of the Benin Formation. It is composed of unconsolidated and loose sediments; predominantly gravel, sand, silt and clay of Tertiary to recent age. The sands comprising of medium - coarse grained, moderately sorted, subangular to subrounded grains constitute more than 80% of the aquifer materials. The Benin Formation in Calabar area has been divided into two major water bearing units: the upper gravelly and the lower sandy groundwater aquifers. The upper aquifer has mean thickness of 52.7m and average static water level of about 35.0m. The static water level varies from as low as 22.10 to 68.80 m during the wet season. Groundwater table elevation varies from 10m to 50m in the central part. The regional groundwater flow is in the north/south with divide at central parts of Calabar area (Edet and Okereke; 2002). Present day tectonic activities are dominated by the NE - SW Ifewara - Zungeru complex fault system that cuts across the metamorphic basement complex and the younger sedimentary rocks of Nigeria (Abija, 2019b).

Method of Study

Studies involved geotechnical field investigations comprising the boring of 6 deep geotechnical boreholes to a depth of 30m using shell and auger cable percussion rig, 200mm diameter conductor casing and 100mm sampling tubes; soil sampling, in situ static standard penetration tests (SPT) and laboratory analysis. Disturbed samples were collected at 0.75m depth intervals and or where there is a change in lithology. Undisturbed cohesive soils were taken from the boreholes with conventional 100mm diameter and 450mm long open tube sampler. The length of the sample recovered is recorded and compared with the distance the sampler is driven. All the samples recovered from the borings were examined and roughly classified in the field. Standard penetration tests (SPT) were performed every 1.5m advanced through cohesionless soils aimed at assessing the relative

density of the cohesionlesss soils. In this test, a 50mm diameter sampling spoon was driven into the soil in two stages. The initial 150mm penetration of the spoon is the seating drive while the last 300mm penetration forms the test drive. The number of blows required to effect the last 300mm penetration below the seating drive provides an indication of the relative density of the cohessionless soil stratum tested. The penetration resistance, N in blow counts was recorded for use in other geotechnical applications. Groundwater level was measured during boring and 24 hours after completion of the boring.

The geotechnical laboratory programme involved identification and classification tests; unconsolidated undrained triaxial shear strength test on clay samples, 38mm diameter and 76mm high using standard triaxial equipment. In the analysis, the soil specimen is enclosed in a rubber membrane and placed in a triaxial cell. The cell is filled with water after which a cell pressure is applied to stimulate the in situ stress on the specimen. The specimen is then loaded to failure with no drainage from the sample. The confining cell pressures used in the test and analysis were 100kPa, 200kPa and 300 KPa. Laboratory consolidation test were carried out on representative cohesive samples using the Oedometer cell to determine the compressibility properties of the foundation subgrades. The test specimen which is cylindrical in shape is placed carefully into a standard double drained fixed ring oedometer that confines the materials to zero lateral deformation during the test. Porous stones are placed at upper and lower parts of the test specimen in order to permit gravitational drainage and dissipation of water contained in the sample thus permitting volume changes. Vertical loads are applied incrementally measuring corresponding vertical displacement for each increment thus obtaining the change in the volume for each increment with time. Each load increment is obtained for a period of approximately 24 hours or until the change in height with time becomes negligible. All tests were carried out in accordance with the specifications of BS 1377 and ASTM methods for testing of soils for civil engineering purposes.

Method of Data Analysis

Studies involved geotechnical field investigations comprising the boring of 6 deep geotechnical boreholes to a depth of 30m using shell and auger cable percussion rig, 200mm diameter conductor casing and 100mm sampling tubes; soil sampling, in situ static standard penetration tests (SPT) and laboratory analysis. Disturbed samples were collected at 0.75m depth intervals and or where there is a change in lithology. Undisturbed cohesive soils were taken from the boreholes with conventional 100mm diameter and 450mm long open tube sampler. The length of the sample recovered is recorded and compared with the distance the sampler is driven. All the samples recovered from the borings were examined and roughly classified in the field. Standard penetration tests (SPT) were performed every 1.5m advanced through cohesionless soils aimed at assessing the relative density of the cohesionlesss soils. In this test, a 50mm diameter sampling spoon was driven into the soil in two stages. The initial 150mm penetration of the spoon is the seating drive while the last 300mm penetration forms the test drive. The number of blows required to effect the last 300mm penetration below the seating drive provides an indication of the relative density of the cohessionless soil stratum tested. The penetration resistance, N in blow counts was recorded for use in other geotechnical applications. Groundwater level was measured during boring and 24 hours after completion of the boring.

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ASTM methods for testing of soils for civil engineering purposes.

Method of Data Analysis

The shear strength of the subgrades was determined using the Mohr Coulomb criterion (1) while equation (2) bearing capacities of the clayey soils was employed to calculate the bearing capacity of the foundation soils (Das 1999, Murthy 2012). The bearing capacity of the sand layers was estimated based on the standard penetration test results using equation (3) (Terzaghi and Peck 1967). Equation (3) was applied in the determination of the corrected SPT N value. In both clay and sand layers, a safety factor of 3.0 was factored into the results to yield the allowable bearing capacity.

$\tau = C + \sigma_{N} \tan \phi$	(1)
$Q_u = 1.3 CN_c + \sigma_z N_q + 0.38 BN_y$	(2)
Q _{ult} = 0.22Ncor x 0.1073 (Mpa)	(3)
$N_{corrected} = 15 + 0.5 (N - 15)$	(4)

Prediction of Settlement of the Clay and Sand Layers

The amount and rate of settlement of a footing due to a given load per unit base area is a function of the dimensions of the base, and of the compressibility and permeability of the foundation materials between the base and a depth that is at least one and a half times the width of the base (Bell 2007). Clay swelling index is an input for predicting settlement of foundation footings as they are under inundation. Clay swelling and expansivity were investigated considering their characteristic effects on pile uplift loads and equations (8) (Nagaraj and Murthy 1985) and (9) were employed for the determination (Charles, 2001).

$C_{s} =$	0.0463	(LL/100))G.	(5))
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Expansivity =
$$\underline{\mathbf{I}_{p}\mathbf{F}_{0.425}}_{100}$$
.....(6)

Where $C_s =$ Swell index, LL = Liquid limit, $G_s =$ Specific gravity; $I_p =$ plasticity index, $F_{0.425} = \%$ medium sand.

Consolidation settlement has been predicted for the clays soil layers. The elastic (S_e) and long term (S_t) settlements of the clay layer were estimated using equations (7), (8) and (9) respectively for normally

consolidated clays. Settlement of the sand layers in millimeters was determined based on standard penetration test (SPT-N) values averaged over a depth not less than the footing width or diameter after (Meyerhof 1965).

 $\mathbf{S}_{e} = \mathbf{M}_{v} \mathbf{H}_{e} \Delta \mathbf{P} \dots (7)$

 $\mathbf{S}_{t} = \underline{\mathbf{C}_{s}\mathbf{H}_{c}} - \log_{10}(\underline{\mathbf{P}_{0}} + \underline{\mathbf{\Delta}}\mathbf{P}) \text{ (Das 1999).....(8)}$

 $S_{sand} = 1.9q/N$ (Meyerhof 1965).....(9)

Where

 $M_v = Coefficient of volume compressibility of the clay$

- $H_c = thickness of the clay layer$
- $\Delta P = design load$

 $P_0 = Overburden pressure$

 $C_s = Swell index,$

- Cc = compression index = 0.009(LL -10) (Terzaghi and Peck 1967)
- $e_0 = initial void ratio$

 $C_v = Coefficient of consolidation$

q = applied foundation load,

N = average SPT N over a depth not less than the width or diameter of the footing.

Prediction of Axial Pile Capacity

In consideration of the proposed loading jetty with a capacity of 444.8kN, and site conditions, straight shafted, open ended steel, tubular piles which are useful where large lateral loads and extensive lengths apply have been chosen for the design. Pile capacity analyses was carried out for driven straight, shafted, close-ended tubular steel piles, 406mm, 508mm, 600mm, 750mm and 900mm in diameter. In the analysis each borehole was selected and the soil parameters used for the calculation of the ultimate and allowable pile capacities applying a safety factor of 3.0. The design parameters for the unit base resistance and unit shaft resistance proposed by the American Petroleum Institute, (API) were used. The following general equations were adopted.

 $Q_u = Q_b + Q_s$ (Das, 1999, Murthy, 2012).....(10)

 $Q_{\mu} = q_{\mu} A_{\mu} + f_{s} A_{s}$ (Murthy, 2012).....(11)

For cohesionless soils (c=0) $\mathbf{Q}_{b} = \boldsymbol{\sigma}_{vb} \cdot \mathbf{N}_{a} \cdot \mathbf{A}_{b}$(12) While for cohesive soils

For shaft resistance in cohesionless soils equation (14)

$$\sigma'_{vs}K_s$$
.tan δ .A_s.....(14)

while shaft resistance in cohesive oils is given by equation (15)

Equations 12, 13, 14 and 15 were applied in the determination of the base and shaft resistances in clay and sands in each of the boreholes and summing up. Thus the ultimate pile capacity is the summation of the four (4) equations and is given as (16)

$$\mathbf{Q}_{u} = \boldsymbol{\sigma}_{vb} \cdot \mathbf{N}_{q} \cdot \mathbf{A}_{b} + \mathbf{c}_{u} \cdot \mathbf{N}_{c} \cdot \mathbf{A}_{b} + \boldsymbol{\sigma}'_{vs} \mathbf{K}_{s} \cdot \mathbf{tan} \boldsymbol{\delta} \cdot \mathbf{A}_{s} + \boldsymbol{\alpha} \cdot \mathbf{c}_{u} \cdot \mathbf{A}_{s} \dots (16)$$

Where,

 Q_u = ultimate axial pile capacity,

- Q_{b} = total base resistance
- Q_s = ultimate shaft resistance,
- f_s . = unit shaft resistance,
- f_{b} = unit base resistance,
- σ'_{vs} = average effective overburden pressure over soil layer,
- K_s = coefficient of lateral earth pressure against shaft wall,
- δ = effective interaction angle between pile wall and the soil,
- α = pile wall adhesion factor,
- C_{u}^{-} = average undrained shear strength of the clay over the pile penetration depth,
- $A_s = exposed area of pile shaft in the soil layer,$
- σ'_{vb} = effective overburden pressure at the pile base,
- C_u = undrained shear strength of the clay at the pile base,
- N_{c}, N_{q} = bearing capacity factors,

 A_{b} = cross-sectional area of pile base.

Estimating Pile Length (L_p) and Depth of Embedment

Estimating the length of a pile is a difficult task that requires good judgment (Das 1999). In consideration of a working load of 444.8kN, and soil characteristics, the pile length was estimated across the foundation area. Equation (17) was used to obtain the embedment depth and pile length (L_p).

Where,

Qu = Ultimate load on the pile

- N_c = bearing capacity factor
- C_u = undrained shear strength of the clay at the pile base,
- $A_{b} = cross-sectional area of pile base.$
- C_{u}^{-} = average undrained shear strength of the clay over the pile penetration depth,
- d = pile diameter

Results and Discussion

The subsurface soil stratigraphic profile indicates inhomogeneity and lateral faces change with the area investigated. Generally, a very soft peaty and fibrous clay layer underlie to a depth of 1.5m in all the borehole locations. Borehole locations 1 and 2 depicted a 7-layer subsurface profile. Underlying the top peaty and fibrous clay to a depth of 13.0m is a soft silt clay layer which is underlain by medium - fine grained sand to 15.5m. Another silty clay layer with a thickness of 3m underlie, followed by medium - coarse grained sand to a depth of 24.5m. At 24.5 to 26m is a soft sandy clay which is underlain by coarse to gravelly sand to 30.0m. All the sand layers have corrected SPT N values vary from 12-14 classifying as medium density in compactness and stiff sands in terms of consistency. Boreholes locations 3, 4 and 5 are characterized by a 3-layer lithologic profile commencing with a very soft peaty and fibrous clay to 1.5m, a soft silt clay from 1.5 to 21.0m and medium grained, medium density and stiff to very stiff (SPT N-corrected range of 13 - 17) sand to 30m. Borehole 6 location is characterized by a 5 lithologic profile starting from the very soft peaty and fibrous clays to 1.5m which is underlain by a soft silt clay to 8.5m overlying a medium density and stiff, medium grained sand. This layer is underlain by a very soft clay to 21.0m beyond which a medium density and stiff fine to medium grained sand underlie to 30.0m. The index, strength and compressibility characteristics of the foundation subgrade layers are presented below.

Engineering Properties of the Foundation Subgrades

The clays underlying the foundation are characterized as very soft to soft intermediate plasticity (CI) inorganic clays to high plasticity organic clays (CH) under the unified soils classification system (USCS) of soft to medium consistency with high compressibility, and medium to high swell potential. 3.7% of the specimens were of intermediate plasticity while 96.3% were of high plasticity. 3.7% were of medium swell potential, 7.4% of high swell potential while 88.9% were of very high potential class. On the basis of consistency limits using the method of Carter and Bentley, (1991), 48.15% of the clays are Ca - montmorilonite, 29.63% were Illite while 14.81% were Kaolinitic clays (Table 1). The natural moisture content varies from 20.4% to 89% (Tables 1 and 2), the liquid limit from 40% to 165%, plastic limit from 24% to 72% and plasticity index from 16% to 119% (Table 2 and figure 2). The bulk unit weight ranges from 13.8KN/m³ to 20.8KN/m³(Tables 2 and 3), while the percentage saturation which influences the permeability, shear strength and compressibility varies from 96% to 100% with an overall average of 100% indicating the degree to which the pores are filled with water. The clays have specific gravity ranging from 2.51 - 2.77, swell index varies from 0.047 to 0.209 and expansivity from 3.52 to 26.18 (Table 1).

The clays have low shear strength with values ranging from 10.43kN/m² to 70.913kN/m² with undrained cohesive strength of 5kPa to 58kPa and friction angle of 1° to 5° averaging 1.7° (Tables 2 and 3). The ultimate bearing capacity of circular foundation footings in these clays varies from 265.8 kN/m² for a 405mm diameter tubular steel pile to 365.0 KN/m^2 for a 750mm diameter pile. The corresponding allowable bearing capacity ranges from 106.3 KN/m² to 146.2kN/m² (Table 4). The consolidation and compressibility characteristic indicates that the coefficient of consolidation (C_y) determined using a pressure range of 25 - 800kpa varies from ranges from 2.2 m^2/MN to 86.7 m^2/MN , coefficient of volume compressibility from 0.0011m²/kN to 0.00293m²/kN and permeability from 3.76×10^{-8} cm/sec to 87.52×10^{-8} cm/sec (Table 5). These very low permeability characteristics are attributed to the pore lining and filling effects of the montmorilonites and Illites, making the soils practically impermeable and hence most amenable to volume increase and susceptible to pore breaking and collapse causing settlement. Predicted settlement indicates that the immediate or elastic settlement ranges from 16.36mm to 95.41mm while the long term settlement varies from a minimum 13.35mm to 53.13mm across the boreholes (Table 6).

The sands beneath the foundation are medium density and medium to coarse grained sands. The standard penetration test (SPT-N value) indicates a range of 3 to 19 with corresponding corrected values of 9 to 17. The ultimate bearing capacity (Table 6) determined from the standard penetration test (SPT-N value) indicates a range of 212.5kPa to 401.3kPa while the allowable

BU Depth		Liquid	Plastic	Plasticity	Clay Type	USCS	Plasticity	Swell	
БП	(m)			Mathematical Mathe	(alter Carter and Rentley 1991)	USCS	Class	Potential	
	15	142	68	74	Montmorillonite	СН	High	Very High	
	4.5	136	57	79	Montmorillonite CH		High	Very High	
1	7.5	72	46	46	Illite	СН	High	Very High	
	10.5	58	25	33	Kaolinite	CH	High	High	
	16.5	68	25	43	Kaolinite	СН	High	Verv High	
	3	143	66	77	Montmorillonite CH		High	Very High	
2	6	147	70	77	Montmorillonite	СН	High	Very High	
	3	132	58	74	Montmorillonite	CH	High	Very High	
	6	165	72	93	Montmorillonite	СН	High	Very High	
3	10.5	126	45	81	Montmorillonite	СН	High	Very High	
	15.0	118	42	76	Illite	СН	High	Very High	
	19.5	143	69	74	Montmorillonite	СН	High	Very High	
	4.5	80	28	52	Illite	СН	High	Very High	
4	7.5	162	43	119	Montmorillonite	СН	High	Very High	
4	10.5	54	27	27	Kaolinite	CH	High	High	
	13.5	80	32	48	Illite	CH	High	Very High	
	1.5	141	58	83	Montmorillonite	CH	High	Very High	
	6	134	46	88	Montmorillonite	СН	High	Very High	
5	12	77	27	50	Illite	СН	High	Very High	
	15	129	47	82	Montmorillonite	СН	High	Very High	
	16.5	40	24	16	Kaolinite	CL	Low	Medium	
	1.5	147	70	77	Montmorillonite	CH	High	Very High	
	6	163	45	119	Montmorillonite	CH	High	Very High	
6	9	112	46	66	Illite	CH	High	Very High	
0	12	104	29	75	Illite	CH	High	Very High	
	16.5	115	33	82	Illite	CH	High	Very High	
	19.5	72	26	46	Illite	СН	High	Very High	
			Total %			Total % very			
% Montmorillomite		48.15	of High	96.3		high well	88.9		
			plasticity			potential			
0.000			Total %			Total %			
% Illite		29.63	of	2.7		High swell	7.4		
<u> </u>				intermedi	3.7		potential		
	% Kaolii	nite	14.81	ate			Total %	3.7	
				plasticity			meanum		

Table 1: Soil characterization by Atterberg limits

bearing capacity varies from 141.67 kPa to 267.53kPa (Table 6). The settlement of the sands varies from 52.33cm to 211.28cm (Table 7).

Predicted Ultimate and Allowable Axial Pile capacities

The estimated ultimate and allowable axial pile

capacities for closed ended, circular steel piles with diameters of 406mm to 900mm have been presented in Figures 3a - 1. Results shows increase in pile capacities (ultimate and allowable) with depth. The top clay layer's ultimate and allowable pile capacity in borehole locations 1 and 2 varies from 29kN and 7kN respectively at 1.5m increasing linearly to 288kN and

Borehole	Soil Tune	Ensineering Personators	Range of Values			
No.	Son Type	Engineering rarameters	Min.	Max.	Ave	
		Natural Moisture content, (%)	20.4	97.4	56.6	
		Liquid limits, (%)	58	147	109	
		Plastic limit, (%)	25	70	49	
		Plasticity index, (%)	33	79	57	
		Bulk unit weight, kN/m ³	14.9	20.8	18.1	
		Saturated unit weight kN/m ³	15.01	15.53	15.30	
		Initial Void ratio	1.85	1.99	1.95	
	Peaty, fibrous	Degree of saturation, (%)	96	100	99	
	Clay	Specific gravity	2.51	2.7	2.65	
1 & 2		Undrained cohesion (kPa)	5	58	27	
		Undrained angle of internal friction (deg)	1	5	3	
		Swell Index	0.068	0.184	0.013	
		Expansivity	4.45	15	11.85	
	Medium –	SPT Blow Count (Number)	7	16	11	
	Coarse Sand	SPT-N corrected	11.0	15.5	13.0	
		Natural Moisture Content, (%)	63.6	89	78	
		Liquid limits, (%)	40	163	115	
		Plastic limit, (%)	24	72	<u>48</u>	
		Plasticity index, (%)	16	119	69	
		Bulk unit weight, kN/m ³	13.9	16.0	15	
		Saturated unit weight kN/m ³	15,07	15,52	15.28	
	Soft Silty Clay	Initial void ratio	1.87	2.04	1.96	
2 4 5 9 6		Degree of saturation, (%)	100	100	100	
3, 4, 5 & 0		Specific gravity	2.54	2.77	2.68	
		Undrained cohesion (kPa)	5	10	8	
		Undrained angle of internal friction (deg)	1	2	2	
		Swell Index	0.047	0.209	0.143	
		Expansivity	3.52	26.18	21.08	
	Fine -Medium	SPT Blow Count (Number)	3	19	11	
	Sand	SPT-N corrected	9.0	16.5	25.0	



95.67kN respectively at 15.0m in sand for the 406mm diameter tubular steel pile. The clay layer within a depth range of 16.5m and 18.0m recorded a decrease in ultimate and allowable pile capacity which further increases to 707kN and 236kN respectively at 24m depth in the medium – coarse grained sand (Figures 3a – 3d). The trend of variation in pile capacity was similar for all pile diameters (508mm-900mm) for the straight shafted tubular piles. However, expectedly, there is a decrease in pile capacity with pile diameter. The largest pile diameter (900mm) depicted ultimate and allowable pile capacities of 76kn and 34.67kN respectively at 1.5m depth and 3311kN and 1104kN respectively at

Borehole No.	Depth of Sample (m)	Moisture Content (%)	Bulk Unit Weight (KN/m ²)	Undrained Cohesion (kPa)	Angle of Internal Friction (⁰)	Shear Strength (KN/m ²)	Description of Sample	
	12.0	97.4	14.9	7	2	17.89	Coft Cilter	
1	15.7	20.4	20.8	19	5	48.93	Soft Sifty	
	17.25	46.9	16.7	45	1	51.13	Clay	
	12.75	73.6	15.6	5	1	10.43	Soft Silty	
2	16.8	42.4	17.2	58	2	70.913		
	18.0	49.9	16.9	25	1	31.48	Clay	
	5.25	89.0	14.7	5	1	10.43	S-A to all to	
4	10.5	63.6	16.0	8	2	19.0	Soft to silty Clay	
	13.5	84.3	14,7	8	1	13.5		
6	19.5	81.1	13.9	10	1	15.52	Very soft to silty Clay	

Table 3: Results of undrained triaxial Compression Test on the clay samples

Table 4: Bearing capacities of the Representative clay layers

DU M.	Sample	nple Shear strength Parameters		Ultimate Bearing Capacity (KN/m ²)			Allowable Bearing Capacity (KN/m ²)				
BH NO	Deptn	م 0	C	Pil	le diame	ter (mm	ı)	Pi	ile diam	eter (mn	n)
	(111)	Ψ	Cu	405	508	600	750	405	508	600	750
1	12.0	2	7	265.8	265.8	265.8	265.8	106.3	106.3	106.3	106.3
1	15,5	5	19	674.7	674.7	674.7	674.7	269.9	269.9	269.9	269.9
1	17.5	1	45	633.3	633.3	633.3	633.3	253.3	253.3	253.3	253.3
2	12.5	1	5	251.8	251.9	251.9	251.8	100.7	100.7	100.7	100.7
2	16.5	2	58	765.1	765.1	765.1	765.1	306.0	306.0	306.0	306.0
2	18.0	1	25	506.4	506.4	506.4	506.4	202.6	202.6	202.6	202.6
4	10.5	2	8	260.2	260.2	260.2	260.2	104.1	104.1	104.1	104.1
4	13.5	1	8	222.3	222.3	222.3	222.3	88.9	88.9	88.9	88.9
6	13.5	1	10	365.4	365.4	365.4	365.4	146.2	146.2	146.2	146.2

Table 5: Consolidation characteristics of the clay layers

BH	Depth (m)	Pressure Range (kPa)	Coefficient of Consolidation (C _v) (m ² /yr)	Coefficient of Volume Change (M _v) (m ² /kN)	Coefficient of Permeability cm/sec
1	1-13	25 - 800	2.2	00.0292	3.76 x 10 ⁻⁸
2	17 - 18	25 - 800	86.7	0.001.1	87.52 x 10 ⁻⁸
3	19 – 20	25 -800	2.7	0.00293	5.24 x 10 ⁻⁸

30m (Figures 3k - 1). Borehole locations 3, 4 and 5 with similar soil profile depicted ultimate and allowable axial pile capacities of 29kN and 9.67kN respectively at 1.5m depth for the 406mm diameter pile increasing linearly to 936kN and 312kN respectively at 30m depth (Figures 3e - h). Similarly, a linear increase from in ultimate pile capacity from 44kN at 1.5m depth as well as allowable pile capacity from and 14.67kN at same depth of 1.5m to 1288kN and 429.3kN for the ultimate and allowable pile capacities at 30m depth for the 508mm diameter; 55kN and 18.33kN at 1.5m depth to 1629kN and 543kN respectively at 30m for the 600mm diameter, 81kN and 27kN at 1.5m depth to 2275kN and 758.3kN at 30m for the 750mm diameter and 112kN and 37.3kN at 1.5m depth to 3017kN and 1005.7kN respectively at 30m depth for the 900mm diameter straight shafted, tubular steel piles (figure 2e-h). The pile capacities for borehole 6 location presented in Figures 3k and 31 indicates a 29kN ultimate and 9.7kN allowable capacities at a depth of 1.5m for the 406mm diameter pile and varying

Depth (m)	SPT N	N _{Cor}	Density	Lithology	Ultimate Bearing Capacity (kpa)	Allowable Bearing Capacity (kpa)
	947 A.			Borehol	e 1	
15	7	11	Medium	Sand	2597	173.13
21	10	12.5	Medium	Sand	295.1	196.7
24	12	13.5	Medium	Sand	318.7	212.47
30	16	15.5	Medium	Sand	365.9	243.93
	29 82			Borehol	e 2	-
19.5	13	14	Medium	Sand	330.5	220.33
22.5	9	12	Medium	Sand	283.5	189.00
28.5	13	14	Medium	Sand	330.5	220.33
	~ ~			Borehol	e 3	
21	11	13	Medium	Sand	259.7	173.13
24	16	15.5	Medium	Sand	365.9	243.93
27	19	17	Medium	Sand	401.3	267.53
30	13	14	Medium	Sand	330.5	220.33
				Borehol	e 4	·
19.5	3	9	Loose	Sand	212.5	141.67
22.5	5	10	Medium	Sand	236.1	157.4
25.5	13	14	Medium	Sand	330.5	220,33
30	8.5	11.8	Medium	Sand	277.7	185.13
				Borehol	e 5	
22.5	10	12.5	Medium	Sand	295.1	196.73
25.5	11	13	Medium	Sand	306.9	204,60
28.5	18	16.5	Medium	Sand	389.5	259.67
30	10	12.5	Medium	Sand	295.1	196.73
				Borehol	e 6	
22.5	9	12	Medium	Sand	283,3	188.87
25.5	13	14	Medium	Sand	330.5	220.33
28.5	9	12	Medium	Sand	283.3	188.87
30	14	14.5	Medium	Sand	342.3	228.2

 Table 6: Bearing capacities of the sand Layers based SPT-N (ASTM D1586)

linearly to 80kN and 26.7kN at 6m; and 271kN and 90.3kN at 19.5m. The ultimate and allowable pile capacities a depth 28.5m were 965kN and 321.7kn in the sand formation. Pile diameter 508mm, 600mm, 750mm and 900mm diameter steel pile capacities maintain the same trend of variation with depth indicating as in all the boreholes the effect of layer lithology and surcharge load on the pile axial capacities. Pile embedment depth (Table 7) which varies with the pile diameter and range from 11.55m for the 900mm pile to 43.3m for the 403mm diameter pile. These however may vary and pile refusal depths are recommended in such cases.

Foundation Design Considerations

Clay water interaction gives rise to swelling, volume

expansion, heave, water expulsion, shear strength reduction, increase in consistency and settlement which most pronounced in Montmorilonitic clays (Gillot, 1986) and piles in swelling and expansive clays are subjected to uplift loads. The use of under-reamed pile bulbs which together with the piles outer sleeve in the active zone offers resistance to the uplift forces hence are recommended for placement in the stable, medium to coarse grained, medium density and stiff to very stiff sand layers where the effects of moisture content changes are minimal or absent. In consideration of the effects of pore pressure build during pile driving, pile installation should commence from the centre towards the edges and pile interval should be determined to limit drainage effects.





BH	Estimated Pile Length (m)								
BH 1	43.3	30.26	23.03	15.82	11.55				
BH 2	43.3	30.26	23.03	15.82	11.55				
BH 3	43.3	30.26	23.03	15.82	11.55				
BH 4	43.3	30.26	23.03	15.82	11.55				
BH 5	43.3	30.26	23.03	15.82	11.55				
BH 6	43.3	30.26	23.03	15.82	11.55				

Borehole 2

Borehole 3

Borehole 4

Borehole 5

Borehole 6

16.36

21.5

95.41

24.11

21.77

Conclusions

The engineering geological characteristics of the proposed project site indicates a low shear strength, intermediate plasticity (CI) inorganic clays to high plasticity organic clays (CH) of soft to medium consistency with high compressibility under the USCS classification, and medium to high swell potential. The underlying sands of medium density, medium to coarse grained size fractions occurs to 30.0m with clay interval ranging from 13.0 to 17.5m in thickness. Considering that no pile design is complete without a pile load test to assess the load carrying capacity, it is recommended that pile load test be carried out to ensure the design loads meet safety standards.

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