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Geotechnical Investigation for Foundation Design at the West Bank of Light House Creek, Lagos Deep Offshore Logistics Free Zone, Nigeria

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Abstract

This study is aimed at investigation for foundation designs at the West Bank Creek, Lagos deep offshore. All field tests were conducted in accordance with standard geotechnical procedure. The soil profiles obtained within the depth explored at the site consist essentially of two soil zones. They are very soft silty clays and medium dense sands. Results revealed that within the depths bored, a relatively high deposit of clay overlies the boreholes from the river-bed to average depth of 3.0 m. In BHs 8, 10, 11 and 12, the clay extends beyond 3.0m thickness with varying depths ranging from 5.0 m to 8.0 m. However, prevalent deposits of sand underlie the clay to the end of the boreholes. In BHs 6 and 9, the entire holes are characterized by huge deposits of sand formation. Notably, the sandy formation exhibited appreciable Standard Penetration Test (SPT) blows indicating sands of medium densification. Based on the field investigations and comprehensive studies, the estimated volume of available sand fill material is 691,863 m^3. Sand volume estimate was limited to -10.50 m from the river-bed. All the boreholes have potentials for sand borrows. Scooping of the clayey materials is required to expose the sand deposits. Dredging operation with appropriate dredger should be limited to the area covered by the survey.

Keywords: sub-soil, sub-sea sediments, foundation, geotechnical, boreholes, offshore, Lagos deep offshore logistics free zone

1. Introduction

Geotechnical site investigation involves among others, the need to assess the general suitability, safety and economical design of foundations and temporary works, knowing the nature of each stratum and engineering properties of the soil and rock as well as foresee and provide solutions against difficulties that may arise during construction due to ground conditions (Bolton, 1981; Braja, 2005; Clayton et al., 1996). Several studies regarding the geotechnical properties of foundation soils in the creek area of southwestern Nigeria have also been carried out by some researchers. Adepelumi and Olorunfemi (2000) established the geological/geo-electrical sequence of the reclaimed Lekki Peninsula, Lagos, Nigeria. They identified certain sand columns whose thicknesses revealed the geomorphological features of the original pre-fill terrain. Oyedele and Olorode (2010) investigated the prevalent subsurface geological conditions responsible for differential settlement of various degrees peculiar to structures at Medina Estate, Gbagada, Lagos. Their study revealed that the area is underlain to a depth of 14 m by Recent Lithoral Alluvium and Coastal Plain Sands of the Dahomey Basin. It was discovered that foundation soils at shallow depths consist of an extensive layer of materials with extremely low shear strength and corresponding high volume of compressibility.

The complexity of a site investigation depends upon the nature of the ground conditions and the type of engineering structure (Bell, 1990; 2011). Accordingly, a site investigation should attempt to foresee and provide against difficulties that may arise during construction because of ground and/or local conditions. Such investigations should reveal the strata that would be significantly affected by the structural load, the groundwater condition, the degree and extent of weathering, the structural orientation of the rock masses is equally important (Adepelumi et al., 2009; Lekmang et al., 2016). Foundation studies usually provide subsurface information that aid in the design of structures (Warmate & Nwankwoala, 2018). It is thus useful to understand the geotechnical properties of the soil material (index and engineering) so as to make use of the materials in ways that they are best suited since the geotechnical properties of soils, to a large extent determine its suitability for various purposes (Johnson & Degraff, 1988; Ejembi, 2016; Ameh et al., 2018). It is in this regard, that the sub-surface geotechnical investigation of the need to evacuate any existing debris or wrecks from the river bed, deepen the river channel and ascertain the underlying soil along the Light House Creek to ensure stability of the existing quays and foundation designs to support proposed jetty and future structures led to the geotechnical investigations. The information regarding detailed characterization of foundation soils, as well as geotechnical properties of moisture-prone clay in the composition of the soils are scanty. The study area therefore, is a fast growing and future Free Trade Zone which offers great opportunities for infrastructural development. This study intends to characterize the foundation soils, and investigate response of the moisture-prone clay component of the foundation soils to structural loads. The study also aims at carrying out particle size distribution analyses for engineering soil classifications as well as the determination of the bearing capacity and settlement rates necessary for foundation design of structures. The data generated and analyzed will help in ascertaining the suitability of the site for future infrastructural development.

2. Geology of the Area/Accessibility

The area (Figure 1) falls within the geographical setting of Lagos and Dahomean Embayment which consist of the alluvial deposits of South Western Nigerian basin. Lagos is underlain by the Benin Basin. The rocks of the Benin Basin are mainly sands and shales with some limestone which thicken towards the west and the coast as well as down dips to the coast (Oteri & Atolagbe, 2003). Five litho-stratigraphic formations covering the cretaceous to Tertiary ages have been described to be prominent in this geological belt. The formations from the oldest to the youngest include: Abeokuta Group (Cretaceous), Ewekoro Formation (Paleocene), Akinbo Formation (Late Paleocene-Early Eocene), Oshosun Formation (Eocene) and llaro Formation (Eocene) (Adeyemi & Osammor, 2000). The Abeokuta Group presents an unconformity with the basement complex. Locally, the superficial deposits of the area are made up of coastal plain characterized by sand bars, lagoons and creeks (Adegoke et al., 1980). It should be noted however that the recent coastal deposits which occur widely in Lagos State includes the tertiary beds from the Benin Formation which stretches from Calabar in the east through Lagos state to the borders of Benin Republic in the west (Kogbe et al., 1983). Access to the site is generally by water transport. The site could be accessed via the Liverpool Jetty or Coconut Jetty along Apapa-Oshodi Expressway, Lagos.

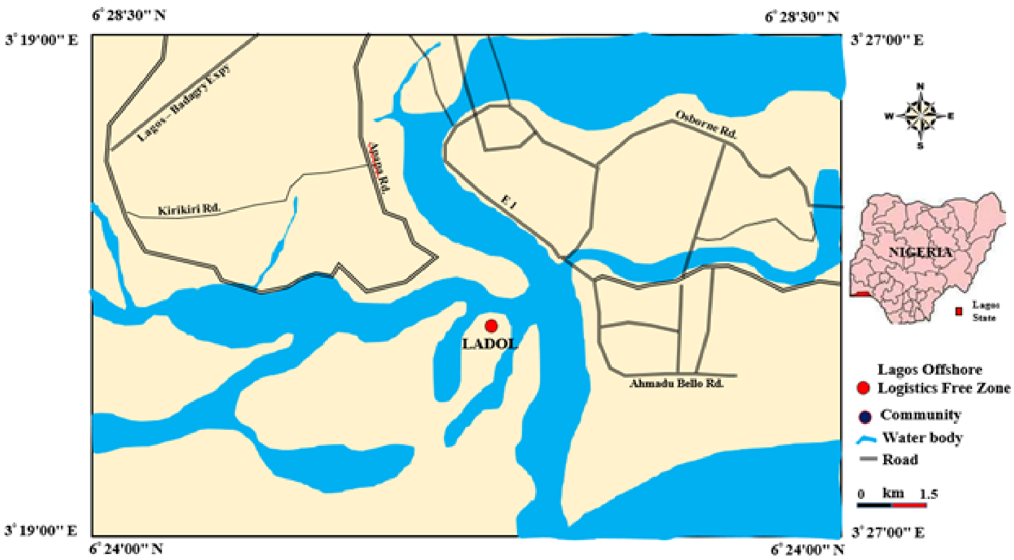

Figure 1. Map showing study location

3. Methods of Study

This involves 12 nos Electric Dutch Cone Penetration Test (CPTu) to refusal and 12 nos random sampling of river-bed sediments. The boreholes were drilled by the shell and auger cable percussive drilling method. The rig positioned on a floating element was fitted with a free fall auger. The auger was lifted to a height of about 1.0m above ground level, and allowed to 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 by means of a clerk. The auger is then brought to the surface where the soil retained in it is emptied out. To prevent collapse of the borehole wall, the holes were lined with casings or shell corresponding to the size of the auger being used for the drilling. As the drilling continues, the auger drops into the open hole until the time sample is to be taken. 4 inch American Petroleum Institute (API) casing strings were for the soil coring.

Representative undisturbed and disturbed samples were taken at regular intervals of 1.00 m depth or where there is change in soil strata. The samples were used for a detailed and systematic description of the soil in each stratum in terms of its visual properties and for laboratory analysis. Undisturbed samples were recovered from the cohesive layers by driving a 100 mm diameter sampler (U2) through a total distance of 450 mm.

Standard Penetration Test (NSPT) was carried out at 3.0 m interval on granular formation. A 50 mm diameter split spoon sampler is driven 450 mm into the soil using a 63.5 kg hammer with a 760 mm drop. The penetration resistance is expressed as the number of blows (N-value) required to obtain 300 mm penetration below an initial 150 mm penetration through any disturbed ground at the bottom of the borehole

3.1 Cone Penetration Tests

Electric Cone Penetrometer of 100KN capacity was used in the cone resistance soundings. The equipment consists of CPT mainframe, Piezo-Cone, Depth encoder, Transducer and enabling software for data capture. Continuous sounding procedure was adopted in the test. The cone was first forced into the ground a distance of 10cm by the application of force to the outer sounding tubes. The cone is then pushed out a distance of about 4cm by the application of force and the magnitude of the force required to achieve this, is transmitted via the transducer to the computer system.

The principal feature of the cone penetrometer is that the tip resistance to penetration by the cone (Sanglerat, 1972), the sleeve friction resistance, pore-water pressure and friction ratio were assessed independently during the study. Importantly, these parameters were color-coded for identification. The tip resistance denoted as Qc was measured by the load cell. The sleeve friction resistance, fs is measured by tension load cell embedded in the sleeve. The friction ratio, Rf is the ratio of the skin friction divided by the tip resistance and the in-situ pore-water pressure measures the pore stress in relation to water in the void spaces within the soil.

3.2 Sub-sea Sediment Sampling

Sediment samplings were collected randomly from the river-bed. The Vaan Veer Grab was fitted to the tripod rig and lowered into the river. Prior to lowering the Grab into river, the sample compartment was opened by suspending the control lever in an upward thrust position. The weight of the grab serves as dead-load. Upon pulling the sling of the tripod rig, the control lever is forcefully released and the sample compartment initially opened drops to the river-bed. As the sling is pulled up by means of winding the handle of the tripod rig, the sample compartment closes and the retained sediment is prevented from dropping off. The soil boring and Cone Penetration test were coordinated prior to the testing. Table 1 below presents the coordinates of the boreholes and the CPTs.

Table 1. Coordinates of boreholes

S/No.	BH/CPT	Easting	Northing
1	BH/CPT-1	543355	710878
2	BH/CPT-2	543467	710869
3	BH/CPT-3	543355	710772
4	BH/CPT-4	543466	710771
5	BH/CPT-5	543408	710695
6	BH/CPT-6	543297	710693
7	BH/CPT-7	543354	710613
8	BH/CPT-8	543461	710615
9	BH/CPT-9	543403	710534
10	BH/CPT-10	543293	710537
11	BH/CPT-11	543356	710454
12	BH/CPT-12	543456	710456

The following laboratory tests were performed on the soil samples:

1) Classification Tests: These will be carried out on both disturbed and undisturbed samples and include but not limited to the determination of natural moisture contents, Atterberg Limit, Sieve Analysis and Unit Weights.

2) Soil Strength Tests: These involve essentially the determination of the relevant strength parameters of cohesion-less samples such as Undrained Unconsolidated Triaxial Tests.

3) Compressibility Tests: This involves the determination of the compressive properties of the soil such as one-dimensional oedometer.

4) Sediment Analysis: This involves determination of chemical properties of the sub-sea sediment in accordance with EGASPIN. The parameters are pH, Conductivity, TPH, TOC, Sulphate, Cation Exchange Capacity, TCC, THC, Nitrate and Phosphate.

3.3 Moisture Content

The water content was determined by drying selected moist/wet soil material for at least 6 hours to a constant mass in a 110 °C drying oven. The difference in mass before and after drying was used as the mass of the water in the test material. The mass of material remaining after drying was used as the mass of the solid particles. The ratio of the mass of water to the measured mass of solid particles was the water content of the material. This ratio can exceed 1 (or 100%).

3.4 Atterberg Limits

Atterberg limits were determined on soil specimens with a particle size of less than 0.425 mm. The Atterberg limits refer to arbitrary defined boundaries between the liquid limit and plastic states (Liquid Limit, WL), and between the plastic and brittle states (Plastic Limit, Wp) of fine-grained soils. They are expressed as water content, in percent.

The liquid limit is the water content at which a part of soil placed in a standard cup and cut by a groove of standard dimensions flow together at the base of the groove, when the cup is subjected to 25 standard shocks (Casagrande, 1932). The one-point liquid test was carried out. Distilled water was added during soil mixing to achieve the required consistency.

The plastic limit is the water content at which a soil can no longer be deformed by rolling into 3 mm diameter threads without crumbling. The range of water contents over which a soil behaves plastically is the Plasticity Index, Ip. This is the difference between the liquid limit and the plasticity limit (WL-WP).

3.5 Particle Size Analyses

Particle size analyses were performed by means of sieving and/or hydrometer readings. Sieving was carried out for particles that would be retained on a 0.075 mm sieve, while additional hydrometer readings were carried out when a significant fraction of the material passes a 0.075 mm sieve. Dry sieving was carried out by passing the soil sample over a set of standard sieve sizes and then shakes the entire units for few minutes with sieve shaker (machine). Particle size is presented on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the same shape regardless of their positions on the particle size distribution plot. The general slope of the distribution curve may be described by the coefficient of uniformity Cu, where Cu = D60/D10, and the coefficient of curvature Cc, where Cc = $(D30)^2/D10 \times D60$. D60, D30 and D10 are effective particle sizes indicating that 60%, 30% and 10% respectively of the particles (by weight) are smaller than the given effective size.

3.6 Unit Weight

The unit weights were determined from measurements of mass and volume of the soil. The unit weight g (KN/m^3) refers to the unit weight of the soil at the sampled water content. The dry unit weight gd, was determined from the mass of oven-dried soil and the initial volume.

3.7 Undrained Shear Strength

This test was performed on relatively undisturbed cohesive soils. Depending on the consistency of the cohesive material, the test specimen was prepared by trimming the sample or by pushing a mould into the sample. A latex membrane with thickness of approximately 0.2 mm was placed around the specimen. A lateral confining pressure of 25 KPa to 100 KPa is maintained during axial compression loading of the specimen. Consolidation and drainage of pore water during testing is not allowed. The test is deformation controlled (strain rate of 60%/h), single stage, and stopped when an axial strain of 15% is achieved. The deviator stress is calculated from the measured load assuming that the specimen deforms as a right cylinder. The presentation of test results includes a plot of deviator stress versus axial strain. The Undrained shear strength, Cu, is taken as half the maximum deviator stress. When a maximum stress has not been reached at strains of less than 15%, the stress at 15% strain is used to calculate undrained shear strength.

3.8 Consolidation

The one-dimensional consolidation referred to Oedometer is performed in a Consolidometer. Clayey soil specimen usually of 63.5 mm in diameter and 25.4 mm thick is placed inside a metal ring with two porous stones, one at the top of the specimen and another at the bottom. The load on the specimen is applied through a lever arm, and compression is measured by a micrometer dial gauge. The metal ring containing the soil specimen is saturated throughout the duration of the test. Each load is usually kept for 24 hrs expect where the compression values becomes insignificant. At the end of the test, the dry weight of the test specimen is determined.

Three basic principles of the test are identified. First, the initial compression which defines the pre-loading stage of the specimen, the second is the primary consolidation where excess pore-water pressure is gradually transferred in effective stress by the expulsion of pore-water. The third principle is the secondary consolidation which occurs after dissipation of the excess pore-water pressure, when some deformation of the specimen takes place because of the plastic readjustment of the soil fabric.

The plot of void ratio (e) against effective pressure (P) for the samples tested, together with calculated values of the coefficients of consolidation (Cv) and of the coefficients of compressibility (Mv). All laboratory tests were carried out in accordance with B.S. 1377 (1995) – Methods of Tests for Soil for Civil Engineering Purposes.

3.9 Engineering Properties of the Soil

The soil profiles obtained within the depth explored at the site consist essentially of two soil zones. They are very soft silty clays and medium dense sands. The geotechnical indexes and engineering parameters of the soil are presented in the Tables 2a and 2b below:

Table 2a. Very soft silty clays

Test Parameter	Average
Moisture Content (%)	59
Liquid Limit (%)	67
Plastic Limit (%)	35
Plastic Limit (%)	32
Plastic Limit (%)	0.76
Plastic Limit (%)	16.08
Plastic Limit (%)	10.41
Plastic Limit (%)	21

Table 2b. Fine sand

Test Parameter	Average
D10 Effective Diameter (mm)	0.09
D30	0.18
D60	0.37
Co-efficient of Uniformity (Cu)	4.05
Co-efficient of Curvature (Cc)	1.00
Co-efficient of Permeability (K) m/sec	6.37×10^{-4}
Angle of Internal friction (f)	35
Bulk Unit Weight (KN/m^3)	12.47
Moisture Content (%)	20.2

4. Discussion of Results

The investigations/study was carried out for the deepening of the Light House Creek, stability for existing quays and for proposed jetty and future structures. The scope of the investigations were 12 nos of soil boring to considerable depths of -15.0 m from the river bed, 12 nos Electric Cone Penetration Test to refusal and 12 nos random sampling of sub-sea sediments. The site stratigraphy showed very soft dark grey silty clayey formation overlying prevalent granular formation of medium densification. Thicknesses of the overlying clayey formation vary from 2.0 m to 7.0 m and the sand extends thereafter to the end of the boring at 15.0 m.

4.1 Dredging Considerations

The relevant information obtained from the borrow pits show that the area is underlain by prevalent medium sand mix underlying clayey over-burden. The sand layers occur above the maximum dredging depth of -15.0 m with significant thickness to satisfy maximum requirements. BHs 1 to 7 are underlain by shallow deposits of soft clays with average depth of 3.0 m. The clays are further underlain by sands to 15.0 m. BH-6 is characterized entirely by sand. BHs 8 to 12 are underlain by high deposits of clays averaged 6.0 m; its further prevails with sands to a greater height. The sands are uniformly-graded fine to medium.

Generally, burrows have potentials but requires scooping the overlying clayey formation to get to the sandy materials. Estimate of the volume of natural geologic material is computed with some degree of simplifications and approximation of actual field conditions. For simplicity, 3 basic parameters were used to determine the volume estimate. They are the length and width of the borehole spacing and depth of exploration (thickness). The conservative values of these dimensions are intentionally chosen to avoid over estimation.

Table 3. Volume estimate of natural geologic materials

BH No.	Length	Width	Thickness	Total Vol. cm^3	Vol. of solid Sand cm^3
BH-1	100m	100m	7.50m	75,000	60,975
BH-2	100m	100m	7.50m	75,000	60,975
BH-3	100m	100m	8.50m	85,000	69,105
BH-4	100m	100m	7.50m	75,000	60,975
BH-5	100m	100m	7.50m	75,000	60,975
BH-6	100m	100m	10.5m	100,500	81,706.5
BH-7	100m	100m	7.50m	75,000	60,975
BH-8	100m	100m	5.50m	55,000	44,715
BH-9	100m	100m	10.5m	100,500	81,706.5
BH-10	100m	100m	4.50m	45,000	36,585
BH-11	100m	100m	3.50m	35,000	28,455
BH-12	100m	100m	5.50m	55,000	44,715
Estimated Sand Volume					691,863

4.2 Piled Foundation

Foundation to support the proposed jetty shall be on piles. Estimate for pile load capacity is based on Terzaghi's (1943; 1967) formula expressed below: Pile design details for reinforced cased piles are based on the cautious estimates as given below in accordance with EN-1997-2. Cu = 21, γclayey = 16.02 KN/m^3, f = 35^o, Nq = 33.30, γsand= 12.47 KN/m^3

Qu = qpu Ab + fs As (i)

Where

Qu = Ultimate bearing capacity

Qpu = Point load capacity

Ab = Sectional area of the pile at its base

Fs = Unit skin friction resistance

As = Surface area of the pile in contact with the soil

Qa = Allowable bearing capacity (using factor of safety of 2.5)

For 450mm Piles

i. Clayey Stratum:

Skin friction resistance for the peat = α.Cu.As

For 450mm piles: 0.75 × 21 × 0.45 × π× 5.0 m = 111N

ii. Lower Sand Stratum

σ at the top of sand stratum 16.08 × 5.0 m= 80.4 N/m^2

σ at the tip of the pile 80.4 + (12.47 - 9.81) × 10.0 m = 107 KN/m^2

Σave= (80.4 + 107)/2 = 93.7 KN/m^2

Angle of internal friction (for sand) fo = 35o, Hence

fs(av) = 93.7 × tan 35o

= 65.6 KN/m^2

Skin friction resistances for the sand layer:

65.6 × π× 0.45dia. × 10.0 m = 927.7 KN

Total skin friction (Qs) for 450 mm diameter pile is:

(111 + 927.7) = 1,038.7 KN

Point Load Capacity

σ at tip of pile = 93.7 KN/m^2

Nq = 33.30 (Terzaghi's shape and depth factor 1943)

Qpu = 93.7 × 33.30 = 3,120 KN/m^2

The limit of maximum value of unit point resistance in sands is 11,000 KN/m^2. Hence, the estimated value will be used.

3,120 × π × 0.452/4 = 496.3 KN

Ultimate Pile load capacity Qu = 1,038.7 + 496.3 = 1,535 KN

Allowable pile load capacity Qa = Qu/2.5 = 614 KN

For 600 mm Piles

iii. Clayey Stratum:

Skin friction resistance for the peat = α.Cu.As

For 600 mm piles: 0.75 × 21 × 0.60 × π× 5.0 m = 148.5 KN

iv. Lower Sand Stratum

σ at the top of sand stratum 16.08 × 5.0 m= 80.4 N/m^2

σ at the tip of the pile 80.4 + (12.47 - 9.81) × 10.0 m = 107 KN/m^2

Σave= (80.4 + 107)/2 = 93.7 KN/m^2

Angle of internal friction (for sand) fo = 35o, Hence

fs(av) = 93.7 × tan 35o

= 65.6 KN/m^2

Skin friction resistances for the sand layer:

65.6 × π× 0.60dia. × 10.0 m = 1,236.7 KN

Total skin friction (Qs) for 600mm diameter pile is

(148.5 + 1,236.7) = 1,385 KN

Point Load Capacity

σ at tip of pile = 93.7 KN/m^2

Nq = 33.30 (Terzaghi's shape and depth factor 1943)

Qpu = 93.7 × 33.30 = 3,120 KN/m^2

The limit of maximum value of unit point resistance in sands is 11,000 KN/m^2. Hence, the estimated value will be used.

3,120 × π× 0.62/4 = 882 KN

Ultimate Pile load capacity Qu = 1,385 + 882 = 2,267 KN

Allowable pile load capacity Qa = Qu/2.5 = 907 KN

For 750 mm Piles

v. Clayey Stratum:

Skin friction resistance for the peat = α.Cu.As

For 600 mm piles: 0.75 × 21 × 0.75 × π× 5.0 m = 186 KN

vi. Lower Sand Stratum

σ at the top of sand stratum 16.08 × 5.0 m = 80.4 N/m^2

σ at the tip of the pile 80.4 + (12.47-9.81) × 10.0 m = 107 KN/m^2

Σave = (80.4 + 107)/2 = 93.7 KN/m^2

Angle of internal friction (for sand) fo = 35o, Hence

fs(av) = 93.7 × tan 35o

= 65.6 KN/m^2

Skin friction resistances for the sand layer:

65.6 × π× 0.75dia. × 10.0 m = 1,546 KN

Total skin friction (Qs) for 750 mm diameter pile is (186 + 1,546) = 1,732 KN

Point Load Capacity

σ at tip of pile = 93.7 KN/m^2

Nq = 33.30 (Terzaghi's shape and depth factor 1943)

Qpu = 93.7 × 33.30 = 3,120 KN/m^2

The limit of maximum value of unit point resistance in sands is 11,000 KN/m^2. Hence, the estimated value will be used.

3,120 × π× 0.752/4 = 1,379 KN

Ultimate Pile load capacity Qu = 1,732 + 1,379 = 3,111 KN

Allowable pile load capacity Qa = Qu/2.5 = 1,244 KN

Table 4. Allowable pile load capacities for the piles

Depth (m)	450mm (KN)		600mm (KN)		750mm (KN)	
	Ultimate	Allowable	Ultimate	Allowable	Ultimate	Allowable
12.0	1,206	482	1,818	727	2,537	1,015
15.0	1,535	614	2,267	907	3,111	1,244

Table 5. Laboratory analytical report for sediment samples

S/No.	**Parameters**	**Method**	**Sub-sea Sediment 1**	**Sub-sea Sediment 2**	**Sub-sea Sediment 3**	**Sub-sea Sediment 4**	**Sub-sea Sediment 5**	**Sub-sea Sediment 6**	**Sub-sea Sediment 7**	**Sub-sea Sediment 8**	**Sub-sea Sediment 9**	**Sub-sea Sediment 10**	**Sub-sea Sediment 11**	**Sub-sea Sediment 12**
1	pH	Meter	6.89	7.13	7.04	6.68	6.84	7.03	7.11	6.87	7.01	6.79	7.12	7.19
2	Conductivity (µS/cm)	ASTM D1125-14	3,669	4,221	5,240	4,880	5,123	4,333	4,627	3,297	3,520	6,330	4,880	3,726
3	Carbonate (mg/kg)	ASTM D4373-96	< 1.0	1.0	1.0	1.0	1.0	1.0	1.0	2.0	2.0	1.0	1.0	1.0
4	TPH (mg/kg)	GC-FID	2.03	3.44	8.22	6.15	7.46	8.52	10.17	14.32	15.81	17.1	12.6	10.38
5	THC (mg/kg)	ASTM 5369	4.33	7.67	12	13.3	8.69	8.08	9.74	18.65	15.6	13.9	22.6	18.4
6	TOC (%)	ASTM D7573-09	0.98	0.74	0.86	0.81	0.93	0.64	0.79	0.96	0.84	0.87	0.87	0.83
7	Cation Exchange Carbon (meqv/mg)	ASTM D7503-10	2.8	2.6	2.8	2.6	2.5	2.6	2.7	2.7	2.9	2.7	2.9	2.9
8	Sulphate (mg/kg)	ASTM C1580-09	75.39	71.44	90.46	88.57	100.02	91.57	93.88	94.3	95.7	82.1	145.2	123.5
9	Nitrate (mg/kg)	ASTM 92104, D4646	5.88	4.72	4.06	5.93	6.09	7.14	7.36	7.22	8.7	8.91	3.24	5.16
10	Phosphate (mg/kg)	ASTM D6146-97	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL
11	TCC (MPN)	ASTM 1604	5.0×10^2	5.29×10^2	4.76×10^3	4.24×10^3	6.83×10^2	2.14×10^3	4.61×10^2	2.11×10^3	4.6×10^2	7.62×10^3	4.38×10^3	5.11×10^3

Table 6. Correlations with N-values for cohesion-less soils

N value	ϕ	Relative Density (%)	Description
<4	25-30	0	Very loose
4-10	27-32	15	Loose
10-30	30-35	65	Medium
30-50	35-40	85	Dense
>50	38-43	100	Very dense

Table 7. Correlations with N-values for cohesive soils

N value	Unconfined compression strength (kPa)	Textural Consistency
<2	<25	Very soft
2-4	25-40	Soft
4-8	40-60	Medium-firm
8-16	60-75	Stiff
16-32	75-100	Very stiff
>32	100-150	Hard

Table 8. Lateral pressure

Borehole No	Depth (m)	Bulk Unit Weight kN/m^3 γ	Angle of Friction Degree	Rankine Co-efficient of Active Earth Pressure Ka	Rankine Co-efficient of Passive earth Pressure Kp	Co-efficient of Earth Pressure at Rest Ko	Poisson Ratio μ
BH-1	3.00	15.90	9	0.729	1.371	0.844	0.458
BH-7	3.00	16.16	6	0.811	1.233	0.895	0.472
BH-8	3.00	16.03	8	0.756	1.323	0.861	0.463
BH-10	3.00	16.23	7	0.783	1.278	0.878	0.468
BH-12	3.00	16.08	8	0.756	1.323	0.861	0.463

Table 9. Undrained triaxial compression tests

Borehole	Depth	Unit Weights		Shear Strength		Poisson's	Specific	Sub-merged	Normal
		Bulk Unit Weight γ	Dry Unit Weight Γ	Undrained Cohesion C	Angle of Friction			Δ	Δ
	(m).	KN/m^3	KN/m^d 3	KN/m^u 2	Deg.	μ		KN/m^{-3}	KN/m^{-2}
BH-1	3.00	15.90	9.88	19	9	0.458	2.67	6.09	18.27
BH-7	3.00	16.16	10.29	21	6	0.472	2.64	6.35	19.05
BH-8	3.00	16.03	10.08	24	8	0.463	2.64	6.22	18.66
BH-10	3.00	16.23	10.34	22	7	0.468	2.63	6.42	19.26
BH-12	3.00	16.08	10.11	20	8	0.463	2.64	6.27	18.81

Table 10. Atterberg limits

Borehole No	Depth (m).	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Liquidity Index	USCS Classification
BH-1	3.00	61	68	36	32	0.78	MH
BH-2	3.00	60	66	36	30	0.80	MH
BH-3	3.00	57	63	33	30	0.80	MH
BH-4	1.00	62	70	38	32	0.75	MH
BH-5	3.00	60	68	34	34	0.76	MH
BH-7	3.00	64	71	38	33	0.79	MH
BH-8	3.00	59	65	34	31	0.81	MH
BH-10	3.00	57	69	36	33	0.64	MH
BH-11	1.00	55	66	30	36	0.69	MH
BH-12	3.00	59	67	35	32	0.75	MH

Table 11. Particle size analysis

BH No	Depth (m)	Moisture Content %	Effective Particle Size D_{10} (mm)	D30 (mm)	D60 (mm)	Coef. of Uniformity $Cu=D_{60}/D_{10}$	Coef. of Curvature $Cc=D_{30}^2/(D_{10}*D_{60})$	Coef. of Permeability $K=C*D_{10}^2$ (m/sec)
BH-1	6	12	0.09	0.15	0.39	4.333	0.641	0.00065
BH-2	9	17	0.09	0.15	0.39	4.333	0.641	0.00065
BH-3	10	22	0.1	0.38	0.54	5.4	2.674	0.0008
BH-4	5	18	0.09	0.25	0.5	5.556	1.389	0.00065
BH-5	6	21	0.096	0.3	0.5	5.208	1.875	0.00074
BH-6	1	25	0.075	0.12	0.425	5.667	0.452	0.00045
	8	16	0.08	0.15	0.38	4.75	0.74	0.00051
BH-7	9	20	0.08	0.09	0.17	2.125	0.596	0.00051
BH-8	15	24	0.08	0.095	0.18	2.25	0.627	0.00051
BH-9	3	24	0.089	0.095	0.18	2.022	0.563	0.00063
	12	25	0.08	0.1	0.22	2.75	0.568	0.00051
BH-10	10	27	0.08	0.099	0.18	2.25	0.681	0.00051
BH-11	12	15	0.09	0.15	0.4	4.444	0.625	0.00065
BH-12	9	17	0.12	0.4	0.68	5.667	1.961	0.00115

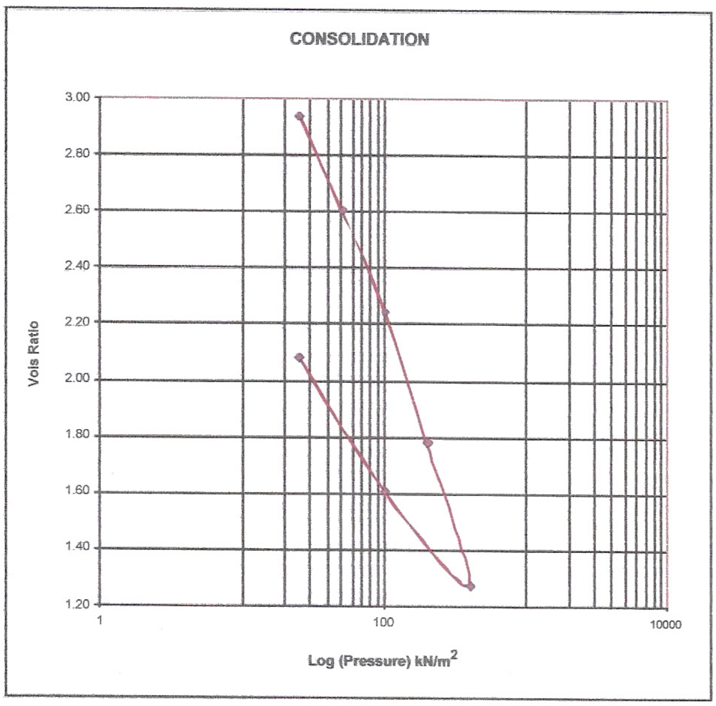

Figure 2. Typical log pressure versus void ratio

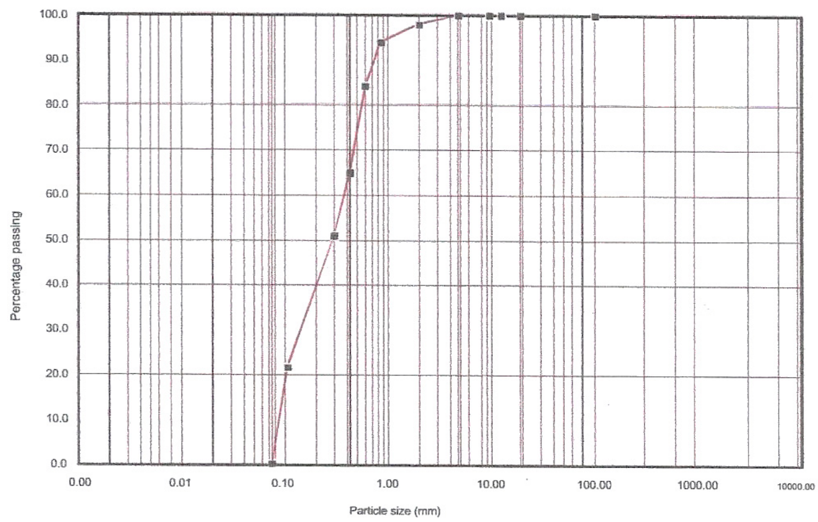

Figure 3. Typical particle size distribution curve

5. Conclusion

The soil profiles obtained within the depth explored at the site consist essentially of two soil zones. They are very soft silty clays and medium dense sands .This study shows that within the depths bored, a relatively high deposit of clay overlies the boreholes from the river-bed to average depth of 3.0 m. In BHs 8, 10, 11 and 12, the clay extends beyond 3.0 m thickness with varying depths ranging from 5.0 m to 8.0 m. However, prevalent deposits of sand underlie the clay to the end of the boreholes. In BHs 6 and 9, the entire holes are characterized by huge deposits of sand formation. All the boreholes have potentials for sand borrows. Notably, the sandy formation exhibited appreciable SPT blows indicating sands of medium densification. Ultimate pile load carrying capacity was estimated for 450 mm, 600 mm and 750 mm axially loaded straight shafted pile terminated at -12.0 m and -15.0 m in the sand stratum. Sand volume estimate was limited to -10.50 m from the river-bed. Scooping of the clayey materials is required to expose the sand deposits. Appropriate dredging operation should be limited to the area covered by the survey.

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