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Geophysical and Geotechnical Investigations of the Sea Floor Sediments for Offshore Subsea Facility Installation in "EMOBS" Oil Fields, Western Niger Delta Nigeria

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Authors' contributions

This work was carried out in collaboration between all authors. Author CCH designed the study. Author IRU performed the statistical analysis. Author ICA wrote the protocol and wrote the first draft of the manuscript. Authors ICA and CCH managed the analyses of the study. Author OMI managed the literature searches. All authors read and approved the final manuscript.

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ABSTRACT

The investigation reveals the stratigraphic sub-division of the site within the depth explored. Basically, though at some depths the lithology is similar, they are different sediments as revealed by the laboratory tests. Essentially, the site consists of silty clay on the surface that is soft in consistency and weak in shear strength. However, the consistency and the strength of the clay improve down depth from 14.50m where it becomes soft-firm. At 30.00 m depth, the clay becomes very firm as some shell fragments (mostly calcareous shells of gastropods and mollusks) occurred within it. Below this clay unit at about 45.00 m depth, a dense to very dense Sand unit of about

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10.00m -12.00m thick occurs. The sand is poorly to moderately graded. Below this sand unit at about 57.0m depth, another firm clay unit occurs. This clay unit becomes intercalated with sand at 60.50m and continues to about 62.50m where it grades into firm clay and continues to the termination depth at 66.00m. Based on the findings, a number of recommendations have been formulated for the safe and cost effective development of this Oil Field.

Keywords: Lithology; shear strength; sea bed; geohazards.

1. INTRODUCTION

This research intends to unravel geological conditions suitable for the installation of some facilities at operational field offshore. The development involves the construction of living quarter and hence requires performing sea bed investigation survey for foundation design parameters at the site.

In order to have an initial conceptualization of the possible Geological properties underlying the site, therefore to carry out a geotechnical investigation of the site.

The scope of this research involves:

Drilling a borehole at the site down to a depth of -60m below mud line, including the use of steel casing to advance boring through any formation down to the stipulated depth

- ❖ To perform Piezocone penetration test (PCPT) to -100m or refusal whichever comes first in accordance with specification.
- ❖ To carry out field/insitu test.
- ❖ To carry out all drilling and sampling in accordance with the technical requirements.
- ❖ To perform standard penetration tests (SPT) in cohesionless soils in accordance with the technical requirements.
- ❖ To conduct laboratory testing of soil samples collected including basic soil classification tests and unconsolidated undrained triaxial test.

1.1 Geological Setting and Stratigraphy

A review of the Geology of the ‘emobs’ oil fields reveals that it falls within the near shore marine/ wooded back swamps environment. These are zones where vast amount of sediment are deposited by rivers in their search for lines of flow. The area lies within the clastic sedimentary wedge of the coastal plain sands of the Benin Formation within the Niger delta. From field observation, the flood plains covering most of the surroundings areas become inundated during the peak of the flood. The area is therefore greatly influenced by the hydrology of the rivers. The stratigraphy of the site generally conforms to the cyclic sedimentation that is common to the Niger delta. It consist of soft clay/silty clay at the surface of the seabed. This unit is underlain by a sand unit. Another clay unit underlies the sand unit. Generally, it reflects the transgression – regression cycles which combined with the framework of the Niger Delta as at the time of deposition to result in the type of sedimentation we have in the present day Niger Delta. The stratum of the soil encountered at the site is summarized in the borehole log. The achieved drilling position was (297568.18E, 170331.57N).

1.2 Ground Water Conditions

The water levels observed in the borehole during the drilling was around 5.0m above the existing seabed level. However, it is possible the water level may rise higher than these level during extended periods of heavy rainfall or decrease during dry season.

2. MATERIALS AND METHODS

Followed the procedures in [1] and [2].

3. RESULTS AND DISCUSSION

3.1 Borehole Drilling/Sampling

Borehole drilling and sampling together constituted major activities during field work. Borehole drilling was carried out using the conventional light cable percussion rig.(later down the hole, rotary drilling rig was used). Shell and Auger were used in retrieving the soil samples. Disturbed samples were collected at regular intervals of 0.75m and also at the changes in soil type. Undisturbed samples were obtained with the conventional open tube sampler, which measures 75mm in diameter and 450mm long. All the samples were examined and classified in the field, properly stored and transported to the laboratory for necessary tests. Standard penetration tests (SPT) were performed in the soils to assess the relative density of soil strata. In the test a 50mm diameter spoon sampler was driven 450mm into the soil with a 63.5Kg hammer falling through a height of 760mm. The initial 150mm penetration is the test drive. The number of blows required to effect the remaining 300mm penetration was recorded as the SPT (N) value. Borehole characteristics including the SPT blow counts are illustrated in Borehole logs below:

BOREHOLE LOG

PROJECT TITLE : SEA BED INVESTIGATION FOR OFFSHORE FACILITY INSTALLATION IN EMOBS FIELDS WESTERN NIGER DELTA NIGERIA

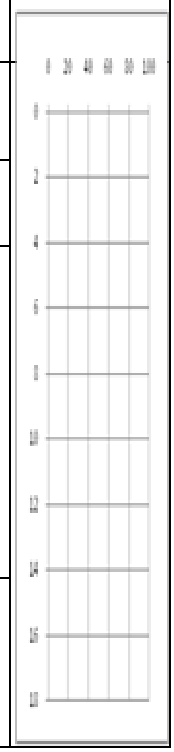
BOREHOLE NO. 1 DEPTH :0.0m to 66m WATER DEPTH : 5.0 m

START OF DRILLING: 02/12/17 END OF DRILLING 10/12/17

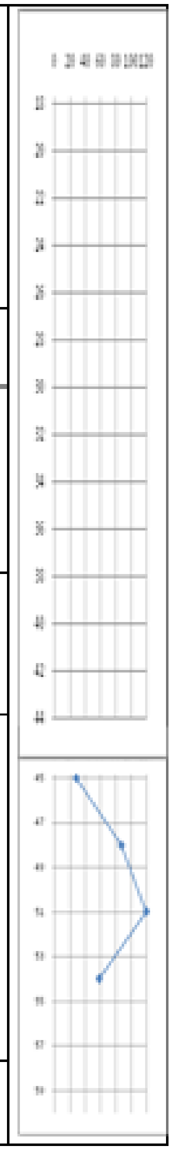
LOCATION : WESTERN DELTA COORDINATE : 297568.18E, 170331.57N

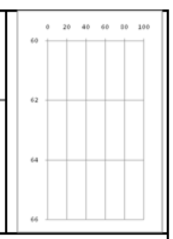
DRILLING EQUIPMENT/METHOD : DODO PERCUSSION/ ROTARY DRILLING RIGS

Depth (m)	Sample type	Lithologic Description	Strata Log	Undrained Cohesion (Kpa)	Friction Angles, ϕ (°)	SPT N (blows/ 0.3m)
0.00 - 0.75	D	CLAY, very soft , dark grayish.				
0.75 - 1.50	D					
0.50 - 2.25	UD					
2.25 - 3.00	D	CLAY, Silty,with shell fragments at 3.75m to 4.5 m depths. Soft,grayish – brownish,		13	2	
3.00 - 3.75	D					
3.75 - 4.50	D					
4.50 - 5.25	D	CLAY, Silty ,Soft ,dark grayish				
5.25 - 6.00	UD					
6.00 - 6.75	D			12	2	
6.75 - 7.50	D					
7.50 - 8.25	D					
8.25 - 9.00	D					
9.00 - 9.75	D					
9.75 -10.50	D					
10.50-11.25	D					
11.25-12.00	UD					
12.00-12.75	D			14	2	
12.75-13.50	D					
13.50-14.25	D	CLAY, Silty, , dark grayish .				
14.25-15.00	D					
15.00-15.75	D					
15.75-16.50	D					
16.50-17.25	UD			9.5	1	
17.25-18.00	D					



18.00-18.75	D	CLAY , Silty Soft ,dark grayish		
18.75-19.50	D			
19.50-20.25	D			
20.25-21.00	D			
21.00-21.75	D			
21.75-22.50	UD			
22.50-23.25	D		9	2
23.25-24.00	D			
24.00-24.75	D			
24.75-25.50	D			
25.50-26.25	D			
26.25-27.00	D			
27.00-27.75	D			
27.75-28.50	UD			
28.50-29.25	D		15	3
29.25-30.00	D	CLAY , Silty with shell fragments, firm, dark grayish		
30.00-30.75	D			
30.75-31.50	D			
31.50-32.25	D			
32.25-33.00	D	CLAY, Silty with shell fragments, firm, dark grayish		
33.00-33.75	D			
33.75-34.50	UD		12	2
34.50-35.25	D			
35.25-36.00	UD			
36.00-36.75	D			
36.75-37.50	D		18.5	3
37.50-38.25	D			
38.25-39.00	D			
39.00-39.75	D	CLAY , becoming Sandy, very firm, grayish to brownish		
39.75-40.50	D			
40.50-41.25	D		20.5	4
41.25-42.00	D			
42.00-42.75	D			
42.75-43.50	D			
43.50-44.25	D			
44.25-45.00	D	SAND , Silty, dense - Very dense, dark brownish		
45.00-45.75	D			
45.75-46.50	D			
46.50-47.25	D			
47.25-48.00	D			
48.00-48.75	D			
48.75-49.50	D			
49.50-50.25	D			
50.25-51.00	D			
51.00-51.75	D			
51.75-52.50	D			
52.50-53.25	D			
53.25-54.00	D			
54.00-54.75	D			
54.75-55.50	D			
55.50-56.25	D			
56.25-57.00	D			
57.00-57.75	D	CLAY, Sandy, very firm, grayish		
57.75-58.50	D			
58.50-59.25	D			
59.25-60.00	D			
60.00-60.75	D	CLAY, Silty with Sand intercalations , very firm, grayish.		
60.75-61.50	D			
61.50-62.25	D			
62.25-63.00	D			
63.00-63.75	D	CLAY ,silty, firm grayish .		
63.75-64.50	D			
64.50-65.25	D			
65.25-66.00	D	END OF BORING		





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CHECKED BY:PROFESSOR ODIGI MINAPUYE ISAAC

3.2 *In situ* Tests

To complement laboratory tests, insitu tests were also carried out on the samples recovered from the borehole. Field tests such as Piezocone penetration tests were also carried . The following are the field insitu tests carried out:Piezocone Penetration Tests, Standard Penetration Tests, Pocket Penetrometer and Torvane.

3.3 Piezocone Penetrometer Test

A piezocone penetration tests was conducted to refusal below mudline.

The plots of the tip, sleeve and cone with depth are presented in Appendix.

3.4 Standard Penetration Tests

The Standard Penetration Tests (SPT) were performed solely on sand strata. Standard penetration tests can be used to assess the relative density of soil strata. In the test a 50 mm diameter spoon sampler was driven 450 mm into the soil with a 63.5 Kg hammer falling through a height of 760 mm. The initial 150 mm penetration is the test drive. The number of blows required to effect the remaining 300 mm penetration was recorded as the SPT (N) value. Results are shown on the borehole log in Appendix A.

3.5 Laboratory Tests

A series of classification, strength and compressibility tests were carried out in the laboratory. The tests were performed in accordance with British and ASTM standards. Details of the type of tests are given below.

3.6 Moisture Content

The water content was determined by drying selected moist/wet soil material for at least 18hours to a constant mass in a 110℃ 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%). Reference test standard: BS 1377: Part 2:1990.

3.7 Unit Weight

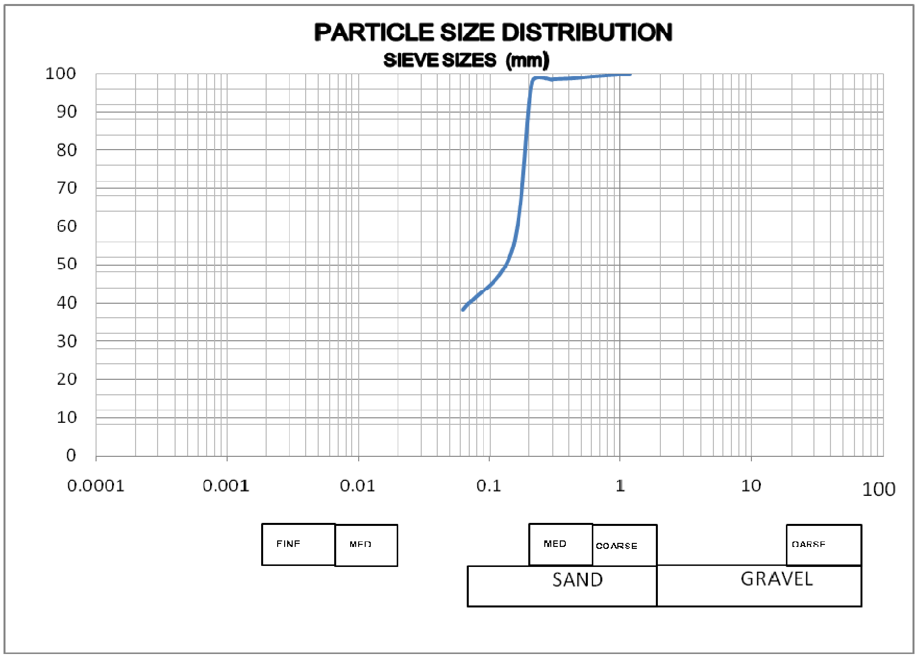
The unit weights were determined from measurements of mass and volume of the soil. For the cohesive soils, a specimen was obtained from a standard steel cylinder with cutting edge, which was pushed manually into the extruded soil sample. Preference is given to a 100 ml cylinder (area ratio of 12%), but a volume of 33.3 ml (area ratio of 21%) may be used when insufficient homogenous sample is available. Specimens of non-cohesive soils were obtained by selecting a part of a cylindrical soil sample, trimming surfaces, and measuring height and diameter. The unit weight γ (kN/m3) refers to the unit weight of the soil at the sampled water content. The dry unit weight γd, was determined from the mass of oven-dried soil and the initial volume [3].

3.8 Particle Size Analysis

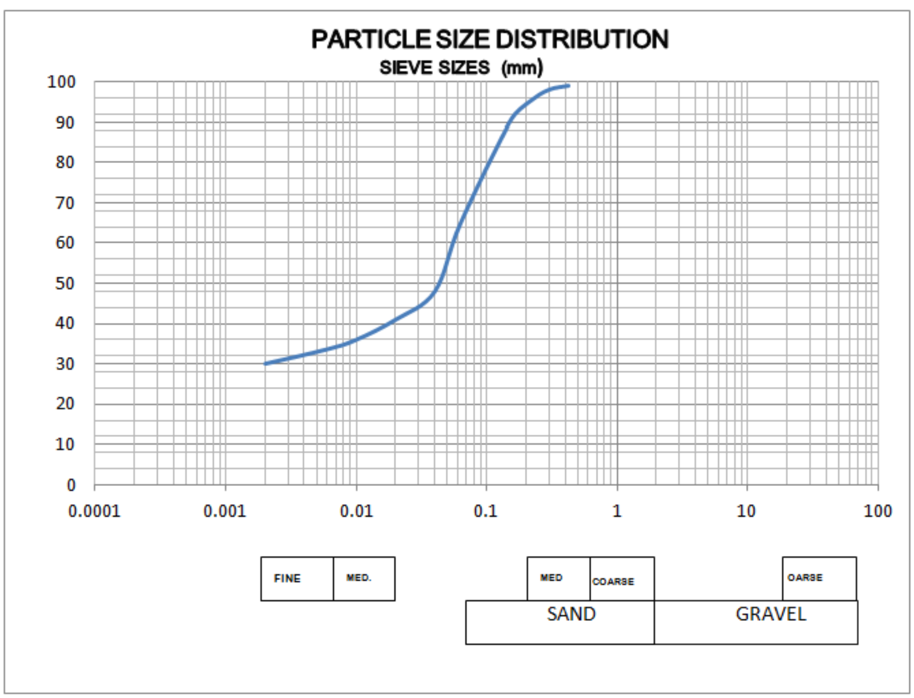
Particle size analyses were performed by means of sieving and/or hydrometer readings. Sieving was carried out for particles that would be retained on a 0.063 mm sieve, while additional hydrometer readings were carried out when a significant fraction of the material passes a 0.063 mm sieve.

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) [4]. The hydrometer method allows measurement of the density of a suspension consisting of fine-grained soil particles and distilled water, to which a dispersion agent is added. This suspension was mixed using a high-speed stirrer. Testing was performed in a thermostatically controlled water bath (25℃ ± 0.5℃). The particle size was calculated according to Stokes' Law for a single sphere, on the basis that particles of a particular diameter were at the surface of the suspension at the beginning of sedimentation and had settled to the level at which the hydrometer is measuring the density of the suspension. A value of 2.65 t/m3 was assumed. The hydrometer results for selected particle sizes are presented as a percentage of the total mass of the soil sample. 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 as seen in the graphs below. 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 x 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. Reference test standard [5].

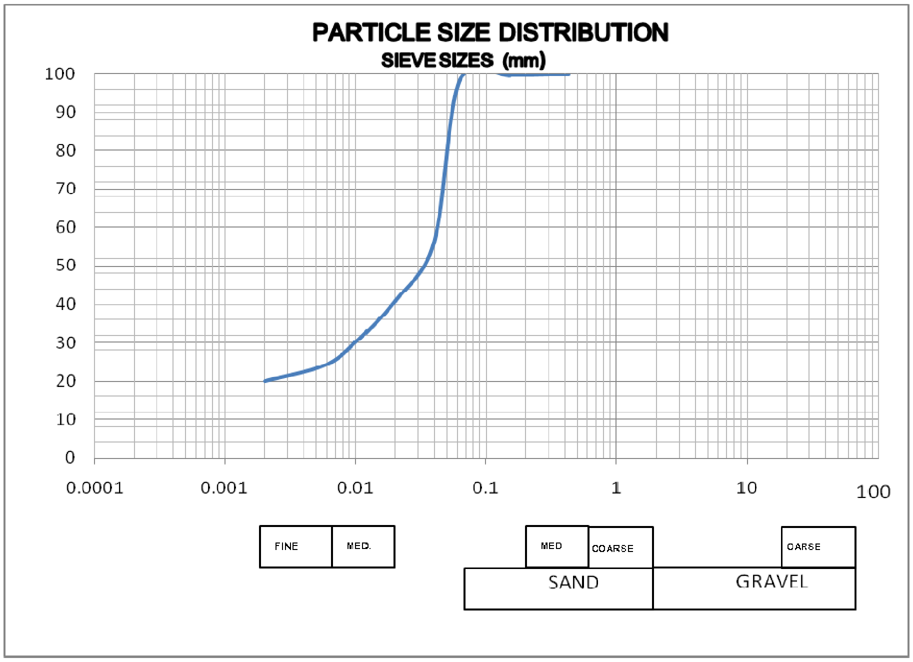
Sample Depth: 4.6m



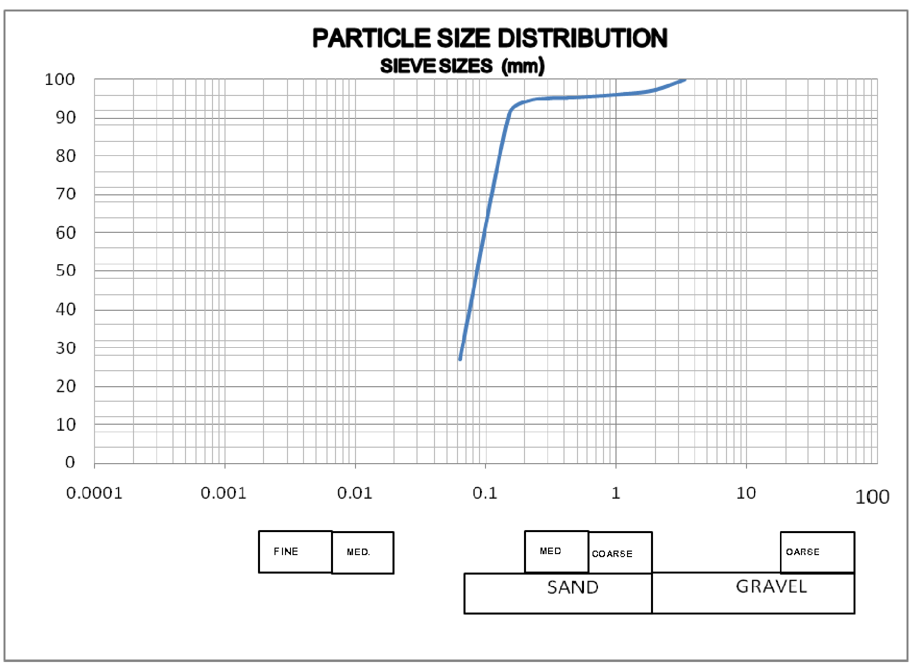
Sample Depth: 20.0m



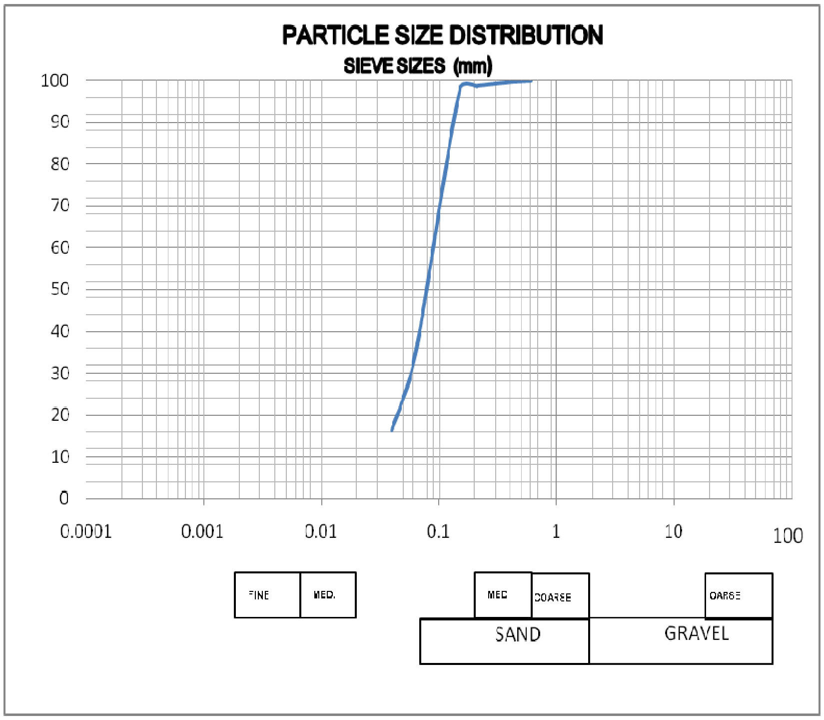
Sample Depth: 9.0m



Sample Depth: 45.0m



Sample Depth: 2.25m



3.9 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. 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 as seen in the graphs below. This is the difference between the liquid limit and the plasticity limit (WL-WP) [5].

ATTERBERG LIMITS DETERMINTION

COORDINTE: 297568.18E.170331.57N
DESCRIPTION OF SOIL: Very Soft Grayish CLAY BOREHOL NO.: 1
DEPTH OF SAMPLE : 0.00-0.75m

Liquid Limit = 61.8%
Plastic Limit = 27.7%
Plasticity Index = 34.1%

ATTERBERG LIMITS DETERMINTION

COORDINTE:297568.18E.170331.57N

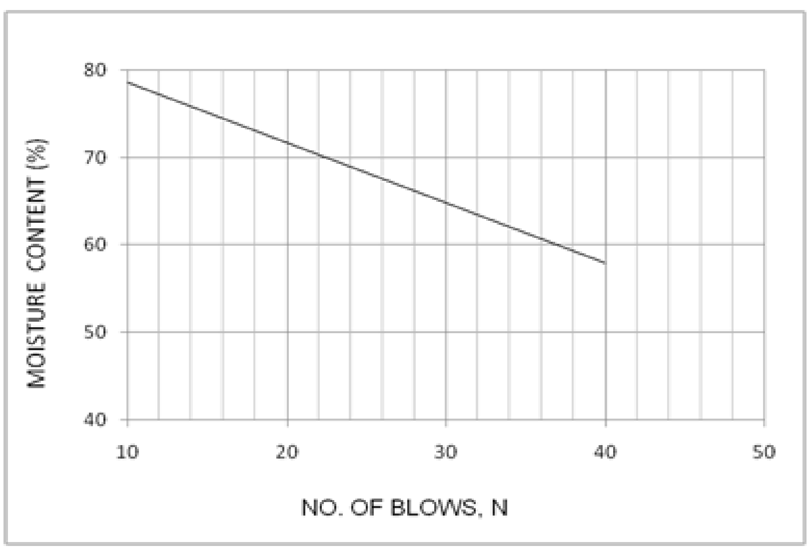
DESCRIPTION OF SOIL : Soft Grayish CLAY BOREHOL NO. : 2

DEPTH OF SAMPLE : 3.0-3.75m

Liquid Limit = 65.9%

Plastic Limit = 27.5%

Plasticity Index = 38.4%



ATTERBERG LIMITS DETERMINTION

COORDINTE:297568.18E.170331.57N

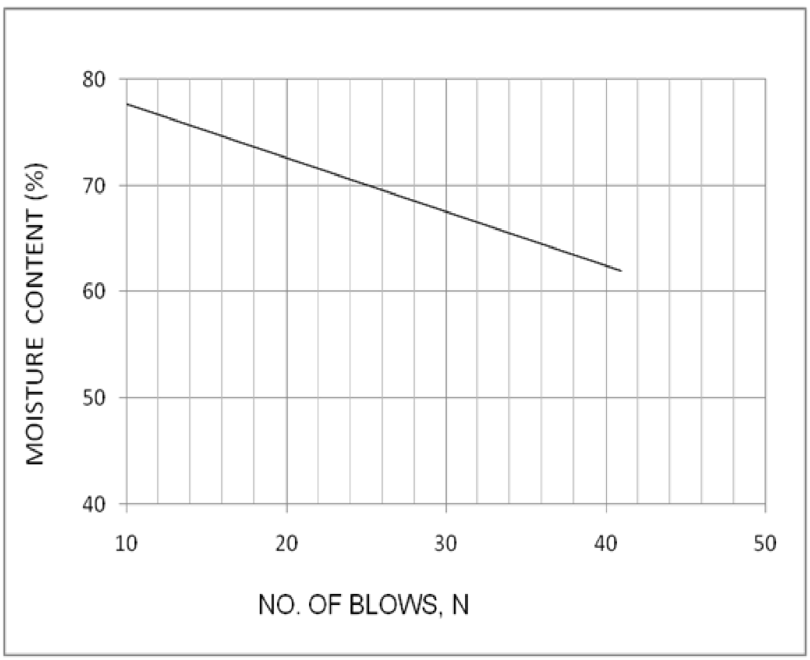
DESCRIPTION OF SOIL : Soft Grayish Silty CLAY BOREHOL NO. : 3

DEPTH OF SAMPLE : 4.50-5.25m

Liquid Limit = 69.7%

Plastic Limit = 27.1%

Plasticity Index = 42.6%



ATTERBERG LIMITS DETERMINTION

COORDINTE:297568.18E.170331.57N

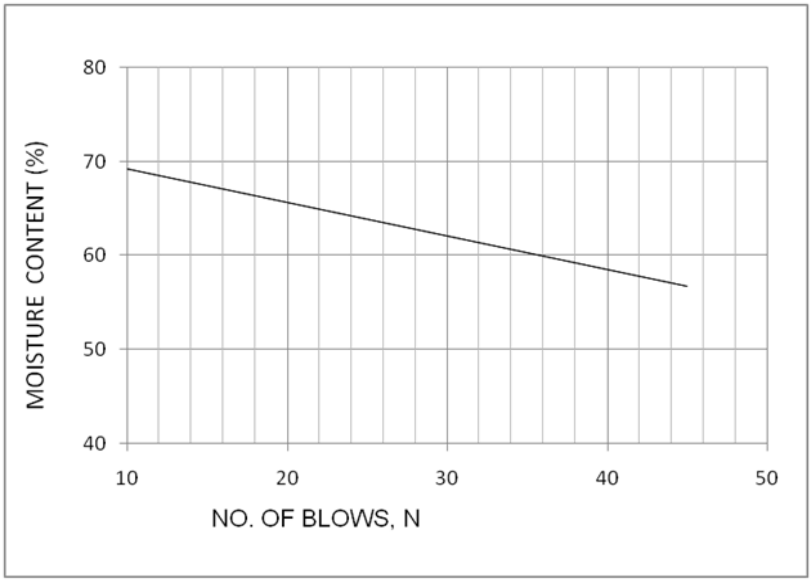
DESCRIPTION OF SOIL : Soft Grayish Silty CLAY BOREHOL NO. : 4

DEPTH OF SAMPLE : 5.25-6.0m

Liquid Limit = 62.3%

Plastic Limit = 28.9%

Plasticity Index = 33.4%



ATTERBERG LIMITS DETERMINTION

COORDINTE:297568.18E.170331.57N

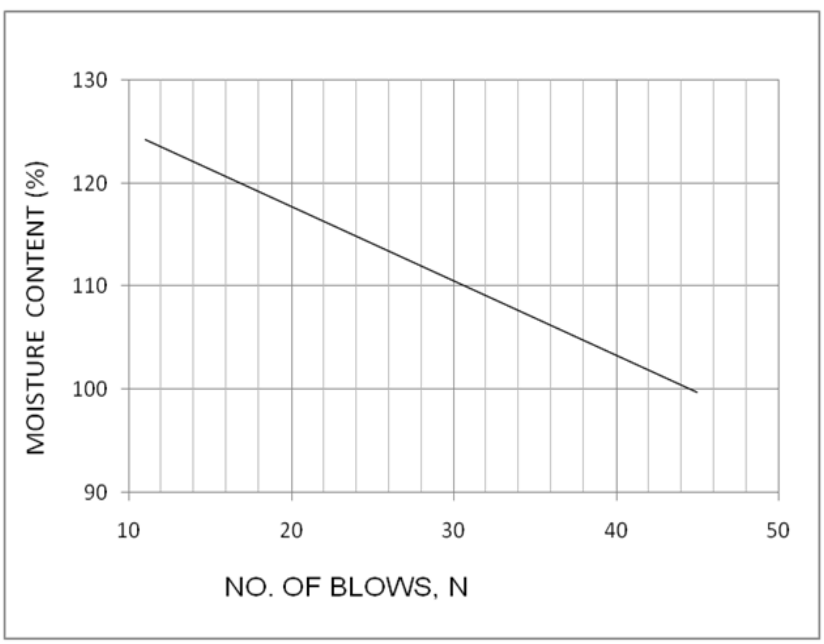
DESCRIPTION OF SOIL : Soft Grayish Silty CLAY BOREHOL NO. : 5

DEPTH OF SAMPLE : 9.00-9.75m

Liquid Limit = 111.3%

Plastic Limit = 30.8%

Plasticity Index = 80.5%



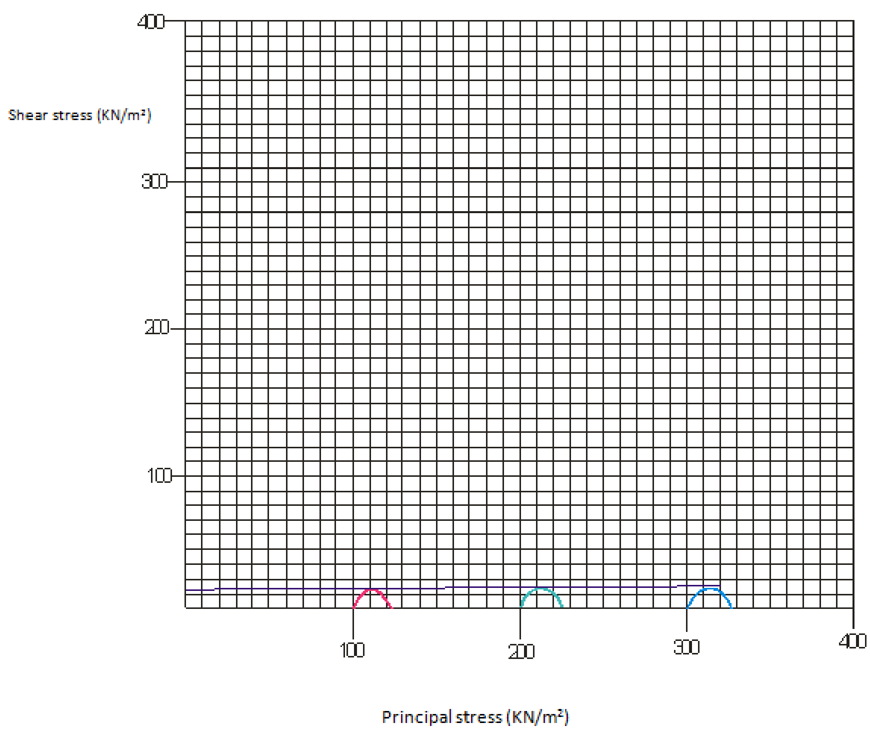
3.10 Unconsolidated Undrained Triaxial (UU)

This test was performed on undisturbed samples of 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 100 kPa to 300 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 [5]. 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. As seen in the graphs below;

UNCONSOLIDATED UNDRINED TRIXIAL TEST

Location: Western Niger Delta Borehole No. : 1 Depth : 2.25m

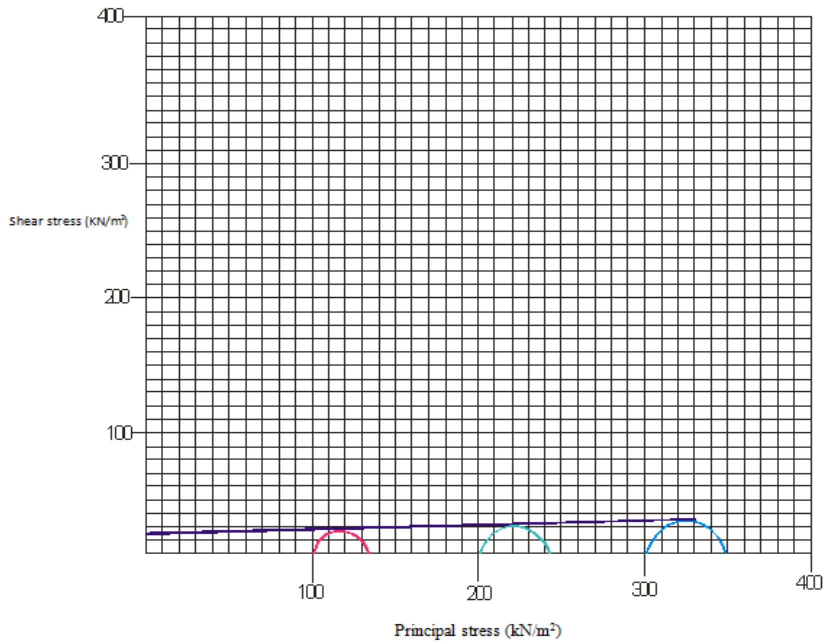
RESULT		
Undrained Cohesion (KN/m^2)	Angle Of Int. Friction(Φ)	Moisture Content (%)
13	2	53.8



UNCONSOLIDATED UNDRINED TRIXIAL TEST

Location : Western Niger Delta Borehole No. : 2 Depth: 28.0m

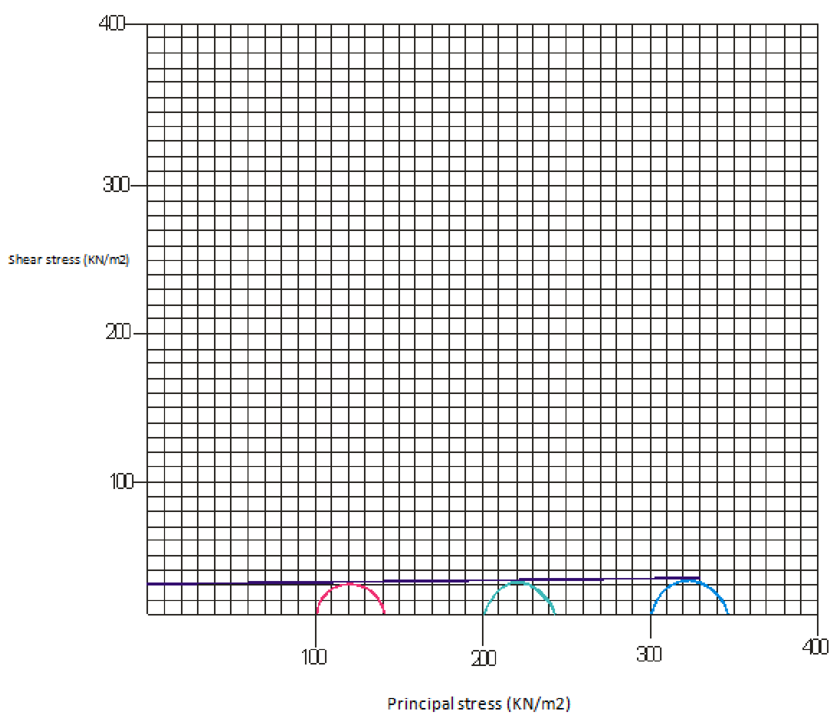
RESULT		
Undrained Cohesion (KN/m^2)	Angle Of Int. Friction(Φ)	Moisture Content (%)
15	3	50.8



UNCONSOLIDATED UNDRINED TRIXIAL TEST

Location : Western Niger Delta Borehole No. : 3 Depth : 34.0m

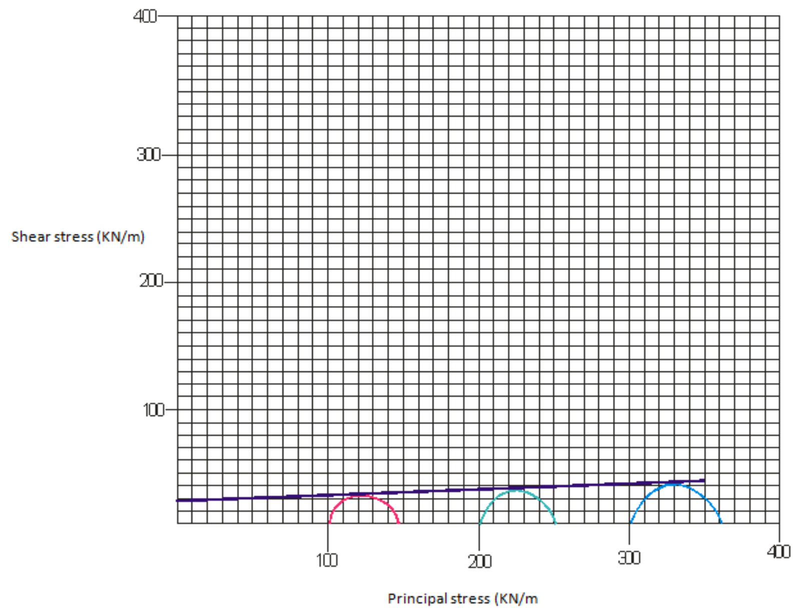
RESULT		
Undrained Cohesion (KN/m^2)	Angle Of Int. Friction(Φ)	Moisture Content (%)
10	2	41.4



UNCONSOLIDATED UNDRINED TRIXIAL TEST

Location : Western Niger Delta Borehole No. : 4 Depth : 36.0m

RESULT		
Undrained Cohesion (KN/m^2)	Angle Of Int. Friction(Φ)	Moisture Content (%)
19.5	3	46.0m



UNCONSOLIDATED UNDRINED TRIXIAL TEST

Location : Western Niger Delta Borehole No. : 5 Depth : 40.0m

RESULT		
Undrained Cohession (KN/m^2)	Angle Of Int. Friction(Φ)	Moisture Content (%)
20.5	4	46.8

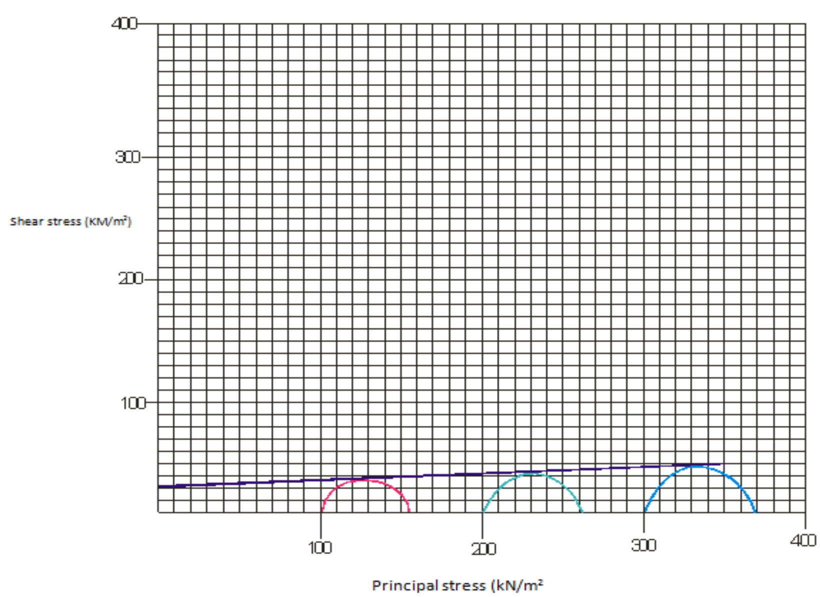


Table 1. Summary of index and engineering parameters

Sandy clay			
Parameters	**Min.**	**Max.**	**Ave.**
Natural moisture content (%)	20.3	94.0	57.15
Liquid limit (%)	61.5	117.0	89.25
Plastic limit (%)	25.0	31.3	28.15
Plasticity index (%)	35.5	85.7	60.6
Bulk unit weight (kN/m^3)	14.8	17.5	16.15
Dry unit weight (kN/m^3)	9.0	12.4	10.7
Specific Gravity	2.50	2.64	2.57
Undrained cohesion ((KN/m^2)	9	20.5	14.75
Undrained angle of internal friction (deg)	1	4	2.5

3.11 Consolidation Test

Selected cohesive samples were taken for laboratory consolidation. The test specimen is cylindrical in shape and is introduced carefully into a standard double drained fixed-ring oedometer that confines the material to zero lateral deformation during the test.

The tests were performed in accordance with relevant standards. The soil sample was subjected to an incrementally increasing vertical load [6]. Each load increment is maintained for a period of approximately 24 hours or until the change in height with time is negligible. Results of the tests have been provided in Table.

3.12 Chemical Tests

Four different chemical tests were conducted on the soil samples taken from the site, namely: P^H (ASTM D 4972), Chloride (BS 812 PART 117), Sulphate (BS 812 PART 118) and Carbonate test was also conducted on the representative soil samples.The results are tabulated in table.

3.13 Direct Shear Test

Direct shear test was conducted on a representative sample to relate shear strength at failure directly to normal stress. Essentially, the sample was subjected to a fixed normal stress and a shear stress was induced along a predetermined plane until shear failure of the soil occurred. This was conducted for cohesionless soil samples [4].

3.14 Geotechnical Properties of the Soil

The soil investigation within the boring and the depth explored revealed that the site is characterized by near-surface deposit very soft clay to soft silty clay that is of low to medium plasticity .The clay unit is conformably underlain by sand of poor to moderate grading. Another clay unit underlies the sand unit and continues to the termination depth. The geotechnical characteristics of the soil were determined from the laboratory and field work. The information gathered and tabulated in (Table 1) is disclosed as follows:

The surface of the sea bottom consist of very soft clay (marine mud) . This unit grades into silty clay down depth. Another clayey unit occurs below the sand unit. The clay is predominantly of moderate plasticity according to the unified system, dark to light grey, soft in strength consistency. The ranges of variations in the index and engineering properties of the clay are summarized in (Table 1).

4. FINDINGS AND INTERPRETATIONS

A careful assessment of the field and laboratory test results revealed that the sand unit is fine to coarse-grained, loose to medium dense in relative density, grayish to dark brown in colour. The deposit overlies sandy clay unit and underlies the soft clay/ soft silty clay unit.

The soil investigation was conducted with the main aim to determining the prevailing subsoil conditions at the proposed project site and to provide geotechnical data, for the evaluation of the type of foundation suitable for the intended structure.

The feasibility of adopting a shallow foundation for the proposed structures may not be possible due to environmental conditions surrounding the project site. Pile foundation is therefore recommended for the project.

In the entire depth of the borehole, the SPT N-values within the sandy deposit ranges between

28-120, while the undrained cohesion for the clay were low (9.0-20.5). The sulphate content of the soil is high and the P^H tends to be slightly alkaline.

5. RECOMMENDATION

The investigation was carried out in accordance with accepted geotechnical practice. The conclusions and recommendations reached in this report are based on the data obtained from the Soil Borings, Piezocone Penetrometer test, field observations and laboratory tests performed. It is not envisaged that soil conditions will vary significantly from those described. However, some variations may be possible. The extent of the variations may not be evidenced until construction work commences. Should such variation occur, it will be necessary to evaluate the geotechnical significance of such variations which could result in further investigation and supplementary recommendations.

COMPETING INTERESTS

Authors have declared that no competing interests exist.

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