GEOTECHNICAL PROPERTIES OF SOILS ALONG SOME COASTAL PARTS OF THE NIGER DELTA, NIGERIA

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Abstract

Geotechnical properties of soils along some coastal communities in parts of the southern Niger Delta was carried out with a view of linking the upland and some coastal communities in the southern Niger Delta and the determination of engineering properties of the subsoils for bridge foundation design. Field studies comprised of drilling of 6 boreholes which included cone penetration tests and standard penetration tests. Laboratory studies included classification tests, triaxial, consolidation and California bearing ratio (CBR). The results revealed that the soil profile consists of a top organic clay (CL) from 0 – 3m, underlain by 6 m thick clayey sand (SC) layer, which is in turn underlain by 10m thick layer uniformly to poorly graded (GP-SP) sands and gravelly. Water table was encountered at the depth of 0.1-1.6m. The clay layers have the following average values: moisture content 80%, LL 76%, PL 26.6%, PI 49.9%, and average organic content of 5.6%. The shear strength ranges from 13.67kN/m^2 – 40.87 kN/m^2, optimum moisture content (OMC), maximum dry density (MDD) and soaked California bearing ratio (CBR) range from 13-22.7%, 1.09 – 1.88mg/m^3, and 1 – 34% respectively. Cone penetration test (CPT) met refusal at 8m depth. A 400mm diameter pile transferred to a depth of 30m produce allowable bearing load of 7.6 x 10^{7}kN. Based on the soil profile and the mechanical design parameters, pile foundation terminating at the depth of 30 meters was recommended at river crossings.

Keywords: Geotechnical properties, Engineering properties, Foundation design, River crossing.

Introduction

Civil engineering construction such as bridges, tunnels, buildings, dams and highways are founded below or on the surface of the earth. For firmness of these engineering construction, soil acceptable for foundation design is needed. This is because foundation of structures constructed on compressible soil often leads to excessive settlement (Roy and Bhalla, 2017). The current increase in cases of building collapse in Nigeria's coastal cities has drawn attention to the importance of proper geotechnical investigation for suitable infrastructural development (Amadi et al., 2012; Abija *et al.,* 2018). Instead of considering gigantic structure, the subsoil and foundation deserves thorough evaluation and characterization of the soil's strength parameters. The solidity of a structure located in the subsurface is hinged on the quality of the foundation, the quality of material used, location of the structure, unconsolidated fills, groundwater table, erosion due to flooding and the presence of expansive clays (Das, 1999; Murthy, 2012).

Geotechnical investigation is undertaken to determine the attributes of the subsurface materials and to understand the stratigraphic analysis of geotechnical systems (Mayne et al., 2002). Soil is an unconsolidated material which can be used for various engineering geological purposes (Clayton *et al.,* 1995). The geotechnical properties are important in determining the suitability of a soil for engineering purposes and not by physical observation nor clear resemblance with other soil types, because the properties of a soil are subjected to different variations (Teme, 2002).

In Rivers State, Southern Nigeria, the proposed Trans-Kalabari Road project, from Rumuelumeni in Obio/Akpor Local Government Area to Tombia in Degema Local Government Area will enhance rural accessibility, development and improve the economy of the area. Therefore, there is need for the geotechnical investigation of the subsurface materials along the proposed road to enable proper engineering design for foundation and pavement. The aim of this work is to determine the engineering properties of the soil types for safe, durable and economic design for foundation and pavement.

Geologic setting of the study area

The Niger delta is distinguished by systems of rivers, creeks and channels which drain the hinterland, transporting sediments and water to the Atlantic Ocean (Abam, 2016). This process has created an extensive sedimentary region, roughly 30,000km^2 in area. The growth of Delta began during the Cretaceous times, the area currently occupied by the Niger Delta was the failed armed of the R-R-R (ridge-ridge-ridge) triple junction. As reported by Short and Stauble (1967), African and South America continents division occurred as a result of the development of the R-R-R triple junction which started the emergence of the Delta. Lithostratigraphically, the Delta has three Formations namely; the Akata, Agbada and Benin Formations (Short and Stauble 1967; Itiowe and Lucas 2020a; Itiowe and Lucas 2020b; Itiowe et al. 2020; Itiowe et al., 2021).

The study area is located in Rivers State of Nigeria, its location lies between Latitude N04° 47' 34.3" and N04° 48' 05.5" and Longitude E006° 5' 23.6" and E006° 55' 14.7". The site is accessible through the Eagle Island Road in Port Harcourt, Rivers State.

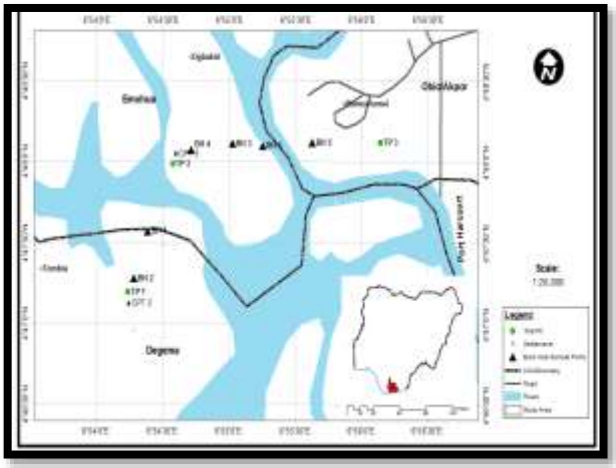

Figure 1. Map of Study Location

Methodology

Field investigation

Six boreholes were drilled to a depth of 18 meters each, using a light weight drilling rig mounted on a tripod. The shell and auger percussion drilling method were employed. Soil samples were collected at regular interval of 1-5 meters. Samples collected were then examined, identified and classified on the field, before being packaged and transported to the laboratory for further analyses. Standard penetration tests were carried out a cohesionless soil. Insitu cone penetration test was also carried out to obtain the soil bearing capacity and was conducted using a 2.5 tons CPT machine. The test involved advancing a 60 feet steel cone with base area of 10cm^3 into the ground with the view to ascertaining the resistance (stiffness) of the soil. This was achieved by securing a winch frame to the ground by means of anchors. These anchors provided the necessary power to punch the cone into the ground at the rate of 2cm/sec. The resistant to penetration registered on a pressure gauge connected to the pressure capsule was recorded in the end series of cone resistance. The sleeve friction reading was plotted against depth and the bearing capacity of sub-soil horizons were calculated. The overall aim was to determine the engineering geological properties of subsoils at the proposed site.

Laboratory investigation

In the laboratory, all the soil samples obtained from the field were subjected to a more detailed visual inspection and description. Thereafter, representative sample from each of the soil samples were used for laboratory tests. Test procedures for field investigation were done in accordance with British Standard BS 5930 (1990), while the test procedures for the laboratory investigation are in accordance with British Standards BS 1377 (1990). The tests carried out during the laboratory investigation include: natural moisture content, atterberg limit tests, particle size distribution analysis, bulk density, organic content determination, undrained triaxial test, consolidation test and California bearing ratio. Details of the test methods are found in standard geotechnical test books (Murthy, 2002).

Results and Discussion

Soil stratigraphy

The soil stratigraphy was obtained from the boring and field penetration test data. Figure 2 shows correlation of the six boreholes. The logs show four soil types between 0 – 18m in the subsurface and they are as follow: Layer I: Inorganic clay top soil, grey; Layer II: Sandy clay, soft, and grayish in color; Layer III: Sand, fine to medium to coarse, grey, gravelly; and Layer IV: Stiff clay, grayish. Table 1 shows soil properties according to layers.

Layer I: Inorganic clay

This layer contains inorganic clay, top soil and grayish layer, these materials represent the top soil in the four borings at the proposed site. This layer extends from a depth of 0 -1.5m, 0-3m and 0-1.5m in boreholes 1, 2 and 5 respectively. It has the thickest depth from 0-10m in borehole 6. Under the Unified Soil Classification System (USCS), this layer can be classified as inorganic clays (CL) of low to medium plasticity.

Layer II: Sandy clay

This layer contains grayish, soft and sandy-clayey layer. This layer underlies the top soil in four of the boreholes. It occurs between 1.5m -5.3m, 3.1-6.2m, 2-5.3m, 0-10m, 1.5-10.3m in boreholes 1, 2, 3, 4 and 5 respectively. The sand is very fine in this layer. These layers have a moderately low moisture content value compared to the first layer.

Layer III: Sand

This layer constitutes grey, fine to coarse gravelly sandy materials. It represents the third layer in boreholes 1, 2 and 5; while it represents the second layer in borehole 4 and 6. This layer extends from a depth of 5.3-9.5m, 6-9.4m, 10.5-17m in boreholes 1, 2 and 5 respectively. These sands are dense and have low moisture content values. The soil type in this layer is uniformly to poorly grade and are good foundation materials.

Layer IV: Stiff clay

This layer contains stiff clay and grayish in colour. It represents the fourth layer in boreholes 1 and 2. This layer extends from depths of between 9.5-12m, 9.3-10.3m in boreholes 1 and 2 respectively. The clay is very stiff with a low moisture content compared to other clay layers encountered in other boreholes.

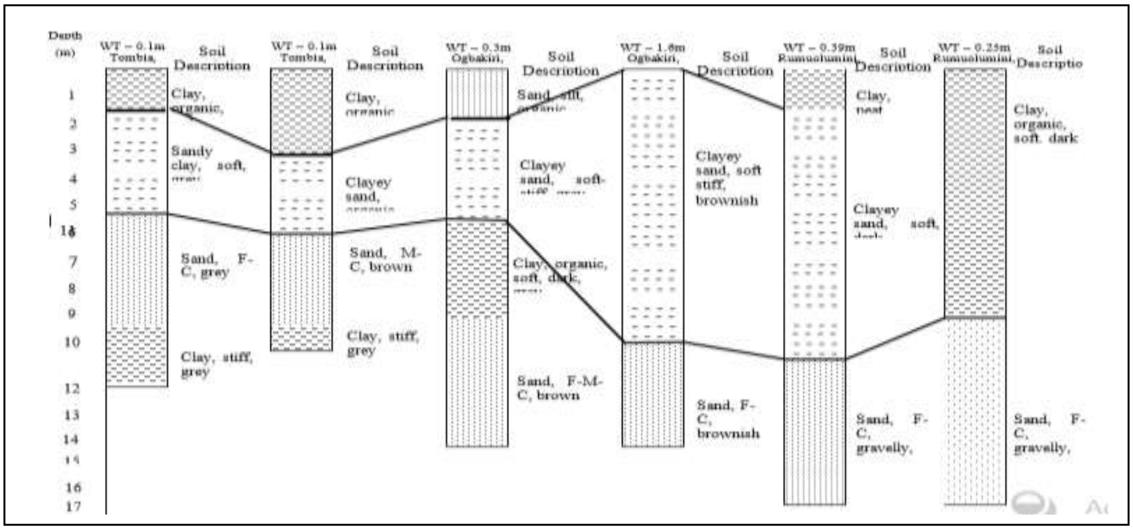

Figure 2. Correlation of BH. 1 – BH 6

Natural moisture content
This is to determine the amount of inter-particle water present in a quantity of soil in terms of its mass. The result of the test reveals that the moisture content values at the top layer of the soil are very high due to the high organic content. Mainly high in borehole 1, 3, 5 and 6, and low in boreholes 2 and 4. This result shows that the inter particle water content present in organic clay is generally much higher than that of the sands (Table 1). The greater the amount of water or moisture a soil contains, the less interaction there will between adjacent particles resulting in a high compressibility and low shear strength of the soil.

Atterberg limits
The liquid limit values vary between 41-186%, 20-55%, 26-29%, 24-37%, 52-78%, 54-64% for boreholes 1, 2, 3, 4, 5 and 6 respectively. The plasticity index values vary between 27-131.6%, 10-28%, 17-22%, 11-19%, 26-38% and 29-30% for boreholes 1, 2, 3, 4, 5, and 6 respectively. When the results of the plasticity and liquid limit of the layers 1 and 4 of boreholes 1 to 6 were plotted on the Casagrande plasticity chart (Figure 3), seventeen of the samples fall above the “A’’ line, which indicate that soil are predominantly inorganic clay of low to medium plasticity. While the four samples in layer 1 and 4 fall below the “A’’ line, which indicates that the four samples are inorganic silt.

Particle size distribution
The result of the particle size distribution (Figures 4) reveals that the sand types in the boreholes range from fine to coarse and gravelly in size. The sands are predominantly uniformly to poorly graded (GU - SP) as shown by the values of their coefficient of uniformity (CU) which fall between 2.21–3.27. According to Hazen (1930) statistical method, the coefficient of permeability (K) is of the order 10-2cm/sec. This shows that the sands are of medium to high permeability, according to the classification of Terzaghi and Peck (1967). The AASHO classification reveals that the sand encountered in the boreholes fall in the A-2-4 clay and indicating good sub grade materials, A-2-4.

Bulk density
The result reveals that the clay soils have high bulk density than the sandy soil. The bulk density for borehole 1 ranges from 2, 718.6 – 93, 398.6kN/m^2, while the bulk density for boreholes 2 and 6 are quite low ranging between 4.24 – 22.32 kN/m^2, the low unit weight values indicate low bearing capacity values for the soil samples.

Organic content
The result of the organic content reveals high organic content of 13.62% in borehole 6, while it varies between 2.3-6.4% in borehole 1. The high organic content values have to be taken into consideration when construction properly starts. Borehole 3 has a moderately organic content value of 2.76%. The top layers of the soil in Boreholes 1 and 6 have to be removed and replaced with less organic and stable material.

Undrained unconsolidated triaxial test
A quick undrained unconsolidated triaxial test was performed on the clay soil samples. Detailed laboratory test is presented in Table 2. The result reveals that the soil have a shear strength (t) of 22.92 kN/m^2, undrained cohesion (cu) of 10kN/m^2, undrained friction angle (ϕ) of 2. The shear strength (t) value for stiff clay in the borehole is 40.87kN/m^2, ϕ of 2, undrained cohesion (cu) of 29kN/m^2. For borehole 4, the shear strength is very high ranging from 103.37 – 104. 29 kN/m^2, (ϕ) of 7 – 8, and undrained cohesion of 57 – 65 kN/m^2 which shows that clay samples in borehole four have high

shear strength (t) values than other clay encountered in other boreholes.

Standard penetration test (SPT)

Standard penetration test (SPT) results show that the soil in layer II which consists of loose, fine to medium to coarse, sometimes gravelly has allowable pressure of 723.9 kN/m^2 and 456.3 kN/m^2 at the depth of 16m and 12m respectively. The allowable net soil pressure (q allow) was calculated from the SPT N-values, using the method of Peck, Hanson and Thornburn (1974). Sand in borehole 1 has N-values of 92 and 58, while some boreholes with medium dense have SPT N-values of 22 and 32 with allowable pressure (q allow) of 173 and 251.7kN/m^2.

Pile capacity analysis

The pile bearing capacity values of the subsurface materials at the proposed site were evaluated by means of the quick undrained unconsolidated triaxial test. The results of the bearing capacity and pile capacity analyses are shown in Table 3. Analysis of the result shows increase in ultimate bearing capacity (qu) of pile and decrease in allowable bearing capacity (qw) of pile, and with increase in depth of penetration of the soil. Thus, the shaft resistance is higher within the dense sand than in the medium dense sand. Hence, the ultimate bearing capacity (qu) is higher in the dense sand than in the medium dense sand. For borehole one, the ultimate bearing capacity (qu) for 5m and 25m are 177.43 kN/m^2 and 761.52 kN/m^2 with an allowable bearing capacity (qw) of 59.14 and 253.84kN/m^2. Boreholes 2 has ultimate bearing capacity (qu) that is very high in the sandy horizon of 15 and 20 meters, the values are 26, 014.0kN/m^2 and 34, 008.1 kN/m^2, with an allowable bearing capacity of 8, 671.33 kN/m^2 and 11, 336.0 kN/m^2. These values imply that the sand is dense and can carry any pile. The pile analysis on the sand confirms with the AASHTO (1993) classification of the sieve analysis result and the CPT result on the sandy horizon of the boreholes, which states that the sandy horizon is good subgrade material, which will be for foundation purpose. The high plasticity and swelling potential of the clays are indicative of the potential foundation problems that will be associated with them.

California bearing ration (CBR) and compaction results

CBR test gives an indication of the shear strength of a soil especially in road construction. The result reveals that Tombia has a MDD and OMC values of 1.3% and 23%, with values of 1% and 4% for soaked and unsoaked. While at Ogbakiri, the MDD and OMC values gave 1.88% and 13%, with values of 7 % and 34% for soaked and unsoaked, which is higher than that of Tombia. The soils in Rumuelimini and Tombia have a low CBR values, while that of Ogbakiri has high, but from AASHTO (1993) classification shows that the clay samples in layer I, II and IV are not good subgrade materials, because they were classified as A- 7- 6.

Cone penetration test (CPT)

CPT bearing capacity values were obtained using the Terzaghi and Pecks (1967). The result reveals that at CPT point 1, the depth of refusal was 5 meters; the resistance value of 6 kN/m^2 was derived. The bearing capacity for points 1 and 2 are is 71.55kN/m^2 and 84.65 kN/m^2. Depths of 6m and 8m have a high bearing capacity of 58.60 kN/m^2 and 80.74 kN/m^2. At 8m deep the density of the soil encountered refusal. For point 3 at Ogbakiri, the bearing capacity gave 76.83 kN/m^2 and 101.58kN/m^2 at the depth of 7m and 8m which is denser. This means that density of a sandy soil influences the bearing capacity result. The bearing capacity increases with increase in depth and density.

Settlement analysis

One-dimensional consolidation test was carried out over a pressure range of 12.5 – 400 kPa on cohesive soil. Coefficient of volume compressibility ranges (mv) for 3m ranges from 5.25 – 3.24m^2/MN, Coefficient of consolidation (cv) ranges from 1.1 m^2/yr to 0.4 m^2/yr. While that for 11.25m, a pressure range of 25 - 800 kPa was carried out on the cohesive soil. Coefficient of volume compressibility ranges (mv) from 3.42 – 1.4 m^2/MN, the coefficient of consolidation (cv) ranges from 53.6 m^2/yr -16.1 m^2/yr (Table 5). The soil ranges from moderately compressible to high compressibility. Settlement analysis result reveals that total settlement of the soil is 6.1 x 10^{-1} to 9.8 x 10^{-2} cm and rate of settlement of t50 is 1.80 m^2/yr. to 4 x 10^{-2} m^2/yr. which the rate of settlement for t90 is 7.78 to 1.7x10^{-1} (Table 6).

Foundation discussion

The lithology logs reveal the occurrence of sand between 4.5m – 9m and 10m – 18m. SPT N' value corrected are classified as medium to dense sands. The bearing capacity analysis reveals that the dense sand has higher ultimate and allowable bearing capacity between 761.52 kN/m^2 and 253.84 kN/m^2 and pile bearing capacity analysis for base resistance in sand and ultimate shaft resistance of 9.3 x 10-5 kN/m^2 to 7.6 x 10-4 kN are therefore preferable for the termination of pile. End bearing steel pile terminates within the medium dense to dense sandy horizon of 15 to 18 meters deep. While the pile should terminate at 30m for the design of the foundation of the bridge. Pile loading test which determines the load bearing capacity of a pile should be carried out. This test should not be conducted until at least thirty days after driving the pile to permit pile soil adjustment to be completed. Table 7 shows summary of the engineering properties of the soils.

Table 1: Natural moisture content results

Layer	Description of soil	Moisture content (%)					
		BH. 1	BH. 2	BH. 3	BH. 4	BH. 5	BH. 6
I	Clay, inorganic, grey	38.8	25	26.3	0	198.3	126.5
II	Sandy clay, soft, grey	148.5	13	26.3	22.6	59.8	0
III	Sand, F-M-C, gravelly, grey	14.7	13	14.9	19.6	67.0	19.9
IV	Clay, stiff, grey	23.8	20.4	0	0	0	0

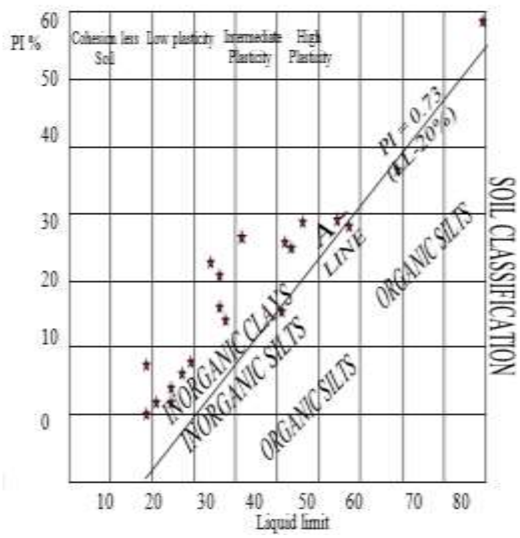

Figure 3. Casagrande plasticity chart particle size distribution

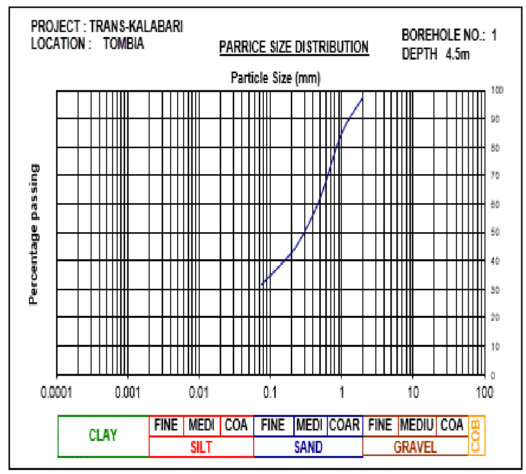

Figure 4. Particle size distribution curve

Table 2. Undrained triaxial test

BH.	Depth (m)	Cohesion interception cu (kN/m^2)	Effective normal stress σ_n (kN/m^2)	Angle of shear resistance (ϕ)	Shear strength T = C + σ_n tan θ
1	1.5	10	370	2	22.92
	11.25	29	340	2	40.87
2	2	10	320	1	15.58
3	3	9	325	2	20.34
	6	8	325	1	13.67
4	3	57	330	8	103.37
	8	65	320	7	104.29
5	6	12	340	1	17.93
6	9	10	330	1	15.76

Table 3. Pile analysis

BH.	Depth (m)	P^1 (kN/m^2)	ϕ^0	Nq-1	P^1_{av}	Q_b kN	Qs kN	Qult kN	F.S	Qallow kN
1	4.5 - 9	30,451.0	2	0.22	15,225.5	1.4×10^{-4}	1.2×10^{-3}	1.4×10^{-3}	3	4.6×10^{-4}
	9 – 15	48,870.15	2	0.22	24,435.5	9.3×10^{-5}	7.6×10^{-4}	8.57×10^{-4}	3	2.8×10^{-4}
	15 – 20	48,932	2	0.22	24,466.0	9.2×10^{-5}	7.8×10^{-4}	8.7×10^{-4}	3	2.9×10^{-4}
	20 - 25	50,418	2	0.22	25,209	9×10^{-5}	7.6×10^{-4}	8.5×10^{-4}	3	2.8×10^{-4}
2	0-3	32.7	1	0.1	16.35	132766.5	34511.19	1.6×10^{5}	3	5.5×10^{4}
	3-6	62.16	1	0.1	31.08	252134.6	65715.34	3.1×10^{5}	3	1.0×10^{5}
	6-9	61.89	38	76.5	30.94	1.9×10^{8}	65419.3	1.9×10^{8}	3	6.3×10^{7}
	10-15	90.75	1	0.1	45.37	3.6×10^{5}	9.5×10^{4}	4.5×10^{5}	3	1.5×10^{5}
3	10 – 15	112.7	2	0.22	0.73	4×10^{-2}	3.4×10^{-1}	3.8×10^{-1}	3	1.2×10^{-1}
4	0-11	242.44	8	1.21	121.22	1.1×10^{7}	2.5×10^{5}	1.1×10^{7}	3	3.7×10^{6}
	11-15	110.2	34	51.64	55.1	2.3×10^{8}	1.1×10^{5}	2.3×10^{8}	3	7.6×10^{7}
5	0-2	8.48	1	0.1	4.24	3.4×10^{4}	8.9×10^{3}	4.2×10^{4}	3	1.4×10^{4}
	2-12	100	2	0.22	50	8.9×10^{5}	1.0×10^{5}	9.9×10^{5}	3	3.3×10^{5}
	12-18	141.96	35	56.75	70.98	3.2x10	1.5×10^{5}	3.2×10^{8}	3	1.0×10^{8}
6	10 - 18	155.2	1	0.1	0.73	6.4×10^{-2}	2.4×10^{-1}	3.1×10^{-1}	3	1×10^{-1}

Table 4. Consolidation test (one dimensional) Tombia

Borehole No.	Depth of sample (m)	Pressure range (kPa)	Coefficient of consolidation, Cv (m^2/yr)	Coefficient of volume compressibility, mv (m^2/MN)	Coef. of permeability, k (cm/sec) $X10^{-8}$	Description of sample
1	3.0	12.5-25	1.1	5.25	17.9	Clay, high Compressibility
		25-50	1.3	4.36	17.6	
		50-100	1.7	4.84	25.6	
		100-200	0.4	4.09	5.1	
		200-400	0.4	3.24	4.0	
Natural moisture content= 85.4%		Void ratio e_o =2.336		Degree of saturation=100%	Specific gravity = 2.74	Bulk unit weight= $14.9kN/m^3$

Table 5. Consolidation test (one dimensional) Tombia

Borehole No.	Depth of sample (m)	Pressure range (kPa)	Coefficient of consolidation C_v(m^2/yr)	Coefficient of volume compressibility mv (m^2/MN)	Coef. of permeability k(cm/sec) $X10^{-8}$	Description of sample
1	11.3	25-50	53.6	3.42	57.0	Clay, moderate compressibility
		50-100	29.2	4.62	41.9	
		100-200	32.5	4.44	44.8	
		200-400	18.1	2.71	15.2	
		400-800	16.1	1.47	7.3	
Natural moisture content= 34.2%		Void ratio e_o =0.950		Degree of saturation=90%	specific gravity = 2.56	Bulk unit weight= $17.3kN/m^3$

Table 6. Summary of settlement analysis result

BH	Depth (m)	Consolidation Coefficient 0.009, CC (LL – 10)	Initial Void Ratio, eo	Thickness of sample, H	Initial over burden pressure (kN/m^2), P_o	Weight of Initial Building to be Built, P_Δ	t_{50} % (m^2/yr)	t_{90} % (m^2/yr)	Total Settlement, S (cm)
1	3	1.584	2.336	3	67.65	120	1.80	7.78	6.1×10^{-1}
	11.25	0.378	0.950	2.5	194.62	120	4×10^{-2}	1.7×10^{-1}	9.8×10^{-2}

Table 7. Summary of the engineering properties of the soils

Parameter	Minimum	Maximum	Average
Thickness (m)	3	10	6.5
Natural Moisture Content (%)	1	10	5.5
Liquid Limit %	20	78	49
Plastic Limit %	9	54.4	31.7
Plasticity Index %	10	38	24
% fines (< 0.074mm)	5	10	5
% Sand	10	38	24
Bulk Density kN/m^2	4.24	93,398.6	46,701.32
Undrained Triaxial (kN/m^2)	13.67	104.29	58.98
CBR %	4	34	19
Organic Content Determination	2.3	6.4	4.35
Angle of Internal Friction (θ)	1	8	4.5
CV (m^{2}yr) Coefficient of Consolidation	0.4	1.7	1.05
Mv (kN/m^2) Coefficient of Compressibility	3.24	5.25	4.245
SPT	17	53.5	35.25
CPT	5	8	6.5
USC Classification	CH	CL	MH

Conclusion

Geotechnical properties of soils along some coastal parts of the Niger Delta, Nigeria have been carried out successfully. From the results and discussion, the following conclusions were drawn: layer I composes of inorganic clay, top soil, grey; layer II constitutes of sandy clay, soft, and grayish; Layer III contains sand, fine to medium to coarse, grey, gravelly; layer IV has stiff clay, grayish. The clay samples present are medium to high plasticity and have low shear strength. This will lead to detrimental settlement of any structure if they are used as foundation material. Furthermore, the clays are moderately to highly compressible and have high organic content. The sands are uniformly to poorly graded and are dense. The dense horizon of the sand has high ultimate and allowable pile bearing capacity results than the medium dense region. The CPT met a depth of refusal at 8 meters as a result of the density of the sand. The results reveal that the sand bodies encountered in the boreholes of the horizon are good foundation materials for the construction of the road, as subgrade materials are suitable for the foundation of the proposed bridges along the river crossing or any civil engineering structure.

Recommendations

1. The design of the proposed road will require sand of medium – coarse grain size for the purpose of back fill material for subgrade up to 3 meters deep.
2. The foundation design for the proposed bridge should be a deep foundation with pile. An end bearing steel piles that can be terminated within the dense horizon of 18 and 20 meters should be used for the construction of the proposed bridge. The choice of deep foundation is to enable the transmitted structural load that will be at the denser horizon of the sand.
3. Base on the high presence of organic content particularly at the top layer of the soil we recommend steel pile.
4. Pile load test is strongly recommended to a certain the actual pile load carrying capacity.

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