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Subsoil Geotechnical Evaluation for the Design and Construction River Bridge Along Onuinyang Ikuano Road, Bende LGA, Abia State, Nigeria

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ABSTRACT

This study investigates the ground condition and engineering properties of the subsurface soils along Onuiyang Ikuano Road in Bende LGA, Abia State, Nigeria using geotechnical method. The aim is to determine geotechnical parameters for the design and construction of the abutement and pier of a river bridge to link two sections of the road. The investigation adopted both field methods which included boring, soil sampling and SPT, and laboratory tests to determine moisture content, Atterberg limits, particle size distribution, soil shear strength parameters such as cohesion and internal friction angle. The results indicated three (3) distinct lithologic units with varied thickness. They are soft to medium brown lateritic clay at the top followed by a loose to medium dense, fine to medium sand. The third layer is a medium dense, fine to medium sand. The SPT N-values varied from 6 to 27 at various depths. The moisture content ranged from 11.9 to 37.4%. The liquid limit varied between 20 – 42% while the plastic limit is between 9 and 31%. For the shear strength parameters, cohesion ranges from 56 to 69 KN/m^3 with corresponding frictional angle of 8^o to 9^o respectively. A pile foundation to be founded at 20.0m depth was recommended based on the geotechnical information of the soil and in-situ densities of the soil. Allowable load carrying capacities of the piles were computed to be between 600.2 and 646.2 KN for 450mm diameter pile, and 1050.9KN and 1128.6 KN for a pile of 600mm diameter respectively. The study highlights the critical role of geotechnical surveys in design of safe, resilient and sustainable civil structures. However, it is recommended that the foundation engineer should confirm the proposed capacities and eventually design the foundation based on the specific loadings.

Keywords: Geotechnical evaluation, pile foundation, soil shear strength, load carrying capacity, abutement and pier

INTRODUCTION

As part of development efforts, a road was proposed to link some communities in Bende Local Government Area of Abia State, Nigeria. However, a river separating the communities calls for need to construct a River bridge to link the two ends of the road. A detailed geotechnical soil investigation for the proposed bridge becomes necessary to provide geotechnical parameters for sustainable foundation design of the bridge.

The complexity of the investigation depends on the nature of the ground conditions and the type of structure (Bell, 2011). A site investigation, if properly done, foresee and provide against difficulties that may arise during construction due to poor ground conditions. It often reveals the nature of the subsurface strata and the groundwater conditions, thus, providing information that aid in the design of civil structures (Warmete & Nwakwoala, 2018).

Furthermore, geotechnical investigations play a pivotal role in assessing subsurface conditions and properties critical for designing effective engineering structures. These investigations involve methods such as boring and soil sampling, Standard Penetration Test (SPT), Cone Penetration Tests (CPT), and laboratory analysis which provide essential data on soil strength, density, and other engineering properties (Palmisano & Rus, 2023).

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Geotechnical properties of soil such as specific gravity, density index, particle size distribution, and shear strength are determinants of soil behavior under load (Roy & Dass, 2012). Shear strength is particularly critical for the stability of any superstructure. Analysis of shear strength parameters of cohesion and angle of internal friction enable engineers to predict the soils response to imposed vertical structural load. Inadequate assessment of shear strength can lead to structural failures which can result to costly repairs (Roy & Dass, 2012; Agbede *et al.,* 2014).

It is in view of the above that this investigation was carried out at the project site to provide appropriate ground information for design of the foundation systems for the abutement and pier locations of the bridge.

Project Location

The project is located along the Onuiyang-Ikuano Road in Bende Local Government Area of Abia State in the Niger Delta. The local geology of the area is typical near-surface lateritic deposits which is underlain by medium sand consisting of series of sand stress reflecting shallow marine to coastal depositional environment (Obaje, 2009).

Geological Setting of the Location

The location is an integral part of Southern Nigeria. The area falls within the Cretaceous – Recent Sediments of South-Eastern Nigeria consisting of shale and limestone belonging to the Asu river group (Anthony *et al.,* 2022).

The soil sediments consist of rather poorly bedded sandy shale which is known as the Abakalike shale with embedded limestone lenses. The limestone beds have thickness of 30m in some places. The area belongs to beds of the south-eastern portion of the Calabar basin assigned to the Odukpani formation, the Ajali Formation as well as the lower coal measures known as “Mamu” Formation.

There is a coal bearing section of the Formation which is predominantly milestone and sandy clay. The formation sediments were deposited during the late Tertiary to Early Cretaceous period. The Ajali sand-stone consist of thick, friable, poorly sorted sandstone and often marked by repetitive banding of coarse and fine-grained layers (Agumanu, 1989). The lower coal measures comprises mainly course-grained, alternating sediments of grey sand, dark, sandy shale and carbonaceous shale containing think band of impure coal in place at various horizons, with estimated thickness of between 300m to 350m (Obaje, 2009). These formations give rise to the line of prominent hills along the Eastern margins of the escarpment.

MATERIALS AND METHODS

The study involved both field and laboratory analysis. Field investigation includes soil boring and sampling, and in-situ Standard Penetration Test (SPT).

Soil Sampling and In-situ Testing

Soil sampling and in-situ characterization of sub-soils were done using percussion soil boring procedure. The soil sampling was conducted with light Cable Percussion Rig. The shell/auger device of the rig cuts through the soil to penetrate the hole whereby soil materials are retained inside the barrel by means of a clerk. The barrel is then brought to the surface where the soil retained in it is emptied out. To prevent collapse of the borehole wall, 4-inch diameter steel casing were used to line the hole. Representative samples were taken at regular intervals of 1.0m depth or where there is change in soil strata.

Relatively undisturbed soil samples were taken from the borehole using a Shelby Tube Sampler. The sampler was connected to the bottom of the drilling rods, and is lowered to the bottom of the borehole. Soil sample was collected with minimal disturbance. Upon extraction, the recovered UDS were sealed at both sides of the tubes in order to maintain the soil samples natural condition.

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Standard Penetration Test (SPT)

Standard Penetration Test (NSPT) was taken to determine the in-situ densities of the soil. A 50mm diameter split spoon sampler is driven 450mm into the soil using a 63.5kg hammer with a 760mm drop. The penetration resistance is expressed as the number of blows (N-value) required to obtain 300mm penetration below an initial 150mm penetration through any disturbed ground at the bottom of the borehole. The test procedure followed ASTM D-1586 provisions.

Laboratory Testing

Soil and groundwater samples were tested to determine some geotechnical parameters. Laboratory analysis was done in accordance with BS/EN/ISO 17892 and the groundwater sample was tested in accordance with APHA4500.

The following tests were carried out;

i. **Classification Tests:** These are the index properties of the soil namely;

- Moisture Content (Wn)
- Atterberg Limits (Liquid and Plastic Limits)
- Particle Size Analysis
- Specific Gravity
- Unit Weight

ii. **Soil Strength Tests:** This is the strength parameters of the soil.

- Undrained Triaxial Test
- Direct Shear Box Text

ii. **Chemical Properties:** The chemical parameters of the groundwater were evaluated for potential acidic level, chloride and sulphate content.

Moisture Content: The water content was determined by drying selected moist/wet soil materials for at least 6 hours to a constant mass in a 105ºC drying oven. The difference in mass before and after drying was used as the mass of the water in the test material.

Atterberg Limits

Atterberg limits were determined on soil specimens with a particle size of less than 0.425mm. These limits refer to arbitrarily defined boundaries between the Liquid Limit (LL) and the Plastic Limits (PL) states, and between the plastic and brittle states of fine grained soils. They are expressed as water content in percentage.

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 liquid test was carried out. The PL is the water content at which a soil can no longer be deformed by rating into 3mm diameter threads without crumbling. The range of water content over which a soil behaves plastically is the plasticity index. This is the difference between the Liquid Limit (LL) and the Plastic Limit (PL). That is PI = LL – PL.

Particle Size Analysis (PSA)

Particle size analyses were performed by means of sieving. Sieving was carried out for particles that would be retained on a 0.075mm sieve. Dry sieving was carried out by passing the soil sample over a set of standard sieve

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sizes and then shakes the entire unit for few minutes with the sieve shaker. 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 co-efficient of uniformity Cu, where Cu = D_{60}/D_{10}, and the co-efficient of curative Cc, where Cc = $(D_{30})^3$ / D_{10} x D_{60}. D_{60}, D_{30} and D_{10} are effective particle sizes indicating that 60%, 30% and 10% respectively of the particles (by weight) are smaller than the given effective size.

Unit Weights

Unit weights were determined from measurements of mass and volume of the soil. The unit weight γ(KN/m^3) 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.

Unconsolidated Undrained Triaxial Test

This test was performed on relatively undisturbed clay soil. The specimen was prepared by trimming the sample. A latex membrane with thickness of approximately 0.2mm was placed around the specimen. A lateral confining pressure of 100kPa to 300kPa is maintained during axial compression loading of the specimen. Consolidation and drainage of pore water during testing is not allowed. The test is a deformation controlled, single stage and stopped when an axial strain of 15% is achieved. The deviator stress is calculated from the measured load assuming the specimen deformed as a right cylinder.

The results are presented as plot of deviator stress versus axial strain. The undrained shear strength Cu is taken as ½ the maximum deviator stress. When a maximum stress has not been at strains of less than 15%, the stress at 15% is used to compute undrained shear strength (Abam, 2017).

Shear Box Test

Consolidated – drained test performed on the Shear Box Assembly mainframe device on sand was used to determine the failure envelop of the soil. The sample was first subjected to an all-round confirming pressure δ_3 by compression of the chamber fluid. Normal load is applied to the specimen and the specimen is sheared at the pre-determined horizontal plane between the two halves of the shear box measurement of the shear load, shear displacement and normal displacement are recorded. The test is repeated for two or more identical specimens under different normal loads. From the results, the shear strength parameters were determined. The strength of the soil depends in its resistance of shearing stresses which depends on the friction between individual particles and adhesion between the soil particles.

Water Chemistry Analysis

Physio-chemical tests were conducted on the water samples to determine the pH values, chloride and sulphate. Standard laboratory methods were used and adequate precautions were taken as regard to sample containers handling and preservation.

RESULTS AND DISCUSSION

Subsoil Stratigraphy

Below is a summary presentation of the subsoil stratigraphy at the project location which is also presented in Figure 3.

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Borehole 1/ABT 1

The first 3m (0 – 3.0m) is made up of soft to medium brown lateritic clays. Below this layer is a loose to medium dense, brown fine to medium sand with some gravel. This layer is about 11m thick (3 – 14m). N_{SPT} blows ranged from 7 to 19. From 14.0m to the end of boring at 20.0m is medium sand with N_{SPT} values from 20 to 27 blows. Static groundwater level was 3.0m after soil boring.

Borehole 2 PIER

This boring was in water. The water height at the time of boring was 0.50m from the riverbed. From the riverbed to 3.0m is red, loose to medium dense, fine sand which is immediately underlain to the end of boring by medium dense, grey fine to medium sand with gravel. N_{SPT} blow range from 15 to 31.

Borehole 3/ABT 2

This borehole is characterized by soft to firm, brown lateritic clay from top to about 2.0m (0.0 – 2.0m) and followed by brown fine sand with clay-mix up to 3.0m (a thickness of about 1.0m) underlying the clayey fine sand materials is loose to medium dense, grey coloured fine to medium sand with gravel which extends to the terminal depth at 20.0m. The N_{SPT} values range from 6 to 27 blows. Groundwater table was at 3.10m after boring.

Particle Size Distribution

The particle size distribution of the subsoils is presented in Table 1. A summary of the table shows that the effective particle size (D_{10}) ranged from 0.09 to 0.20mm while the co-efficients of uniformity (Cu) and curative (Cc) have values ranging from 2.50 to 4.44 and 0.93 to 1.69 respectively. Also, the co-efficient of permeability (k) is from 0.00065 to 0.00320m/sec.

Table 1: Summary of geotechnical properties of the subsoils

SAMPLE		SPECIFIC GRAVITY Mg/m^3	BULK UNIT WEIGHT kN/m^3	Angle of Friction $Ø_U$	Effective Particle Size D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	Coef. of Uniformity $Cu=D_{60}/D_{10}$	Coef. of Curvature $Cc=D_{30}/(D_{10}*D_{60})$	Coef. of Permeability $K=C*D_{10}$ (m/sec)
BH No	Depth (m)									
BH-1/ABT-1	5.00	2.65	20.1	25	0.10	0.2	0.35	3.50	1.14	0.00080
	7.00	2.63			0.12	0.31	0.48	4.00	1.67	0.00115
	12.00	2.65			0.12	0.20	0.36	3.00	0.93	0.00115
	20.00	2.62			0.20	0.34	0.50	2.50	1.16	0.00320
BH-2/PIER	1.00	2.64			0.14	0.30	0.40	2.86	1.61	0.00157
	4.00	2.61	19.8	26	0.14	0.31	0.46	3.29	1.49	0.00157
	8.00	2.66			0.09	0.21	0.40	4.44	1.23	0.00065
	12.00	2.63	19.6	29	0.18	0.36	0.65	3.61	1.11	0.00259

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	15.00	2.62			0.14	0.32	0.43	3.07	1.70	0.00157
	18.00	2.65	19.4	30	0.13	0.30	0.40	3.08	1.73	0.00135
BH-3	2.00	2.63			0.09	0.21	0.40	4.44	1.23	0.00065
	10.00	2.64	19.7	24	0.12	0.20	0.36	3.00	0.93	0.00115
	14.00	2.61	19.8	27	0.12	0.23	0.40	3.33	1.10	0.00115
	18.00	2.64			0.14	0.30	0.40	2.86	1.61	0.00157

Atterberg Limits

Table 2 presents the Atterberg limits of the subsoils which consist of the liquid limit, plastic limit and the liquidity index. Also the moisture content is presented in Table 2. The liquid limit (LL) and the plastic limit (PL) range from 20 – 42% and 9 – 31% respectively while the liquidity index is between 0.26 and 0.58. The moisture content is averaged 26.42%.

Table 2: Atterberg limits of the subsoils

Borehole No.	Depth (m)	Moisture Content (%)	Atterberg Limits			
			LL(%)	PL(%)	PI(%)	LI(%)
BH – 1 / ABT – 1	1.00 3.00	32.1 37.4	40 42	25 31	15 11	0.47 0.58
BH – 3 / ABT – 2	1.00 3.00	11.9 24.3	20 28	9 18	11 10	0.26 0.63

LL = Liquidity Limit, PL = Plastic Limit, PI = Plasticity Index, LI = Liquidity Index

Values of the Specific Gravity (S_G) Bulk and Dry unit weights and also the Saturated (Submerged) unit weights with depth are presented in Table 3. The specific gravity of the soils averaged 2.63mg/m3 while the submerged unit weight is between 8.1 and 12.2KN/m^3.

Table 3: Unit weights of the subsoils

Borehole No.	Depth (m)	Specific Gravity (S_G)	Unit Weights		
			Bulk Unit wt (KN/m^3)	Dry Unit wt (KN/m^3)	Submerged Unit wt (KN/m^3)
BH – 1 / ABT – 1	1.00 3.00	2.61 2.65	18.4 17.9	13.9 13.1	8.6 8.1
BH – 3 / ABT – 2	1.00 3.00	2.63 2.63	22.0 19.6	19.7 15.7	12.2 9.8

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Shear Strength

The shear strength parameters, cohesion (Cu) and angle of friction (θ) are presented in Table 4. Borehole 1 has average undrained cohesion of 56KN/m^3 with average frictional angle of 9^o which Borehole - 3 has undrained cohesion and frictional angle of 69KN/m^3 and 8^o respectively.

Table 4: Shear strength parameters of the soils

Borehole No.	Depth (m)	Undrained Cohesion Cu (kN/m^2)	Angle of Friction Øu (θ)^o	Soil Type (USCS)
BH – 1 / ABT – 1	1.00 3.00	56	9	CL CL
BH – 3 / ABT – 2	1.00 3.00	69	8	CL CL

Geochemical Test on Groundwater

Physio-chemical tests were conducted on the water samples to determine the pH, chloride and sulphate values following standard methods (APHA 4500). The results are presented in Table 5.

Table 5: Result of groundwater geochemical analysis

Borehole No.	pH	Chloride (mg/l)	Sulphate (mg/l)	Remark
BH – 1 / ABT – 1	7.7	56.3	18.6	Values below the threshold values
BH – 3 / Pier	7.2	42.8	16.1	

From the analysis, the range of values for the pH is 7.2 – 7.7 which is alkaline while the values of chloride and sulphate are from 42.8 – 56.3mg/l and 16.1 – 18.6mg/l respectively.

DISCUSSION

The investigation was carried out for the purpose of evaluating the subsoils engineering properties for foundation designs for a proposed bridge. From boring and laboratory test results, the site is characterized by soft to medium lateritic clay at the top followed by loose to medium dense, fine to medium sand. There is a little variation in the soil samples as there is substantial resemblance among the boreholes.

The loose to medium dense, fine to medium sand may exhibit contractive behavior when sheared. This is coupled with the presence of clay mix within this layer at BH – 3/ABT – 2.

Foundation System for the Bridge

For design purpose, geotechnical information of the soil and in-situ densities of the soil shall form the basic input for foundation estimates (Dickenson *et al.,* 2021) The simple and general geological sequence across the site has been established and fully described, hence recommendations on the most suitable foundation system are provided considering piled foundations.

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The soil stratification as presented in Figure shows a clay upper strata of about 3.0m thickness. This layer is weak and compressible. From 14.0m to 20.0m in borehole 1 and 3.0 to 20.0m in borehole 2 is a medium dense, fine to medium sand with gravel. The SPT N-values also increases with depth within the coarse fraction depth, an indication of denser soil at deeper depth. In terms of competent bearing stratum for the pile, the medium dense sand and gravel with SPT N-values ranging from is considered the most competent layer. The piles should preferably be installed by Continuous Flight Auger (CFA) method or Cast-in-Place (CIP) method to avoid the possibility of negative pore pressures within the medium dense sand layer due to driving in of the piles. The preferable depth of 20.0m is also denser with SPT N-values in the range of 27 to 31.

Pile Foundation Estimates

Load carrying capacity of a single pile diameters 450mm and 600mm driven to depth 20.0m are estimated using equation by Thurman (1964) and corroborated by Meyerhof (1976), Tomilinson (2001), Murthy (2017). Design details for the piles are based on the soil profile and correlated to the following ground information in accordance with EN1536:2009. Soil information related to foundation design are:

(i) Thickness of the various layers
(ii) Standard penetration values (N_{SPT})
(iii) Skin friction of the soil (Morrison & Esonanjor, 2022).

Load carrying capacity for single axially loaded pile is estimated based on the equation below:

$$Qult = Qs + Qb \quad (1)$$

where; Qult = ultimate bearing capacity

Qs = skin friction

Qb = end bearing

Qs = q_s x As

For cohesionless soil (C = 0)

$$Qs = q \times ks \times \tan\varphi \times As \quad (2)$$

For cohesive soil ($\varphi = 0$)

$$Qs = a \times Cu \times As \quad (3)$$

Where q = effective overburden pressure of the soil over the embedded depth of pile

Ko = earth pressure co-efficient at rest

As = cross-sectional area of the pile

a = adhesive factor (taken as 0.45)

Cu = undrained shear strength of clay (kPa)

Qb = q_b .Ab

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For cohesionless soil (C = 0)

$$Qb = q \times Nq \times A_b \quad (4)$$

For cohesive soil (p = 0)

$$Qs = C_b \times Nc \times A_b$$

Load Carrying Capacity of Single Ø450mm Bored Pile at ABT-1 at 20.0m

Skin Friction (Qs) of the clay at 3.0m

$$Qs = P \times \Delta L \times F \quad (5)$$

F = C x α = (56 x 0.45) = 25.2 Cohesion at clay layer = 56 kN/m²

where; P = Perimeter of Pile = π x D = 3.142 x 0.45m = 1.414 m

ΔL = Change in Length

D = diameter of pile

Qs = 1.414 x 3.0 x 25.2 = 106.9kN

Skin Friction of Pile at 17.0m (C= 0)

$$Qs = P \times L \times fs \quad (6)$$

Where; fs = K x q' x Tan(3/4 x 25) = ultimate value of the tangential force per unit area of the pile due to adhesion and skin friction.

q' = γ x L x D = 20.1 x 17.0 x 0.45 = 153.8 kN/m²

fs = 0.577 x 153.8 x Tan 18.75 = 30.1 kN/m²

Qs = 1.414 x 17.0 x 30.1 = 723.5 kN

Total skin fricktion of pile diameter 450mm at 20.0m is (106.09 + 723.5) = **830.4KN**

End Bearing of Pile at 20.0m

Qb = Ab x overburden pressure x Nq (derived from Meyerhof's shape and depth factor table) Ab x q x Nq

Ab = Cross-section of pile = $\pi D^2/4 = 3.142 \times 0.45^2 2/4 = 0.159m^2$

q = effective overburden pressure at 20.0m

= unit weight of soil (thickness)

= (17.9 x 3.0) + (20.1 x 17.0) = 53.7 + 341.7 = **395KN/m²**

Nq = 10.66

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Qb = 0.159 x 395.4 x 10.66 = **670.2kN**

Ultimate bearing capacity of single pile

Qu = Qs + Qb = 830.4 + 670.2 = 1500.6KN

Allowable bearing capacity at 20.0 with FS = 2.5

(FS = Factor of safety) = 1500.6/2.5 = **600.2kN**

Load Carrying Capacity of Single θ450mm Pile at Pier at 20.0m

Cross section of pile (Ab) = $\pi D^2/4$

= $3.142 \times (0.45)^2 /4$

= **0.159 m²**

Perimeter of pile (P) = π x D D where D = diameter of pile

= 3.142 x 0.45 = **1.414 m**

Overburden Factor (Meyerhof's Chart) Nq = **14.72**

Effective overburden pressure (q)

(19.6 - 9.81) x 20.0m = 195.8kN/m²

End Bearing of Pile at 20.0m (Qb)

Qb = Ab x q x Nq = 0.159 x 195.8 x 14.72

= **458.3 kN**

Skin Friction of Pile at 20.0m

α = γ x L x D = 19.6 x 20.0m x 0.45

= 176.4 kN/m²

f = K x q x Tan(3/4 x 28)

= 0.531 x 176.4 x Tan 21 = 36 kN/m²

where tan φ = frictional angle between pile and soil at substratum

Q_s = 1.414 x 20.0 x 36 = **1018.1kN**

Ultimate bearing capacity (Q_u)

$Q_u = Q_b + Qs$

= 458.3 + 1018.1 = 1476.4kN

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Allowable beating capacity at 20.0m for θ450mm pile is Qu/FS where FS is Factor of safety (2.5)

Q_a = 1476.4/2.5 = **590.6kN**

Load Carrying Capacity of Single θ450MM Bored Pile at ABT-2 at 29.0m

Cohesive strength (Cu) = 69kN/m^2

Cross-section of pile (Ab) = $\pi D^2/4$ =

= 3.142 x $(0.45)^2/4$ = **0.159m^2**

Perimeter of Pile (P) = π x D = 3.142 x 0.45 = **1.414m**

Effective Overburden Pressure at 20.0m;

= (22.0 x 3.0m) + (19.8 x 17.0m)

= 66.0 + 336.6 = 402.6kN/m^2

Skin Friction Qs of the clay at 3.0m

Qs = P x ΔL x F

F = c x α = 69 x 0.45 (α = 0.45 = adhesive factor assumed)

= 31.1kN/m^2

Qs = 1.414 x 3.0 x 31.1 = **131.9kN**

Skin Friction of the Pile at 17.0m

α = γ x L x D = 19.8 x 17.0 x 0.45 = 151.5 kN/m^2

f = K x α x tan(3/4 x 26) = 0.5.62 x151.5 x Tan 19.5

= 30.2kN/m^2

f = K x q x Tan(3/4 x 26) = 0.562 x 151.5 x Tan 19.5

= 30.2 kN/m^2

Qs = 1.414 x 17.0 x 30.2 = **725.9 kN**

Total Skin Friction of θ450 pile at 20.0m is:

(131.9 + 725.9) = **857.8 kN**

End Bearing pile at 20.0m = $Q_b = A_b \times q \times N_q$

= 0.159 x 402.6 x 11.85 = **758 kN**

Ultimate bearing capacity of single pile

$Q_u = Q_s + Q_b = 857.8 + 758.6$ **= 1616.4kN**

Allowable bearing capacity at 20.0m with FS = 2.5

Qa = 1616.4 / 2.5 = **646.5 kN**

Similar procedure was used to compute the load carrying capacity of single θ600mm bored pile at ABT – 1 and ABT – 2 at 20.0m, and load carrying capacity of single θ600mm pile at pier at 20.0m. The values are presented in Tables 6 and 7.

Table 6: Allowable load carrying capacities of 450mm single piles

Depth (m)	450mm Diameter Pile Capacity (KN)		
	ABT – 1	Pier	ABT – 2
20.0	600.2	590.6	646.2

Table 7: Allowable load carrying capacities of 600mm single piles

Depth (m)	600mm Diameter Pile Capacity (KN)		
	ABT – 1	Pier	ABT – 2
20.0	1050.9	1050.5	1128.6

CONCLUSION

The investigation was carried out for foundation design parameters for the proposed Rivers Bridge. The investigation involved 3nos. soil borings to 20.0m with soil sampling and SPT. Labouratory analysis of the soil samples and chemical screening of the gropundwater samples were carried out. The results, both field and laboratory geotechnical analysis are presented in various tables, logs and plots. Based on these and the expected load, foundation recommendation for the proposed bridge is given which is a deep pile foundation.

It is believed that the design and construction of all foundation and other earth works will be carried out in accordance with good engineering practice for sustainable result.

Limitation

The subsoils investigation has been carried out in line with acceptable standards and our findings and recommendations were based on the results of field studies and laboratory tests. We therefore do not expect any or much variation during construction. However, it is suggested that the foundation engineer should confirm the proposed capacities and eventually design the foundation based on the specific loadings.

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Minor Principal Stress	Deviator Stress	Major Principal Stress
σ_3 (kN/m²)	$\sigma_1 - \sigma_3$ (kN/m²)	σ_1 (kN/m²)
100	163	263
200	192	392
300	240	540

RESULTS				
Undrained Cohesion Cu(kN/m²)	Angle of Int. Friction ø	Moisture Cont. %	Bulk Unit Weight (kN/m³)	Dry Unit Weight (kN/m³)
56	9	32.1	17.9	13.1