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Geotechnical Investigation of Sub-Soiland Rock Characteristics in Parts of Shiroro-Muya-Chanchaga Area of Niger State, Nigeria

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Abstract: This study aims at determining the litho-stratigraphic sequence to ascertain the engineering properties of the underlying soils and rocks in Shiroro-Muya-Chanchagaarea of Niger State, Nigeria and recommend appropriate foundation types for road construction, bridge and pavement design. Ten boring points/rock coring to 10m below existing ground level were made, out of which six along the piers/abutment alignment and two confirmatory borings each at the abutments. Four (4) boring points were located in water (BH-1, BH-3, BH-4 and BH-5) and two on land (BH-2 and BH-6).The bearing capacities for pile diameters of 406mm, 600mm, 750mm and 1000mmwere computed. Generally, the rocks encountered at the top layer were predominantly weathered and highly fractured with discontinuity in the rock formation due to fractures. However, rocks encountered at lower depths were granitic formation with appropriate recovery lengths. The depths of overburden averaged 4.50m on water and 0.5m on land. The rocks encountered at the near surface werefragmented due to the excavation activities that have taken place while the soils encountered during burrow pitting were hydromorphic soils with clays of moderate saturation. It is recommended that reinforced concrete bored piles be adopted to support steel and for road pavement. It is also suggested that the sub-grades should have average thickness of 150mm with a minimum practical value for spreading and compaction with the load carrying capacity verified by conducting pile load test.

Keywords: *Soils, Foundation, Pavement Design, Rock Formation, Bida Basin.*

Introduction:

Geotechnical engineering practice concerns the application of civil engineering technology to some aspects of the earth. Usually, the geotechnical engineer is concerned only with natural materials (soils and rocks) found at or near the surface of the earth. Recently in Nigeria, building collapse has been on the increase, partly because of inadequate soil investigations to determine the engineering properties of the sub-soils, load bearing capacities, settlement potentials under loads and the soil/water interfaces present. Other causative factors attributable to building collapse include but not limited only to the following: soil conditions, imposed load of the structure more than the bearing (load carrying) capacity of the soils upon which the buildings are erected as well as environmental factors.

The investigation and determination of subsurface engineering foundation structures in Nigeria is not usually fully explored. This has gone a long way in contributing to failures and high cost of operations in such projects (Amadiet al., 2012). The importance of geological structures in foundation investigation for engineering structures like roads, bridges, dams and buildings cannot be underestimated (Annoret al., 1987).

The knowledge of the geotechnical characteristics of parts of Bida Basin, North Central Nigeria is very desirable for design and construction of foundation of future civil engineering structures in order to minimize adverse effects and prevention of post construction problems. Some studies have been carried out on geotechnical properties of the subsoil in parts of Minna (Oke&Amadi, 2008; Okeet al.,2009).
This paper therefore focuses on the determination of the underlying soil/rock stratigraphy and evaluation of some relevant geotechnical characteristics of the subsoil in order to ascertain the subsoil condition and recommend appropriate foundations to support bridge and design for road pavements.

Location and Geology of Study Area:

The area of investigation, Zumba River, BarkinKwogi around Shiroro-Muya-Chanchaga area of Niger State, Central Nigeria (Fig.1) lies within latitude $9^0$30' to $10^0$00'N and longitude $6^0$00' to $6^0$58'E. The Zumba River is an artificial channel created as water passage to regulate the volume of water in theShiroro dam.The climate of the area is similar to what is obtainable in the central Nigeria, with an annual average rainfall of

1250mm with seven months of rainy season and five months of dry season (Idris-Nda, 2005).

Geologically, the area of investigation is an integral part of the Bida Basin, also known as the Mid-Niger Basin or Nupe Basin. It is a NW-SE trending intra-cratonic structure extending from Kontogora. However, Niger State is covered by two major rock formations, namely the sedimentary and basement complex. The sedimentaryrocks are characterized by sandstones and alluvial deposits, particularly along the Niger valley and in most part of the southern region of the state. The northern part is the basement complex characterized by granitic outcrops which dominates the landscapes in Rati, Shiroro and Minna.The basement complex is composed of three lithological units – migmatite gneiss complex, low grade schist belts and the older granite (Truswell& Cope, 1963; Ajibade, 1972; Ajibade&Woakes, 1976). The sedimentological characteristics of the area are lithofacies of granitic outcrops, mining pits and abandoned quarry sites.

The Schist belts in this area occur as two elongated bodies separated by the older granite suite. The tips of the two Formations are separated by a 40km expanse of the older granite suite. However, this study indicates a much smaller separation of less than 10km. the Birnin-Gwari Formation lies to the West of the older granite (the Minna Batholith) while the Kushaka Formation lies to the East. A gravity survey model conducted over the area (Udensi et al., 1986; Udensi&Osazuwa, 2004) showed that the two formations have a maximum thickness of 11 and 6km, respectively.

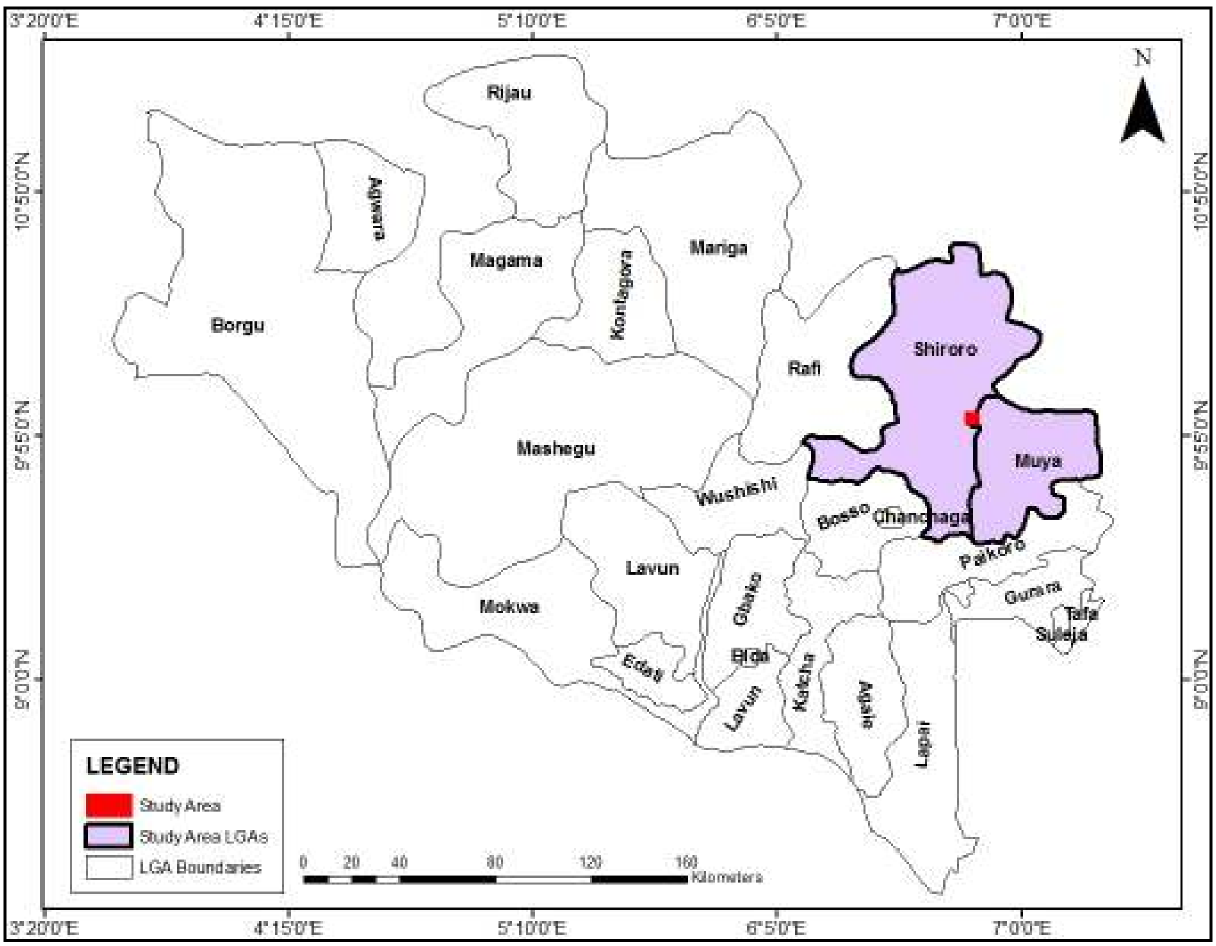

Figure 1: *Map of Niger State Showing the Study Area*

Methodology:

Rock Coring:

Rock coring was carried out with a Nenzi Corer attached with a Diamond bit or alloyed bit and other coring accessories. The procedure adopted was opening of the ground with a 9'' clay cutter which advances the hole at a speed of 3500rph until a hard rock is encountered. The clay cutter is then replaced by an alloyed bit or a diamond bit which will core the rock. Additional extensions of 3m drilling pipes are attached as depth increases. Cored rocks are retained in the core barrel with the help of a catcher inserted in the reaming shell of the barrel to prevent cored rock from falling into the open hole. During the coring exercise, water is used extensively to prevent friction between the rock and the drilling bit and where there is collapse of the hole especially where the overburden is encountered, bentonite slurry is used. Representative cored rocks are packaged in core box and properly labelled. The cored rocks are normally not captured at 1.0m length at the near surface due to discontinuity, suggesting that the natural formation of the area has been tampered with due to mining and excavation activities.

Burrow Pitting:

The burrow pits were excavated with hand auger. The procedure adopted was opening of the ground with the auger by rotating in clockwise direction the T-handle of the auger extension. Representative disturbed soil samples were taken at regular intervals. The soil samples were used for a detailed and systematic description of the soil formation in each stratum in terms of its visual properties and for laboratory analysis.

Nine (9) burrow pits were excavated for soil sampling, four (4) burrow pits were initially executed; two (2) each at both end of the river. Additional five (5) were executed at designated distances from ABT-2. The first burrow pit was executed at 0.00m from ABT-2. Second, third, fourth and fifth burrow pits were all executed at 100.0m distances from each other.

Laboratory Tests:

Detailed laboratory tests were carried out on the represented rock samples to determine their Uniaxial Compressive Strength (UCS) and Point Load [ls]. The tests were carried out in accordance with ASTM 2938 – Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens and International Society for Rock Mechanics (ISRM). For the soil samples, shear strength, stability and classification tests were carried out in accordance with B.S 1377 (1995) – Methods of Tests for Soil for Civil Engineering Purposes.

Uniaxial Compressive Strength:

The uniaxial compressive strength of rock was determined to ascertain the strength classification and characterization of rocks. Tests specimens are cut as carefully as possible to right cylinders. The cuts are parallel and at right angles to the longitudinal axis. The length-to-diameter ratio (L/D) is determined and the mass of specimen taken to the nearest 0.001g. The specimen is placed on the lower platen of the compression machine and the axial load is applied continuously and without shock until the load becomes constant, reduces or pre-determined amount of strain is achieved. Maximum load on the specimen is recorded in Kilo- Newton and the uniaxial compressive strength is calculated by dividing the maximum load by the cross-sectional area of the rock specimen.

Point Load Test:

Point Load Tests (PLT) on rock is an index test also for rock strength classification. PLT method is based upon breaking off a cylindrical rock specimen. However, Broch& Franklin (1972) developed a conceptual model for the derivation of PLT value from uniaxial compression tests.

$\pi P/4A$

Where P = Maximum load from uniaxial compression test

A = Cross-sectional area of the rock specimen

Moisture Content:

The water content was determined by drying selected moist/wet soil material for at least 12 hours to a constant mass in 110^{o}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%) (BS 1377: Part 2: 1990).

Atterberg Limits:

Atterberg limits were determined on soil specimens with a particle size of less than 0.425 mm. The Atterberg limits refers to arbitrary defined boundaries between the liquid limit and plastic states (Liquid limit, W_L), 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 testwas carried out. Distilled 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 (W_L – W_P) (BS 1377: Part 2: 1990).

Particle Size Analyses:

Particle size analyses were performed by means of sieving. Sieving was carried out for particles that would be retained on a 0.075 mm sieve 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 (BS 1377; Part 2: 1990).

Compaction:

Compaction is the densification of the soil by removal of air, which requires mechanical energy. The degree of compaction of a soil is measured in terms of its dry unit weight. A standard mould is filled with soil in three layers. Each layer is compacted by 25 blows of standard hammer, falling through a height of approximately 12 inch. Water is added during compaction as softening agent on the soil particles. The soil particles slip over each other and move into a densely packed position. The dry unit weight is calculated thus: $\gamma_d = \gamma_t/1+w$.

California Bearing Ratio (CBR):

This is an empirical test for estimating the bearing values of highway sub-bases and sub-grades. During the test, the plunger is seated into the soil using a force of 50N at a constant rate of 1 mm/min and the forces recorded at intervals of 0.25mm. The results are then compared to a standard curve for a value of 100% CBR. The CBR is a simple of the corresponding values and where a difference between the value at 25 mm and 5.0mm occurs, the higher value is taken.

Safe Bearing Capacity:

The net bearing is the maximum net intensity of loading the rock/soil can safely support without the risk of shear failure. Consideration of the net safe bearing capacity is determined using Meyerhof's (1953) Net Bearing Capacity of rock:

$q_{ns} = q_u.k_{sp}.R_w$

Where q_{ns} = Net safe bearing capacity

q_u = Uniaxial compressive strength of rock in KN/m^2

k_{sp}= Emperical factor including a factor of safety as 3 and closely spaced joints (0.25)

R_w= Correction for water table (0.50)

Pile Load Capacity:

Pile load capacity was determined by:

$Q_{ult} = Q_{tip} + Q_{side}$

Where Q_{ult} = Ultimate bearing capacity of stratum (q_{ns}* Fs)

$Q_{tip} = \pi/4 * D^2 * q_{ult}$

$Q_{side} = \pi D * D_f * qfriction$

qfriction = Shaft friction for pile equal to 10

D_f =Depth of pile

D=Diameter of pile

Table 1: *Engineering Properties of Soil and Rocks in the Study Area*

BH No.	Density (g/m^3)	Load (kN)	UCS (N/mm^2)	Point Load Test (MPa)
ABT-1	3.17	58.15	29.62	2.33
ABT-2	3.30	62.60	31.98	2.50
ABT-3	3.14	61.97	31.57	2.48
ABT-4	3.05	83.23	42.20	3.33
ABT-5	2.86	82.13	41.84	3.29
ABT-6	3.04	88.74	45.61	3.55

Note: *UCS = Uniaxial Compressive Strength*

Table 2: *Engineering Properties of the Soil*

Test Parameters	Average
Moisture Content (%)	6.0
Liquid Limit (%)	37.6
Plastic Limit (%)	21.1
Plasticity Index (%)	16.4
Optimum Moisture Content (%)	13.3
Maximum Dry Density (mg/m^2)	1.86
CBR (Un-soaked) %	46.0
CBR (Soaked) %	20.3

Results and Discussion:

Tables 1 & 2 Shows The Geotechnical Characteristics And Engineering Properties Of Sub-Soilsand Rocks In The Study Area.

Litho-Stratigraphic Sequences of the Area:

(i) Abutment-1 (BH-1):

Abutment -1 is located in the river. The riverbed is characterized by over-burden of highly weathered rock mixed with silty sand (hydromorphic sand) of average thickness of 5.0m with rock fragments at 4.90m. From 7.0m to the terminal depth at 10.0m is fine grained, dark grey granitic rock with Rock Quality Designation (RQD) ranging from 24% to 68%.

(ii) Abutment-2 (BH-2):

Abutment-2 is located on land. The underlying litho-stratigraphic sequence consists of a layer of grayish brown, pebbly silty sand with mica of 0.50m thickness. It is underlain to 1.0m by light coloured, coarse-grained slightly weathered fractured rock with average Rock Quality Designation (RQD) of 42%. From 1.0m to 2.0m is a rock similar to rock quality described above. However, the RQD is 80%. Underlying the above formation from 2.23m to the terminal depth at 10.0m is medium grained, light coloured, fresh granitic rock with RQD ranging from 83% to 98%.

(iii) Borehole-3 (Pier 1):

Borehole -3 is located in the river and the average water column to the river-bed at the time of coring was 4.80m. The river bed is characterized by grey weathered material of silty sand (predominantly hydromorphic sands) of 3.0m thickness. From 3.0m to 7.0m is coarse grained, light coloured, slightly weathered fractured rock with RQD of 48%. From 7.0m to the terminal depth at 10m is fined grained, light coloured granitic rock with RQD ranging from 52% to 89%.

(iv) Borehole-4 (Pier 2):

This pier is located in the middle of the water, with an average water depth to the river bed 6.0m. The river bed is characterized to 3.0m by light coloured, slightly weathered, granitic rock with presence of rock fragments at the river bed. Underlying the above rock formation to the terminal depth is coarse grained, grey coloured granitic rock with quartz. The average RQD ranges from 34% to 74%.

(v) Borehole-5 (Pier 3):

Borehole -5 (Pier 3) is also located in the water. The average water depth to the river bed is 5.0m. From the river bed to about 2.80m is a layer of light coloured, slightly weathered, granitic rock with rock fragments at the river bed. From the 3.0m to 5.0m is coarse grained, light coloured, granitic rock with quartz and RQD of 33%. From 6.0m to the terminal depth of 10.0m is light coloured granitic rock with RQD ranging from 41% to 67%.

(vi) Borehole-6 (Pier 4):

Borehole-6 is located close to the bank of the river. The borehole is characterized at the top by a sand mass of grayish brown sand with average thickness of 0.60m. The above layer is further underlain by a bolder of 0.60m thickness. From 1.20m to 3.50m is a layer of grayish brown gravelly sand. From 3.50m to 5.50m is fine grained, light coloured granitic rock with sands trapped between the fractures of the rock. From 5.50m to the terminal depth is fine grained, light coloured granitic rock with RQD ranging from 76% to 94%.

(vii) Confirmatory Tests:

The confirmatory tests were carried out to ascertain the conditions of the underlying rock formations at the abutments. Four (4) confirmatory borings were conducted. Two (2) at each abutments 1 and 2. The confirmatory borings were designated as R1 and L1 for ABT-1 and R2 and L2 for ABT-2. At R1 and L1 (ABT-1 confirmatory test points), the average water column to the river bed is 4.0m. From the river bed to 4.0m is a layer of grey weathered rock material mixed with silty hydromorphic sands. From 4.0m to 7.0m isfine grained, dark grey, slightly weathered soft granitic rock and from 8.0 to the terminal depth is fine grained, light coloured granitic rock and RQD ranges from 44% to 63%.

At ABT-2, the confirmatory tests revealed that the topsoil is characterized by grayish brown silty sand with tiny gravels and mica to the depth of 0.80m in R2 and 0.50m in L2. Underlying the top soil to 1.0m is coarse grained, light coloured, slightly weathered fractured granitic rock in both holes. However, sand sediments are present in L2. In R2, from 1.0m to 2.50mis coarse grained, light coloured, slightly weathered fractured rock with observable movements along the points and at L2, the fractured rock extends to 2.80m. Underlying the fractured rock formations to 6.0m is medium grained, light coloured granitic rock with RQD ranging from 22% to 47%.

Generally, the rocks at the top are predominantly weathered and highly fractured, with discontinuity in the rock formation due to the fractures. However, rocks encountered at lower depths are granitic formation with appreciable recovery lengths.Tables 3, 4, 5, 6, 7 and 8 shows pile load capacities. Table 11 shows the Uniaxial Compressive Strength and Point Load Test results

Road Pavement Design:

The need for pavement design is to distribute the applied loadings from the road to the underlying sub-grade without causing distress to the foundations or the overlying sub-grades. The soils encountered along the road alignment are predominantly hydromorphic soils with clay content of moderate base saturation. The performance of the sub-grade is subject to the moisture content. Moisture content is normally variable as water can come from many sources as rainfall, capillary action, seasonal movement of water table and ingress.

The CBR values for un-soaked and soaked are 46% and 20.3%, respectively (Tables 9 & 10). Based on the CBR values, it is suggested that the sub-base should have standard thickness of 150mm which is the minimum practical for spreading and compaction.

Table 3: *Pile Load Capacity for ABT-1*

Depth of BH(m)	Pile Diameter							
	750mm		1000mm		750mm		1000mm	
	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)
5	1502	501	3235	1078	5026	1675	8882	1961
6	1515	505	3254	1085	5049	1683	8913	2971
7	1527	509	3273	1091	5073	1691	8945	2982
8	1540	513	3292	1097	5096	1699	8976	2992
9	1553	518	3311	1104	5120	1707	9008	3003
10	1566	522	3330	1110	5143	1714	9039	3013

Table 4: *Pile Load Capacity for ABT-2*

Depth of BH(m)	Pile Diameter							
	406mm		600mm		750mm		1000mm	
	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)
5	1617	539	3486	1162	5417	1806	9577	3192
6	1629	543	3504	1168	5440	1813	9609	3203
7	1642	547	3523	1174	5464	1921	9640	3213
8	1655	552	3542	1181	5487	1829	9671	3224
9	1668	556	3561	1187	5511	1837	9703	3234
10	1680	560	3580	1193	5534	1845	9703	3245

Table 5: *Pile Load Capacity for BH-3*

Depth of BH(m)	Pile Diameter							
	406mm		600mm		750mm		1000mm	
	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)
5	1597	532	3442	1147	5339	1783	9456	3152
6	1609	536	3461	1154	5372	1791	9488	3163
7	1622	541	3499	1160	5396	1799	9519	3173
8	1635	545	3499	1166	5419	1806	9551	3184
9	1648	549	3157	1172	5443	1814	9582	3194
10	1660	553	3536	1179	5467	1822	9614	3205

Table 6: *Pile Load Capacity for BH-4*

Depth of BH(m)	Pile Diameter							
	406mm		600mm		750mm		1000mm	
	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)
5	2122	708	4590	1530	7110	2370	12588	4196
6	2135	712	4609	1536	7134	2378	12619	4206
7	2148	716	4628	1543	7157	2386	12650	4217
8	2161	720	4647	1549	7181	2394	12681	4227
9	2174	725	4666	1555	7204	2401	12731	4238
10	2186	729	4685	1562	7228	2409	12745	4248

Table 7: *Pile Load Capacity for BH-5*

Depth of BH(m)	Pile Diameter							
	406mm		600mm		750mm		1000mm	
	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)
5	2095	698	4531	1510	7050	2350	12482	4161
6	2108	703	4550	1517	7074	2358	12513	4171
7	2121	707	4569	1523	7099	2366	12544	4181
8	2134	711	4588	1529	7121	2374	12574	4192
9	2146	715	4006	1536	7145	2382	12607	4202
10	2159	720	4625	1542	7168	2389	12639	4213

Table 8: *Pile Load Capacity for BH-6*

Depth of BH(m)	Pile Diameter							
	406mm		600mm		750mm		1000mm	
	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)	Ultimate (KN)	Allowable (KN)
5	2259	753	4888	1629	7675	2555	13592	4531
6	2272	757	4907	1636	7699	2566	13624	4541
7	2284	761	4926	1642	7722	2574	13655	4552
8	2297	766	4945	1648	7746	2582	13686	4562
9	2310	770	4964	1655	7769	2590	13718	4573
10	2323	774	4983	1661	7793	2598	12749	4583

Table 9: *Laboratory Test Results*

Borehole No.	**Moisture Content** %	**Liquid limit** %	**Plastic limit** %	**Plasticity Index** %	**Proctor Compaction** OMC %	MDD Mg/m^2	**California Bearing Ratio (CBR)** Un-soaked %	Soaked %
TP-1	5	36	25	11	12	1.92	65	35
TP-2	5	36	25	11				
TP-3	5	36	25	11				
TP-4	5	36	25	11				

Table 10: *Summary of Laboratory Test Results*

Borehole No %	**Distance** m	**Moisture Content**	**Liquid limit** %	**Plastic limit** %	**Plasticity Index** %	**Proctor Compaction** OMC %	MDD Mg/m^2	**California Bearing Ratio (CBR)** Un-soaked %	Soaked %
TP-1	0.00m from ABT-2	6.1	38	12	26	13.3	1.88	46	18
TP-2	100.0m from ABT-2	6.1	38	12	26				
TP-3	200.0m from ABT-2	6.1	38	12	26				
TP-4	300.0m from ABT-2	8.0	40	27	13	14.5	1.78	28	8
	400.0m from ABT-2	8.0	40	27	13				

Table 11: *Uniaxial Compressive Strength and Point Load Test Results*

BH No	Depth m	L/D Ratio	Mass of Rock G	Area (πD^2) cm^2	Volume cm^3	Density g/cm^3	Load kN	Comp. Strength N/mm^2	Point Load Test Mpa
ABT-1	7.00	2	580	19.63	196.30	2.95	47.30	24.10	1.89
	8.00	2	618	19.63	196.30	3.15	57.80	29.44	2.31
	9.00	2	652	19.63	196.30	3.32	82.00	41.77	3.28
	10.00	2	642	19.63	196.30	3.27	45.50	23.18	1.82
ABT-2	3.00	2	688	19.63	196.30	3.50	58.70	29.90	2.35
	6.00	2	670	19.63	196.30	3.41	51.30	26.13	2.05
	7.00	2	612	19.63	196.30	3.12	55.50	28.27	2.22
	10.00	2	620	19.63	196.30	3.16	84.90	43.25	3.40
BH-3	7.00	2	596	19.63	196.30	3.04	69.00	35.15	2.76
	9.00	2	592	19.63	196.30	3.02	64.90	33.06	2.60
	10.00	2	660	19.63	196.30	3.36	52.00	26.49	2.08
BH-4	5.00	2	616	19.63	196.30	3.14	66.90	34.08	2.68
	7.00	2	590	19.63	196.30	3.01	107.20	54.61	4.29
	10.00	2	590	19.63	196.30	3.01	75.60	38.51	3.03
BH-5	6.00	2	582	19.63	196.30	2.96	79.20	40.35	3.17
	8.00	2	546	19.63	196.30	2.78	73.40	37.39	2.94
	10.00	2	556	19.63	196.30	2.83	94.80	47.78	3.75
BH-6	6.00	2	586	19.63	196.30	2.99	129.91	66.18	5.20
	9.00	2	592	19.63	196.30	3.02	41.51	21.15	1.66
	10.00	2	612	19.63	196.30	3.12	94.8	48.29	3.79

Conclusion:

This study revealed that the rocks encountered at the top layer were predominantly weathered and highly fractured. There was discontinuity in the rock formation which was due to fractures. However, rocks encountered at lower depths are granitic formation with appropriate recovery lengths. The depth of overburden averaged 4.50m on water and 0.5m on land. Results also show that rocks encountered at the near surface werefragmented due to the excavation activities that have taken place in the area. The soils encountered during burrow pitting were hydromorphic soils with clays of moderate saturation. Following the results obtained, it is recommended that reinforced concrete bored piles be adopted to support steel and for road pavement. It is also suggested that the sub-grades should have average thickness of 150mm which is a minimum practical value for spreading and compaction. More importantly, the load carrying capacity should be verified by conducting pile load test. Generally, this study has shown that the knowledge of the geotechnical characteristics of the area as obtained from field study, excavation of trial pit, and laboratory analysis of recovered soil and rock samples have provided valuable data that can be used for designing road pavement and other forms of construction for foundation of civil engineering structures in order to minimize adverse effects and prevention of post construction problems.

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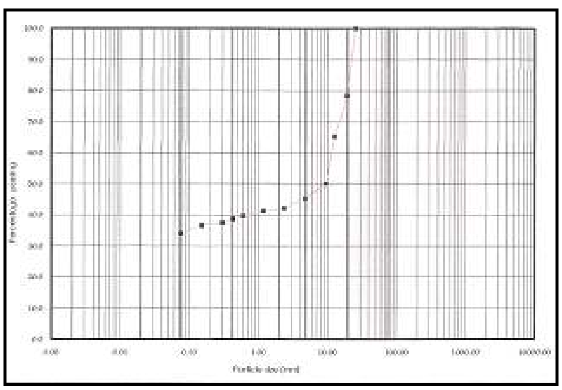

Figure 1: *Particle Size Distribution Curve for BH/PIT-1*

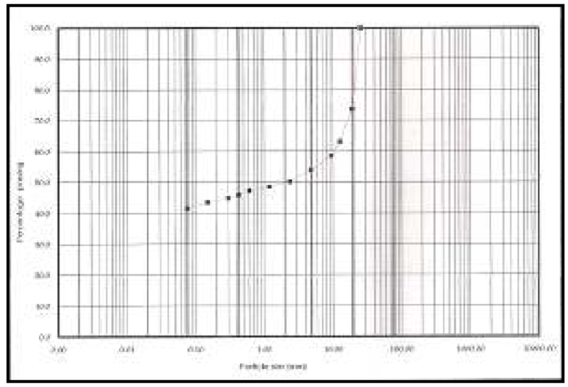

Figure 2: *Particle Size Distribution Curve for BH/PIT-2*

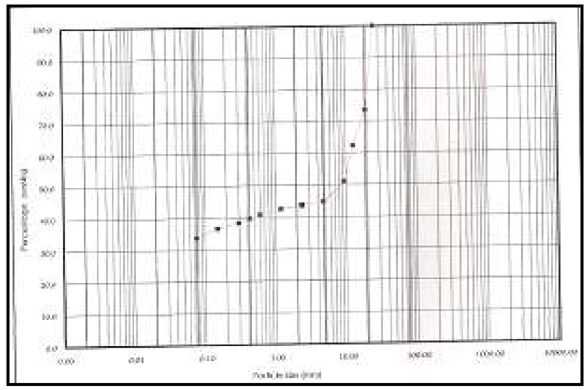

Figure 3: *Particle Size Distribution for BH/PIT-3*

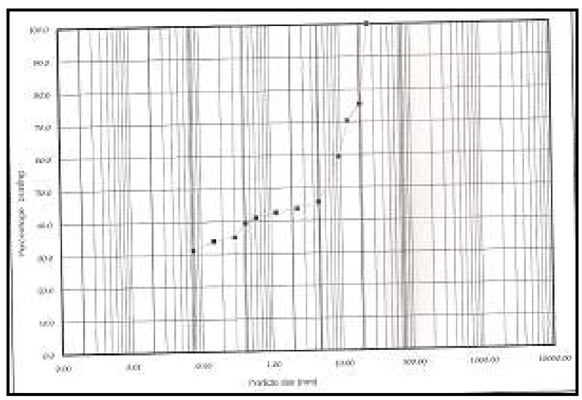

Figure 4: *Particle Size Distribution Curve for BH/PIT-4*

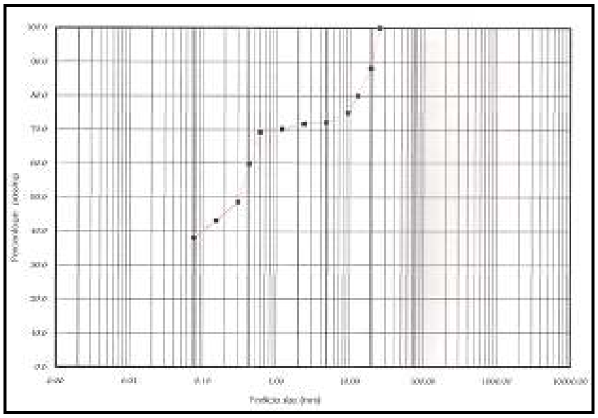

Figure 5: *Particle Size Distribution Curve for Trial Pit -1*

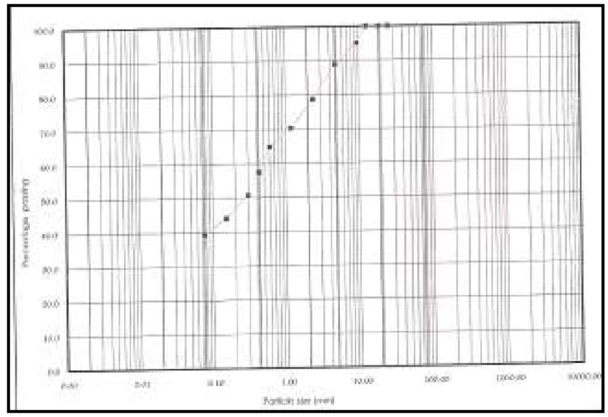

Figure 7: *Particle Size Distribution Curve for Trial Pit-3*

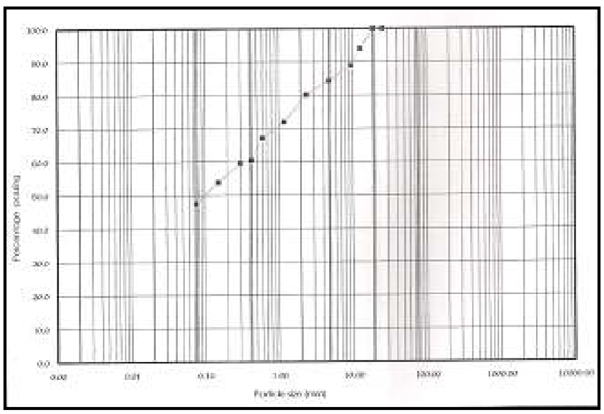

Figure 6: *Particle Size Distribution Curve for Trial Pit-2*

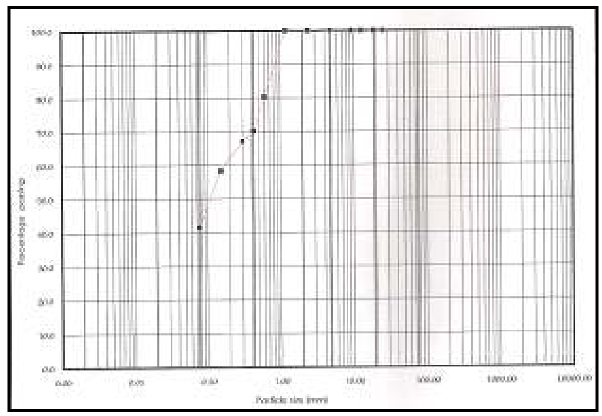

Figure 8: *Particle Size Distribution Curve for Trial Pit-4*

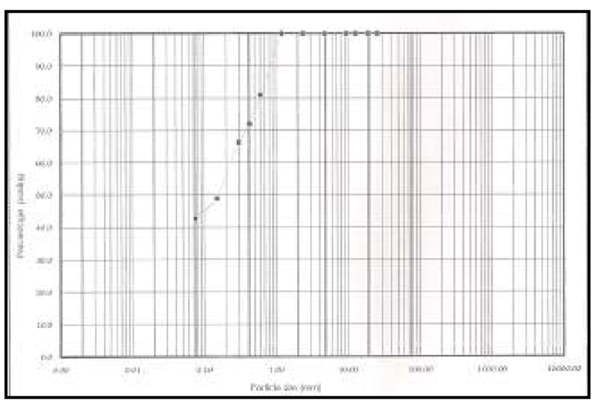

Figure 9: *Particle size Distribution Curve for Trial Pit-5*