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Geotechnical investigations for infrastructural development: A case study of Daki Biyu District, Federal Capital Territory, Abuja, Central Nigeria

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The geotechnical properties of Daki Biyu district in the Federal Capital Territory, Abuja, Nigeria was investigated to ascertain the suitability of the sub-surface soil to support massive infrastructure such as high rise building and industries that might be carried out in the future. The particle size distribution shows that the soil is predominantly sandy-clay to sandy-gravels. The plasticity indices suggest low to medium compressibility while the co-efficient of volume change (Mv) and the coefficient of consolidation (Cv) are generally low for most of the pressure ranges. The average allowable bearing capacity values of 150 kN/m^2 -240 kN/m^2 and an average of 460 kN/m^2 -700 kN/m^2 for the ultimate bearing capacity agrees with the national building code (1983) for safe bearing capacity for cohesionless soils. The sub-surface soils were found to possess good geotechnical properties that are capable of supporting infrastructural development.

Key words: Geotechnical investigation, bearing capacity, compressibility, consolidation, borehole log.

INTRODUCTION

Sequel to the relocation of the Federal Capital Territory (FCT) from Lagos to Abuja on the 12th day of December, 1991, the city has experienced rapid population growth (Dikedi, 2012). This has resulted to the need for land for infrastructural developments such as high and low rise buildings for offices and residential purposes, water supply pipelines to all parts of city, sewage treatment and purification plants as well as construction of fly-over bridges etc. This has posed serious concern to engineers, engineering geologists and planners alike. A good understanding of the geology and geotechnical characteristics of the rocks and soils in the Capital Territory is essential for planning and construction of all engineering structures because of the heterogeneous nature of rocks and soils which these structures are constructed on. The importance of geotechnical investigation as well as engineering geology has lately been emphasized and encouraged due to reported cases of structural failure and collapse especially in Lagos, Abuja and other parts of the country (Ejembi, 2016). Geotechnical site investigation involves among others the need to assess the general suitability, safety and

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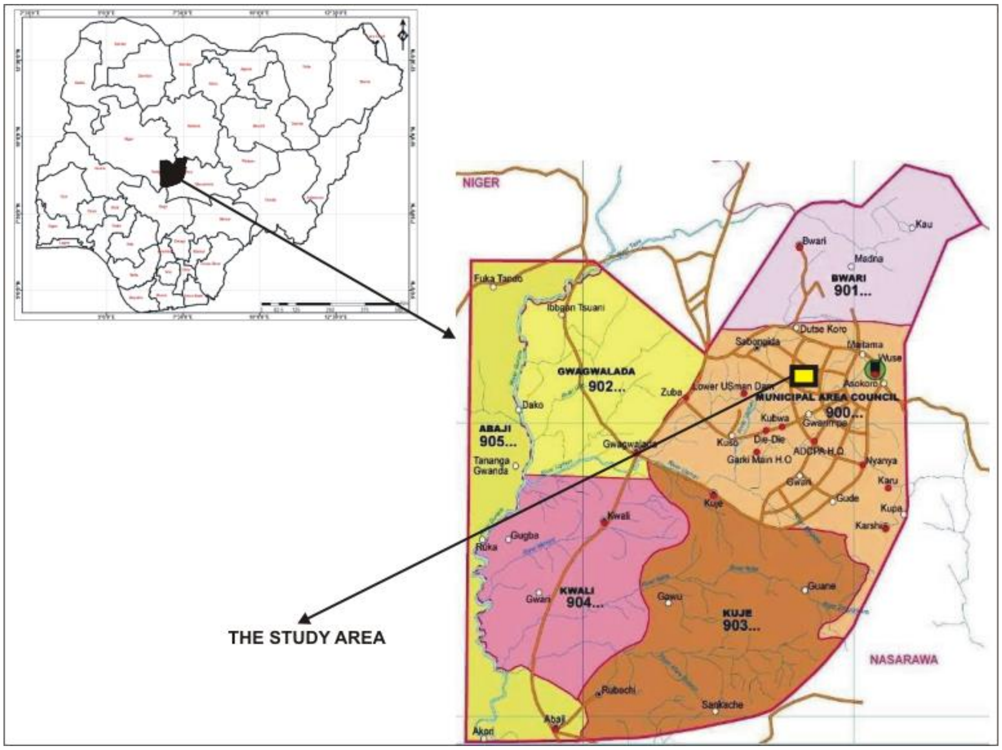

Figure 1. Location of the study area.

economical design of foundations and temporary works, knowing the nature of each stratum and engineering properties of the soil and rock as well as foresee and provide solutions against difficulties that may arise during construction due to ground conditions (Clayton et al*.,* 1996). The complexity of a site investigation depends upon the nature of the ground conditions and the type of engineering structure (Bell, 2011). Accordingly, a site investigation should attempt to foresee and provide against difficulties that may arise during construction because of ground and/or local conditions. Such investigations should reveal the strata that would be significantly affected by the structural load, the ground water condition, the degree and extent of weathering, the structural orientation of the rock masses is equally important. Also worthy of note and investigation are areas that were previously used for mine workings as they may pose as potential areas of subsidence (National Building Code, 1983). This information when carefully recorded and interpreted with accompanying geological sections and where possible a 3D picture of the subsurface strata can be of great assistance in the design and subsequent construction of the super structure.

It is in this regard that the sub-surface geotechnical investigation of Daki Biyu and environs in the Federal Capital Territory, Abuja was conducted with the aim of carrying out particle size distribution analyses for engineering soil classifications as well as the determination of the bearing capacity and settlement rates necessary for foundation design of structures. The data generated and analysed will help in ascertaining the suitability of the site for future infrastructural development.

Location and geology of the study area

The study area is located within the Federal Capital Territory (FCT), Abuja and lies between Latitudes 9.058°N and 9.035°N; and Longitudes 7.412°E and 7.434°E (Figure 1). It covers an area of 6.15 km^2 and is defined by two renowned rock formations; the Zuma Rock and Aso Rock. The lowest elevation is found in the extreme south western part of the city where the flood plain of the River Gurara is at an elevation of about 10 m above sea level. The land rises irregularly eastward, northward and north westwards. The highest part of the territory is in the northeast where there are many peaks over 76 m above sea level. In some other areas within the territory, there are many rounded and isolated hills called inselberg. The Gwagwa plains lie between the major hills and are very extensive. The surface geology is composed of predominantly high grade metamorphic and igneous rocks of Precambrian age which trend in NNE-SSW direction. These rocks consist of gneiss, migmatites and granites. Schist belt outcrops exist along the east margin of the area. This belt broadens southwards and attains a maximum development to the south eastern

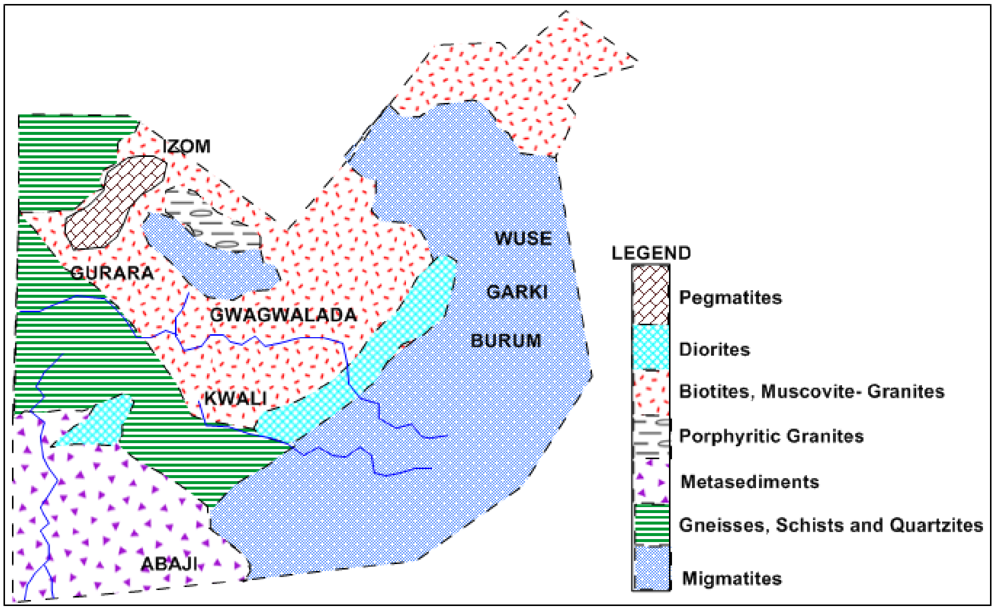

Figure 2. Geology map of Abuja.

sector where the topography is rugged and the relief is high. In general, the rocks are sheared (Kogbe et al*.,* 1983). The rocks can be divided into the following major groups: Migmatite Complex (migmatite, migmatite gneiss, granite gneiss, porphyroblastic granite gneiss, leucocratic granite gneiss), Metamorphosed Supracrustal rocks (mica schist, marble, amphibolites and amphibole schist, fine-medium grained gneiss); and minor intrusions such as rhyolites, quartz-feldspar porphyry, dolerites and basalts. Others include quartzite, pegmatite and quartz vein (Rahaman, 1988) (Figure 2).

METHODOLOGY

This involves the use of two major equipments for the collection of samples; a number 4 2.5 ton Dutch cone penetrometer which was employed for in-situ bearing capacity determination and a number 4 Percussion Dando 150 motorized shell as well as Auger rigs which employ light cable percussion drilling techniques (Johnson and Degraff, 1988). The rigs were used for the collection of disturbed and undisturbed soil samples; and the execution of Standard Penetration Test. A total of sixty penetrometer tests and thirty boreholes drills were executed. The maximum depths of most of the boreholes were determined at depth to basement though at some instances hard pans were encountered and stopped. The resistance to penetration was recorded at every 0.25 m for each penetrometer test point. Some basic sampling and testing procedures employed in the research also included; collection of disturbed samples at each change of strata and execution of standard penetration test in cohesionless and mixed soils at intervals of 1.50 m with disturbed samples recovered from SPT spoon (Sanglerat, 1972). A split spoon, 50 mm was driven into the soil by a mass of 65 kg hammer dropping from a height of 760 mm. The resistance to penetration was then expressed as the number of blows (N-blows) required to penetrating a distance of 0f 300mm below an initial thickness of 150 mm. Collection of undisturbed 100 mm diameter (U_4) samples in cohesive strata and noting depths at which ground water was struck. All samples retrieved from boreholes were carefully logged while representative disturbed samples were subjected to classification test and undisturbed samples were subjected to Oedometer consolidation test which is of immense importance for estimation of settlement. Hence, results obtained were plotted and examined in CorelDraw®, Surfer® and ArcGIS® softwares.

Hydrogeology of the study area

Substantial settlement may occur when water table rises up to or beyond the footing level of shallow foundations (Shahriar et al., 2013). This settlement can threaten the integrity of a structure and may subsequently lead to the collapse of the entire or part of the structure. The hydro stratigraphic units consist of the top soil which is mostly made up of dark grey sandy- silty-clay which ranges in depth from 0 - 0.25m. This is mostly followed by reddish- brown firm sandy-silty-clay with an average depth of 1.30m from the surface. Above this layer is an intercalation of soft sand and clay which ranges in depth from 1.50 - 4.50m. The aquifer consists of reddish-brown hard sandy-silty-clay followed by dark grey hard weathered rock whose thickness ranges from 5.50 - 9.0m. The groundwater levels ranges between 4.50 - 5.25m below the ground surface in almost all the boreholes of the study area and these are however subject to seasonal fluctuations.

RESULTS PRESENTATION

Figure 3 presents the sampled points from the study area while Table 1 shows the particle size distribution and plasticity characteristics of some tested soils. Some of the tested soils were analysed graphically and categorized into various grain sizes (Figures 4 and 5). Again, some of the samples were henceforth tested for

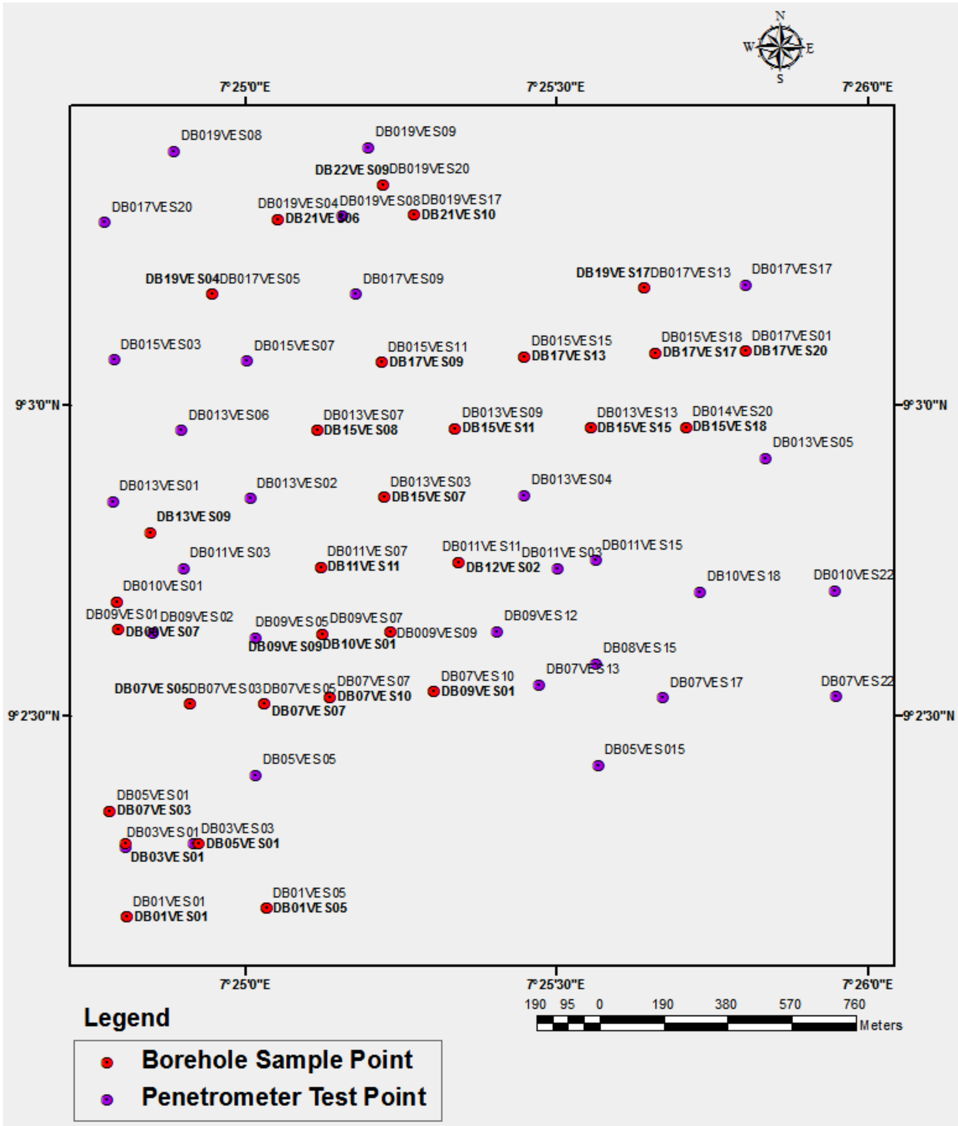

Figure 3. Borehole and penetrometer test sample points of the study area.

their liquid limit competencies. Table 2 shows the consolidation parameters obtained from the compressibility and consolidation analyses of soil samples within the area. Allowable bearing pressure and ultimate bearing capacity of different samples as well as their undisturbed graphical plots are presented in Table 3. The drill details are thus presented in Appendix A.

DISCUSSION

Grain size distribution analysis

The particle size distribution analysed from the disturbed soil samples obtained from boreholes differs significantly. The soil samples are mostly well graded ranging from clay, silt, sand and gravels and in some locations, a combination of two or more of the above named soils are observed. Dense sands and gravels are important foundation soils because they can carry weight in excess of 600 kN/m^2 with minimal settlement (Terzaghi et al., 1967). Therefore, areas with dense sands and gravels can accommodate structures whose weight exceeds 600 kN/m^2 whereas places with particularly loose sand, soft clays and silts should not be loaded above 150 kN/m^2, this to ensure stability and integrity of the structure.

Plasticity characteristics

Plasticity parameters also called Atterberg limits are important indices used alone or with other

Table 1. Summary of particle size distribution and plasticity characteristics of some tested soil samples from the study area.

Borehole no.	Sample no	Sieve analyses result (%)			Atterberg limit		Depth (m)	Casagrande chart classification	Soil description
		Fines	Sand	Gravel	PI	LL			
DB07VES05	03				25	60	1.00	High plasticity	
DB07VES05	06				29	48	3.00	Medium plasticity	
DB07VES07	03				26	44	1.50	Medium plasticity	
DB07VES10	03				16	29	1.00	Low plasticity	
DB07VES10	06				14	33	3.00	Medium plasticity	
DB07VES10	10				74	74	6.00	High plasticity	
DB09VES01	03				28	49	1.50	Medium plasticity	
DB09VES01	08				27	52	5.25	High plasticity	
DB09VES01	04				16	34	9.30	Medium plasticity	
DB11VES11	03				24	55	1.00	High plasticity	
DB11VES11	06				17	34	3.00	Medium plasticity	
DB15VES07	07				17	52	4.50	High plasticity	
DB19VES04	03				16	32	1.00	Medium plasticity	
DB21VES01	02				18	41	0.75	Medium plasticity	
DB21VES06	08	8.50	39.5	52.0			6.00		Sandy gravel
DB21VES06	09	2.5	46.00	51.5			6.75		Sandy gravel
DB21VES10	02	48.5	39.0	12.5			0.75		Sandy clay with some gravels
DB21VES10	03	49.5	44.5	6.0	19	36	1.50	Medium plasticity	Sandy clay
DB21VES10	05	19.38	38.5	42.0			3.00		Gravelly sandy clay
DB21VES10	07	49.5	48.5	2.0			4.50		Sandy clay
DB21VES10	09	20.5	59.0	20.5			6.00		Sandy clay with some gravels
DB22VES09	02	44.5	47.5	8.0			0.75		Sandy clay
DB22VES09	03	44.5	47.5	8.0			1.50		Sandy clay
DB22VES09	04	47.5	44.5	8.0			2.25		Sandy clay
DB22VES09	05	49.0	46.0	6.0	14	44	3.00	Medium plasticity	Sandy clay
DB22VES09	06	43.5	49.0	7.5	15	43	3.75	Medium plasticity	Sandy clay

parameters in characterizing the swelling potential as well as the shear strength of soils based on water content. The relationship between Plasticity Index and Swelling Potential of some northern Nigerian soils as summarized by Ola (1981) as seen in Table 4.

Test results show that samples are of low to moderate compressibility with about 2 samples indicating high compressibility and predominantly exhibiting negligible swelling potentials. The implication of this is that the foundation soil may not experience the twin problem of swelling and shrinking of foundation soils which usually lead to the heaving and settlement of structures.

Consolidation test

The main objective of consolidation test on soil samples is to obtain necessary information about the compressibility of soils for use in determining the magnitude and rate of settlement of structures, Terzaghi, et al. (1996).

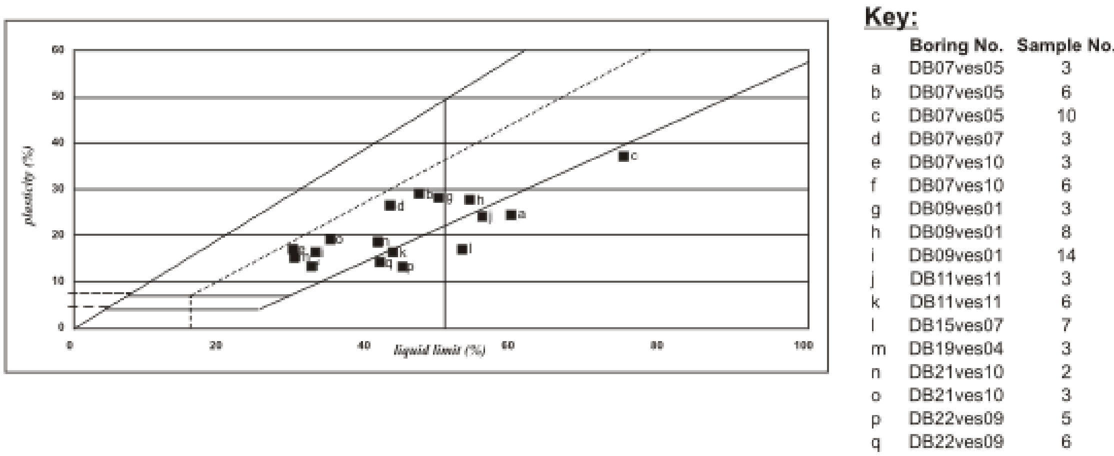

Figure 4. Casagrande plasticity chart plots for the study area.

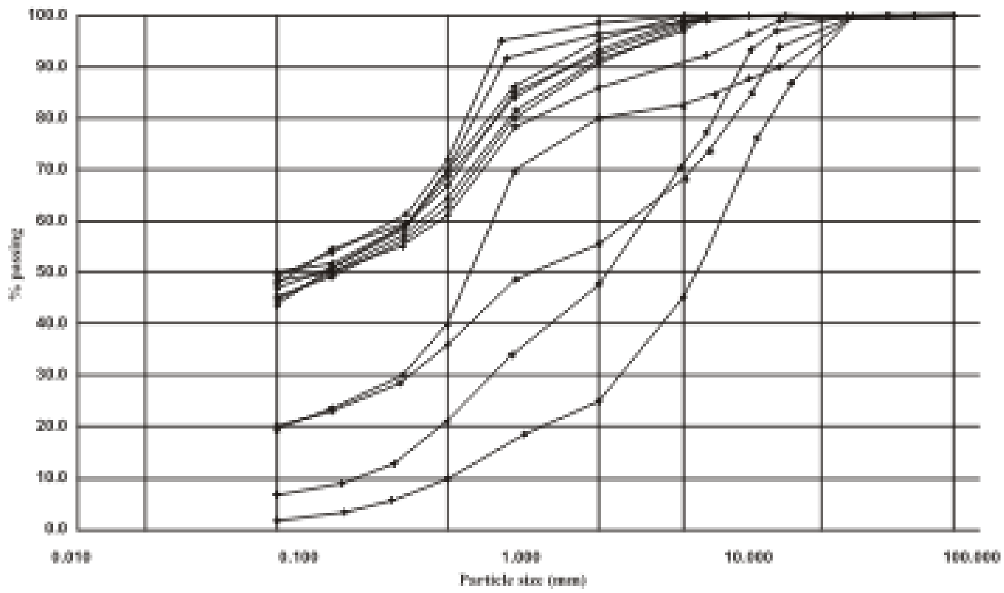

Figure 5. General particle size distribution curves for borehole samples.

Carter (1983) gave 0.05×10^{-3} kN/m^2 as the permissible limit of coefficient of volume change (Mv) for heavy overconsolidated clays, stiff weathered rocks and hard clays. Also, Terzaghi et al. (1967), investigated the coefficient of consolidation (Cv) of some geotechnical materials with granular soils including rock fills having a coefficient of consolidation of 0.02+ 0.01, shale and mudstone 0.03+ 0.01, inorganic clays and silt 0.04+0.01 and 0.05 +0.01 for organic clays and silts.

In view of the values of coefficient of volume change or compressibility (Mv) and coefficient of consolidation (Cv) as presented in Table 2 compared with the standard values given above, implies that the soil is typical of non-problematic lateritic soils. The low to medium value of Mv implies that any structure founded on the soils will not suffer excessive settlement exceeding the maximum permissible limit. Most importantly is the low values of Mv at a constant depth under a given pressure range.

Borehole logs and standard penetration test

The significance of geotechnical investigation specifically for borehole logging and standard penetration test is to know the stratigraphy and strength characteristics respectively of each layer. Most of the borehole logs in the study area show dark grey hard to loose sandy-clay at the upper most layer and occasionally hard lateritic

Table 2. Consolidation parameters of coefficient of volume (Mv) compressibilty and coefficent of consolidation (Cv) of some soil samples.

Sample No.	Depth (m)	Pressure range (kpa)	Mv (m^2)/mn	Cv(m^2/yr)
03	1.50	25 – 50	0.028	0.4
		50 – 100	0.037	0.5
		100 – 200	0.028	0.8
		200 – 400	0.028	0.9
03	1.00	25 -50	0.012	0.6
		50-100	0.020	0.6
		100 -200	0.030	0.4
		200 -400	0.035	0.7
06	3.00	25 – 50	0.054	0.4
		50 – 100	0.024	0.3
		100 – 200	0.032	4.7
		200 - 400	0.044	0.7
03	1.50	25 – 50	0.031	0.2
		50 – 100	0.031	0.7
		100 – 200	0.035	0.8
		200 – 400	0.044	0.7
03	1.00	25 – 50	0.063	0.6
		50 – 100	0.094	0.6
		100 – 200	0.192	0.9
		200 – 400	0.158	1.6
06	3.00	25 – 50	0.070	0.6
		50 – 100	0.050	0.7
		100 – 200	0.070	0.7
		200 – 400	0.050	1.0
03	1.00	25 – 50	0.016	0.3
		50 – 100	0.020	1.3
		100 – 200	0.017	0.6
		200 – 400	0.022	0.4
06	3.00	25 – 50	0.011	0.5
		50 – 100	0.05	0.3
		100 – 200	0.05	0.4
		200 – 400	0.04	0.9
07	4.50	25 - 50	0.030	0.5
		50 – 100	0.020	0.7
		100 – 200	0.012	0.3
		200 – 400	0.020	1.2
03	1.00	25 - 50	0.034	0.3
		50 – 100	0.050	0.5
		100 – 200	0.060	1.7
		200 – 400	0.060	2.1

soils commonly called hard pan which ranges from 0 - 1.50 m in depth. This is followed by reddish brown soft sandy clay to reddish brown hard sandy silty clay which range from 1.50 m - 4.50 m in depth. The cone penetration resistance and bearing capacity indicate that the results are good for structures to be sited on them as they would not suffer excessive settlement due to the weight of the superstructure.

Conclusion

The result of the subsoil geotechnical investigations within the study area has helped in arriving at the conclusions that the top soil is generally thin and ranges in thickness from 0 - 1 m in most of the boreholes. The depth to competent rock is more than 5 m in some of the boreholes while most of the soils possess good

Table 3. Allowable bearing pressure and ultimate bearing capacity at test point**.**

Test point	Depth (m)	Cone penetration resistance (kg/cm^2)	Allowable bearing pressure (kN/m^2)	Ultimate bearing capacity (kN/m^2)
DB 01VES 01	0.25	08	21.6	64.8
	0.50	10	27	81
	0.75	12	32.4	97.2
	1.00	10	27	81
	1.25	15	40.6	121.5
	1.5	20	54	162
	1.75	28	75.6	226.8
	2.00	30	81	243
	2.25	45	121.5	364.5
	2.50	60	162	486
	2.75	100	270	810
	3.00	180	486	1458
	3.25	220	594	1782
DB01VES05	0.25	60	162	486
	0.5	65	175.5	526.5
	0.75	95	256.5	769.5
	1.00	135	364.5	1093.5
DB03VES01	0.25	20	54	162
	0.50	30	81	243
	0.75	40	108	324
	1.00	60	162	486
	1.25	65	175.5	526.5
	1.5	75	202.5	607.5
	1.75	90	243	729
	2.00	150	405	1215
DB 03VES03	0.25	30	81	243.0
	0.50	40	108	324
	0.75	55	148.5	445.5
	1.00	90	243	729
	1.25	105	283.5	850.5
	1.5	135	364.5	1093.5
	1.75	150	405	1215

Table 4. Relationship between plasticity index and swelling potential (Ola, S.A, 1981)

Plastic Index	Swelling Potentials
0-15	Low
15-25	Medium
25-35	High
>35	Very High

geotechnical properties for foundation of structures. In view of the shallow depths to water table encountered in most of the boreholes, it is recommended that structures should be founded at a depth not less than 1 m below the ground water level and the foundation width not less than 1 m. Also, structures to be constructed on loose sand, soft clay and silt can be supported on raft foundation. However, where the proposed structure is to impact a load exceeding 1000 kN/m^2 to the sub-surface, it is suggested that piles can be driven to depth of competent rock or strata.

Conflict of Interests

The authors have not declared any conflict of interest.

REFERENCES

Casagrande A (1932). Research on the Atterberg limits of soils. Public Roads13(8):121-136.
Carter M (1983). In Carter M, Bentley SP (1991). Correlations of Soil Properties, Pentech Press, London.
Clayton CRI, Mattew MC, Simons NE (1996). Site investigation. Second Edition. Blackwell scientific publishers, Oxford. In Bell FG. (2011). Eng. Geo. Second Ed. P 318.
Bell FG (2011). Eng. Geol. Second Ed. Elsevier publishers, India P. 311
Dikedi PN (2012). Geo-electric probe for Groundwater in Giri, Nigeria. Global Journal of Science Frontier Research Physics & Space Science Volume 12 Issue 2 Version 1.0 February 2012: Double Blind Peer Reviewed International Research Journal Publisher: Global J. Inc. (USA) Online ISSN: 2249-4626 & Print ISSN: 0975-5896
Ejembi S (2016). Ten Tragic Building Collapses in Nigeria. Available at: http://www.punchng.com/10-tragic-building-collapses-in-nigeria/
Johnson RB, Degraff JV (1988). Principles of Engineering Geology. John Willey and sons, New York.
Kogbe CA, Ajakaiye DE, Matheis G (1983). Confirmations of a rift structure along the Mid-Niger valley, Nigeria. J. Afr. Earth Sci. (in press).
National building code (1983). Safe bearing capacity for rocks and cohesionless soils.
Ola SA (1981). Expansive Clays in Engineering Construction: A review of Practices. J. Min. Geol. 26(1):88.
Rahaman MA (1988). Recent Advances in the study of the basement complex of Nigeria. In Precambrian Geol. Niger. In Bull. 32:11-43.
Sanglerat G (1972). The Penetrometer and soil Exploration. Elsevier, Amsterdam.
Shahriar M, Sivakugan N, Das BM (2012). Settlement of shallow foundations in granular soils due to rise of water table - A review. Int. J. Geotechnical Eng. Ross J Publishing 6 (4):515-524.
Terzaghi K (1943). Theoretical Soil Mechanics, John Willey and Sons, New York. 510p.
Terzaghi K, Peck RB (1967). Soil Mechanics in Engineering Practice. John Willey and Sons, New York. 729p.

Appendix A. Borehole logs.

Sample No.	From depth	To depth	Lithology (Soil type/Description)	SPT		
				Blows 1st	Blows 2nd	Blows 3rd
DB 01 VES 01	0.00	0.20	Greyish brown loose TOPSOIL			
DB 01 VES 01	0.20	1.50	Mottled reddish brown and grey stiff sandy silty CLAY			
DB01 VES 01	1.50	2.25	Reddish brown stiff sandy Clay	29		
DB 01 VES 01	2.25	3.00	Reddish brown hard sandy Silty CLAY			
DB 01 VES 06	0.00	0.30	Dark grey loose sandy silty CLAY			
DB 01 VES 06	0.30	2.25	Reddish brown hard sandy silty CLAY			
DB 01 VES 06	2.25		Hard pan			
DB 02 VES 01	0.00	0.75	Dark grey dense silty SAND			
DB 02 VES 01	0.75	4.50	Reddish brown stiff sandy silty CLAY	25	29	
DB02 VES 01	4.50	5.00	Dark grey ROCK			58
DB 03 VES 03	0.00	0.25	Dark grey silty SAND, TOPSOIL			
DB 03 VES 03	0.25	1.50	Reddish brown hard sandy silty CLAY			
DB 03 VES 03	1.50	3.00	Reddish brown soft sandy silty CLAY			
DB 03 VES 06	0.00	0.25	Dark grey SILT, humus stained	30		
DB 03 VES 06	0.25	1.50	Reddish brown loose sandy silty CLAY		34	
DB 03 VES 06	1.50	2.25	Reddish brown sandy silty CLAY			
DB 03 VES 06	2.25	5.00	Grey hard sandy silty CLAY			35
DB 07 VES 03	0.00	0.25	Dark grey loose sandy SILT			
DB 07 VES 03	0.25	5.25	Reddish brown hard sandy silty CLAY			
DB 07 VES 05	0.00	0.25	Dark grey hard TOPSOIL			
DB 07 VES 05	0.25	1.50	Reddish brown firm sandy sitly CLAY	35		
DB 07 VES 05	1.50	4.50	Reddish brown hard sandy silty CLAY		54	57
DB 07 VES 05	4.50	5.25	Reddish brown hard sandy silty CLAY, water table at 5.25 m			
DB 07 VES 05	5.25	6.00	Reddish brown soft sandy silty CLAY, water table at 5.25 m			
DB 07 VES 07	0.00	0.25	Dark grey loose sandy SILT			
DB 07 VES 07	0.25	1.50	Reddish brown hard sandy silty CLAY	60		
DB 07 VES 07	1.50	3.00	Mottled reddish brown and grey sandy silty CLAY			
DB 07 VES 07	3.00	4.50	Reddish brown hard sandy silty CLAY		48	
DB 07 VES 07	4.50		Bedrock			
DB 07 VES 10	0.00	1.50	Dark grey hard sandy silty CLAY	44		
DB 07 VES 10	1.50	3.25	Grey stiff sandy silty CLAY		38	
DB 07 VES 10	3.25	6.00	Grey soft sandy silty CLAY			66
DB 09 VES 01	0.00	1.50	Dark grey loose sandy silty CLAY	52		
DB 09 VES 01	1.50	2.25	Dark grey hard sandy silty CLAY		47	
DB 09 VES 01	2.25	9.00	Dark grey hard weathered rock			43

Appendix A. Contd.

DB 09 VES 09	0.00	0.75	Dark grey loose sandy silty TOPSOIL			
DB 09 VES 09	0.75	1.50	Dark brown soft sandy silty CLAY with some gravels			
DB 09 VES 09	1.50	3.00	Reddish brown stiff sandy silty CLAY becoming hard at lower reaches	38		
DB 10 VES 01	0.00	1.50	Reddish brown soft sandy silt CLAY. Water table at 0.8m			
DB 10 VES 01	1.50	3.00	Reddish brown hard sandy silty CLAY	80		
DB 11 VES 07	0.00		Reddish brown hard sandy silty CLAY becoming harder with depth			
DB 11 VES 07	4.25		Reddish brown laterised weathered rock	38		
DB 11 VES 11	0.00	0.25	Dark grey silty SAND	39		
DB 11 VES 11	0.25	3.00	Greyish brown sandy silty CLAY			
DB 11 VES 11	3.00	4.50	Brown hard sandy silty CLAY		58	
DB 11 VES 11	4.50	6.00	Reddish brown stiff sandy silty CLAY			42
DB 12 VES 02	0.00	1.50	Dark grey silty SAND TOPSOIL			
DB 12 VES 02	1.50	3.00	Reddish brown gravelly sandy silty CLAY	39		
DB 12 VES 02	3.00	3.75	Hardpan (lateritic concretion) becoming harder with depth		49	
DB 13 VES 09	0.00	0.75	Dark grey sandy SILT			
DB 13 VES 09	0.75	7.00	Reddish brown hard sandy silty CLAY	58	30	24
DB 13 VES 09	7.00		Dark grey laterised weathered ROCK			
DB 15 VES 07	0.00	0.75	Dark grey silty SAND Topsoil			
DB 15 VES 07	0.75	3.00	Reddish brownhard sandy silty CLAY. Water table at 6.5m	37	48	
DB 15 VES 07	3.00	7.00	Grey mottled soft reddish brownand grey soft silty CLAY, water table at 6.8 m			26
DB 15 VES 07	7.00	8.25	Grey mottled soft reddish brown and grey soft sandy silty CLAY			
DB 15 VES 07	8.25	9.00	Reddish brown soft silty CLAY			
DB 15 VES 11	0.00	3.00	Reddish brown hard sandy silty CLAY			
DB 15 VES 11	3.00		Reddish brown lateritised weathered ROCK	70		
DB 15 VES 15	0.00	1.50	Grey silty SAND			
DB 15 VES 15	1.50		Dark grey rock			
DB 15 VES 18	0.00	1.50	Dark grey silty clayey SAND topsoil			
DB 15 VES 18	1.50	4.50	Reddish brown soft sandy silty CLAY with relics of rock at lower reaches	20	29	
DB 17 VES 09	0.00	0.75	Dark grey silty SAND, Topsoil			
DB 17 VES 09	0.75	3.75	Reddish brown sandy silty CLAY	51	51	
DB 17 VES 09	3.75		Grey laterised weathered ROCK			
DB 17 VES 13	0.00	0.75	Grey humus rich SILT, Topsoil			
DB 17 VES 13	0.75	1.50	Grey brown hard sandy sility CLAY			
DB 17 VES 13	1.50		Brown laterised weathered ROCK	70		
DB 17 VES 17	0.00	0.75	Dark grey silty SAND, Topsoil			
DB 17 VES 17	0.75	7.50	Dark grey weathered ROCK	65	55	60
DB 17 VES 17	7.50		Dark grey hard ROCK			

Appendix A. Contd.

DB 17 VES 20	0.00	0.75	Dark grey silty SAND, Topsoil	
DB 17 VES 20	0.75	3.75	Reddish brown clayey SAND, water table at 2.5 m	
DB 17 VES 20	3.75	6.00	Reddish brown stiff sandy silt CLAY with some gravels	
DB 17 VES 20	6.00		Dark grey fresh BEDROCK	
DB 19 VES 04	0.00	1.00	Dark grey hard silty CLAY	
DB 19 VES 04	1.00		Water table	
DB 19 VES 04	1.00	3.00	Dark grey clayey sand (weathered rock)	
DB 19 VES 04	3.00	3.50	Dark grey laterised weathered ROCK	
DB 19 VES 17	0.00	1.50	Reddish brown stiff sandy silty CLAY	75
DB 19 VES 17	1.50	3.00	Reddish brown laterite concretion (hardpan)	85
DB 21 VES 06	0.00	0.75	Dark grey silty SAND, Topsoil	
DB 21 VES 06	0.75	3.75	Reddish brown stiff sandy silty CLAY with some hard pans	25
DB 21 VES 06	3.75	5.25	Brownish grey clayey SAND (weathered rock), water table at 3.75	89
DB 21 VES 06	5.25	7.50	Brownish grey clayey SAND (weathered rock)	
DB 22 VES 09	0.00	0.75	Dark brown humus rich silt TOPSOIL	
DB 22 VES 09	0.75	4.50	Reddish brown stiff sandy silty CLAY	
DB022 VES 09	4.50		Dark grey fresh BEDROCK	
DB 21 VES 10	0.00	0.75	Dark grey sandy silty CLAY	
DB 21 VES 10	0.75	4.50	Reddish brown stiff sandy silty CLAY	
DB 21 VES 10	4.50	7.50	Dark grey weathered ROCK	
DB 21 VES 10	7.50		Dark grey fresh BEDROCK	