IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)
e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 11, Issue 6 Ver. IV (Nov- Dec. 2014), PP 40-47
www.iosrjournals.org

Evaluation of Sub-Soil Geotechnical Properties for Shallow Foundation and Pavement Design in LASPOTECH, Ikorodu Campus, Lagos State, Nigeria

Adigun Muritala A[1], Olatunji Kayode A[2], Alaboru Favour O[3] and Ogunbajo Abdulhakeem B[4]
Civil Engineering Department, Lagos State Polytechnic, Ikorodu, Lagos State, Nigeria

Abstract: *Sequel to continuous failure of structures all over the country, adequate knowledge of the geotechnical characteristics of underlying soils at construction sites has become very desirable for foundation design and construction of civil engineering structures. This study aims at investigating and establishing the sub-soil types and profile to determine the engineering characteristics of the underlying soils within the Ikorodu Campus of the Lagos State Polytechnic, Ikorodu, Lagos, Nigeria and recommend appropriate foundation and road pavement design. Forty Dynamic Cone penetrometer tests were performed alongside eight borings which were accomplished using percussion rig with augers. Representative soil samples were obtained and analyzed in the laboratory in accordance with relevant geotechnical engineering standards. Result of the study showed that the study area was found to consist of about 0.35m thick of organic top soil followed by 9m thick of reddish lateritic sandy clay. The study also revealed that the superficial lateritic soil has a bearing capacity ranging from 130 kN/m^2 at 1.0m depth to 243 kN/m^2 at 2.5m depth. Shallow foundation with bearing capacity of 150kN/m^2 at footing depth not lower than 1.2m is recommended for general use within the campus. Maximum expected settlement is estimated at 27.84mm. Similarly, a CBR value of 7% is recommended as subgrade CBR value for road pavement design within the campus.*
Keywords: *Geotechnical Properties, Subsurface-soils,, Foundation Design, Pavement Design, CBR.*

I. Introduction

Records and investigations have shown that many of the stakeholders in the building construction industry have not been paying adequate attention to the role of geotechnical information in the planning, design, construction, operation and safety of civil engineering infrastructures. This neglect has been discovered as one of the sources of failure of structures [1 & 2]. Frequent structural failure of civil engineering infrastructures in parts of Lagos Metropolis has become a source of worry to the government, engineering organizations as well as many individuals, hence, a good understanding of the occurrence, composition, distribution, geologic history as well as the geotechnical properties of subsurface soils in the areas where structures are to be erected is necessary.

Lagos State as a whole occurs within an area underlain by sedimentary deposits of the Dahomey Basin which is constituted of five sedimentary formations, viz: the Abeokuta Formation; composed of sands and sandstones with clays, the Ewekoro Formation; composed of Limestones, clays and shales, the Ilaro Formation; composed of Shales and Clays with few sand lenses, the Coastal Plains Sands; composed of sands, silts, clays and traces of peat and the recent alluvial deposits; composed of soft clays, peat and loose sand[3].

In the desire to have a good design and construction of foundation of future civil engineering structures in order to minimize adverse effects and prevention of post construction problems, some general studies have been carried out on geotechnical properties of the sub-soils [4]. To obtain relevant data inputs for the design and construction of foundations for proposed structures, it is important that construction site be geo-technically characterized carrying out sub-soil investigation. This paper therefore, aims at establishing significant subsoil types and profile, investigation of the engineering characteristics of all such sub-soils to generate the required data relevant to the foundation design and construction of structures within the Ikorodu campus of Lagos State Polytechnic.

II. Description of the Study Area

Lagos State is an integral part of Nigeria coastal plain and extended Nigeria continental shelf. The deposits are therefore, geologically young, ranging from the Eocene to the recent Pliocene. The soils are mainly alluvial mix with sand, clay, peat and silt in various proportions. The state has an annual rainfall of 1185mm. The territory lies in typical West African Continental shelf with beaches and bars draining into the big sea. [5].

The project area is located within Ikorodu Local Government Area of Lagos State, Nigeria (Fig. 1) lies within the transitional zone between the Precambrian basement complex rocks of the southwestern Nigeria and the cretaceous sediments of the Abeokuta group in the eastern part of the Dahomey Basins. The basement rocks occurs predominantly in the north, northwest and northeastern part of the field and it is predominantly a Migmatite gnesis complex of biotite granite gnesiss, biotite hornblende gnesiss with varying degrees of fracturing [3]. The southern part of the field is overlain by Ise member of Abeokuta group that conformably overlies the basement rocks. Litho-stratigraphically, Abeokuta group comprises of grits, arkosic sandstone, siltstone and clay with occasional conglomerate of predominantly arenaceous materials. [4]

The clay in the area is mainly residual clay, deposited over the basement complex rocks of the project area. The residual clay was formed by surface weathering, which gives rise to clay by the chemical decomposition of the rocks, containing silica and alumina.

Figure 1 Map of Lagos State showing Ikorodu L.G.A

III. Methodology

3.1 Field Investigation

Field investigation and laboratory tests were conducted to obtain information on the underlying soils within the study area. The field investigations consist of various field activities carried out between end of 2012 and 2014. The field activities include six (8) numbers borings (BH 1 to BH 8) and 40 numbers Dynamic Cone Penetrometer Tests spread across the campus. Fig. 2 shows the campus general layout of the study area. [6].
The borings were accomplished using a light cable percussion (shell and auger) technique with a fully equipped motorized Dando 150 drilling rig. The Boring Tests were accompanied by disturbed and undisturbed samples. The field investigation revealed that closest groundwater level was at 9.00m below the ground surface. Soil samples were obtained and carefully prepared in the field and were transported to the laboratory for further visual inspection and laboratory testing to establish their physical and engineering properties.

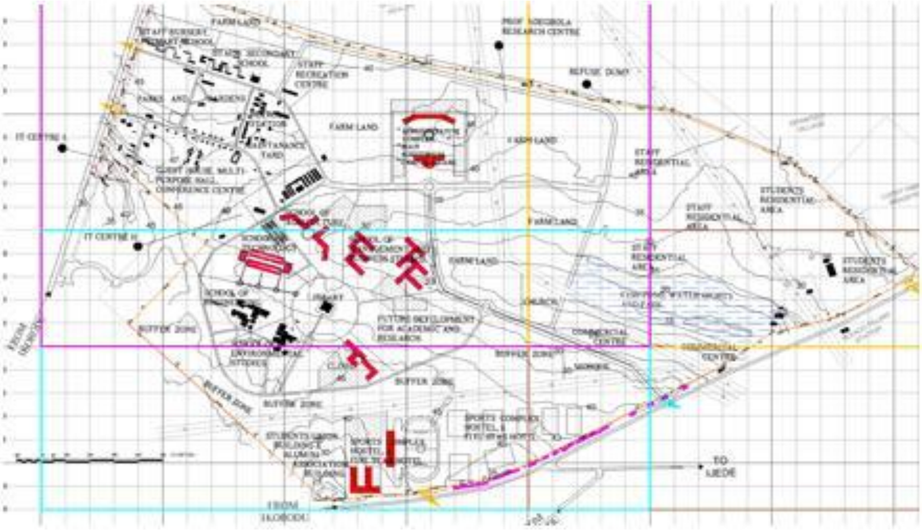

Figure 2. Plan of Lagos State Polytechnic

3.2 Laboratory Testing

All samples obtained in the field were carefully preserved and subjected to more detailed visual inspection and descriptions at the laboratory. Thereafter, representative samples were selected from each stratum for laboratory analysis in accordance with relevant geotechnical engineering standards including BS 1377; 1990. [7].The disturbed soil samples were appropriately subjected to the following laboratory classification tests:(i) Natural moisture content (ii) Atterberg limits (liquid and plastic limits).(iii)Grain size analysis. Sieve analysis of cohesive soils were done by soaking oven-dried samples in water overnight and washing through sieve No. 200

(75 microns opening) while remnants retained on sieve No. 200 were oven-dried and sieved mechanically. Materials finer than sieve number 200 were analysed using the hydrometer method based on Stoke's Law.

The undisturbed and mostly cohesive samples collected in the course of boring were subjected to: (i) Unconsolidated Undrain triaxial tests at cell pressures of 100kPa, and 300kPa; and (ii) Oedometer consolidation test. The shear strength parameters c_u and φ_u of the cohesive soil samples were obtained from the Unconsolidated Undrain triaxial test, while shear strength parameter of the granular soils were evaluated from average SPT-number of the respective stratum. The results of the physical engineering properties are presented in TABLE 1.

The formula developed by Terzaghi [8] has been adopted in estimating the soil bearing pressure. The bearing capacity equation (1) for rectangular footing, equation was used in computing the bearing capacity for shallow foundation:

$$q_u = cN_c\left(1 + 0.3\frac{B}{L}\right) + \gamma D_f N_q + 0.5\gamma B N_\gamma\left(1 - 0.2\frac{B}{L}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

q_u = Ultimate bearing capacity
c = Undrained cohesion of soil
γ = Unit weight of soil
D_f = Footing depth
B = Breadth of foundation
L = Length of foundation
N_c, N_q, N_γ= bearing capacity factors that are non-dimensional and are only functions of the soil friction angle, ϕ

The allowable bearing capacity of the soil has been evaluated with a factor of safety (F.S) of 3.0 and a summary of the allowable bearing capacity are presented in TABLE 2.

3.3 Settlement Analysis

Total consolidation settlement (ρ_c) has been computed for foundation breadth (B) between 1.00-2.50metres, subjected to a allowable bearing capacity of 150kN/m^2.

Based on the soil lithology, the thickness of the consolidating layer for a square foundation is taken as the depth to the point where the induced vertical stress (Δσ) is equal to 0.55qn. The induced vertical stress (Δσ) at the centre of the consolidating layer has been used in computing ρ_c. The consolidation settlement has been computed from the expression below. [9].

$$\rho_v = \mu_g P_{oed} = m_v \sigma_z H = m_v \text{ X } 0.55 q_n \text{ X } 1.5B \quad \ldots\ldots\ldots\ldots\ldots\ldots..(2)$$

where μ_g = Coefficient which depends on the type of clay
P_{oed} = Settlement as calculated from oedometer test
m_v = Coefficient of volume compressibility
q_n = Net foundation pressure
B = Breadth of foundation.

An m_v value of 0.075m^2/MN, which corresponds to the adopted net allowable bearing capacity was used in the settlement analysis. The results are presented in TABLE 3

3.4 The Dynamic Cone Penetrometer tests (DCP)

A total number of forty (40) DCP tests distributed over the developing sections of the study area were performed. The entire area were sub-divided into eight (8) sections. Four to six DCPT tests were carried out in each section depending on size. The average data from each section **DCP** tests were processed to produce penetration index (PI), which is simply the distance the cone penetrates with each drop **of** the hammer. The PI is expressed in terms of millimeters per blow. The results were then used to estimate the subgrade California Bearing Ratio (CBR). The Data from these tests have been analyzed using the relationship developed by Transport and Road Research Laboratory. [10].

$$\log_{10}[\text{CBR}] = 2.48 - 1.057 \log_{10}[\text{PI}] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3)$$

The results of the DCP tests and calculated values of CBR are presented in TABLE 4

IV. Results

4.1. Soil Stratigraphy

The data obtained from the boring, soil sampling, field penetration tests and laboratory tests were interpreted to determine the stratification of subsoils underlying the site. From borehole tests conducted, six subsoil zones have been delineated. This first zone (topsoil) consists of the light brown lateritic/sandy clay deposits intermingled with roots and thickness of about 0.35m. The second zone is a reddish Lateritic sandy clay deposit occurring from depths of 0.35m to 9.75m. Underlying this layer is light mottled clayey silty sand deposit of about 6m thick. This represents the third zone. The fourth zone in the soil profile is the firm to stiff mottled silt sandy clay deposit with average thickness of 3.75m. Underlying is the fifth zone comprising a layer of firm to stiff mottled silt sandy clay deposit. The thickness of this layer is about 2.25m. The sixth zone occurring between 21.75 and 30m is a mottled sandy clay deposit.

4.2 Physical and Engineering Properties of Sub-Soils

The results of field and laboratory analysis of various samples of the soil superficially dominating the site through the boring depths show that the area of study is characterized by lateritic sandy clay within the upper 9.75m. Some selected physical and engineering properties of the soil materials is summarized in Table 1a and 1b.

4.3 Bearing Capacity of soil

The allowable bearing capacity of the soil at depths 1.0m, 1.5m and 2.5m have been evaluated with factor of safety of 3.0. The summary of the net allowable bearing capacity are presented in Table 2. The variation of bearing capacity with foundation depth for square pad footing is depicted in Fig. 3

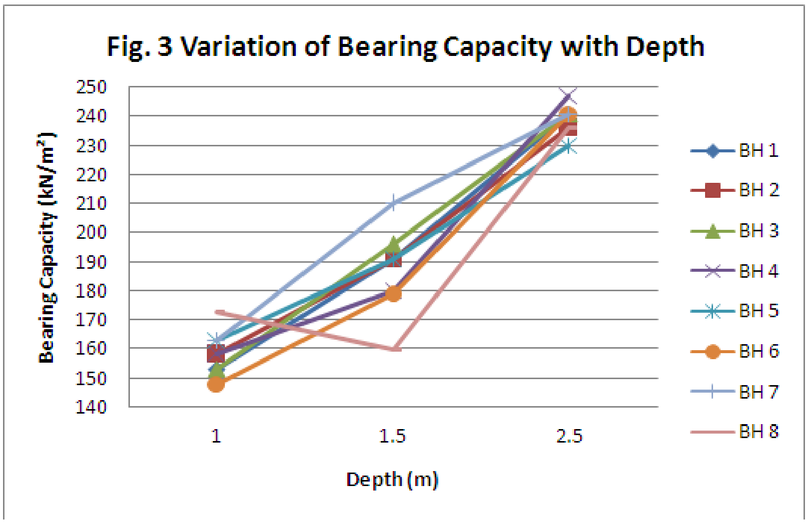

Figure 3. Variation of bearing capacity with depth

4.4 Settlement of soil

The results of the consolidation settlement for foundation breadth, B between 1.0m and 3.0m at the net allowable bearing pressure of 150kN/m^2 are presented in Table 3.

4.5 California Bearing Ratio (CBR) of subgrade soil

The results of the dynamic cone penetrometer tests showing the penetration index and the corresponding CBR values are presented in Tables 4a to 4h.

V. Discussions

Results of the study revealed that the major sub-soils underlying the study area have about 0.35m thick brown lateritic/sandy clay top soil which is followed by reddish Lateritic sandy clay which existed upto about 10m depth underlain by clayey silty sand to a depth of around 16.0m and silty sandy clay from this point to the end of boring at 30m. Appropriate type of shallow foundation will be adequate for expected structural loads within the campus since a medium to firm lateritic sandy clay existed upto a depth of about 10m and is underlain by another medium to dense silty sand. The bearing capacity generally increases with depth from 130kN/m^2 at 1.0m depth to 243kN/m^2 at 2.5m depth. An average bearing capacity of 150kN/m^2 is estimated to be adequate and this could be used in determining the foundation type for structures within the campus.

The computed settlement values as presented for a shallow foundation with footing width between 1.00 – 3.00m is less than the allowable maximum settlements suggested for isolated foundations on clays of 65mm. [11]. Hence the foundation can sustain the suggested allowable bearing pressure of 150kN/m^2.

The DCP tests results indicated that the first 200mm to 400mm layer which consists majorly of top soil is characterized with CBR value of 6.12% to 8.73%. The soil layer below the top soil is characterized with a minimum CBR value of 7.42% and maximum CBR value of 14.08% between 400mm and 1000mm depth

The results also show that penetration index values generally decreases with depth indicating increase in CBR with depth. The values of the CBR increases from 7.42% at 400mm depth to a maximum CBR value of about 18.17% at 2000mm depth.

VI. Conclusion

Upon removal of the organic top soil, the superficial lateritic sandy clay is found suitable in quality to sustain shallow foundation loads by low to medium rise structures. It also possesses enough thickness to completely dissipate the influence of such foundations. An allowable bearing pressure of 150kN/m^2 is recommended for the campus for shallow foundations which could be placed between 1.2m and 2.5m depth. Expected maximum settlement is 27.84mm, this is less than the allowable maximum settlement of 65mm for isolated foundation in clays. An average value of 7% is also recommended as CBR value for the subgrade layer. The pavement thickness of sub-base, base and surfacing should be dependent on this value. However, localized soft point(s) along the road route should be subjected to further and detailed investigation to obtain appropriate design parameters.

References

[1]. R.C Murat, (1970). Stratigraphy and Paleogeography of the Cretaceous and Lower Tertiary in Southern Nigeria (African Geology, University of Ibadan Press, Ibadan, Nigeria) 1970

[2]. A.N Amadi,; C.J Eze,; C.O Igwe,; I.A Okunlola, and N.O Okoye, Architect's and geologist's view on the causes of building failures in Nigeria. Modern Applied Science, Vol.6 (6), 2012, 31 – 38.

[3]. A.I Olayinka,. and O.O Osinowo, Integrated geophysical and satellite imagery mapping for groundwater assessment in a geological transition zone in south-western Nigeria, SAGEEP, Vol. 22, 2009, pp977-987.

[4]. S.O Olabode and J.A Adekoya, Seismic stratigraphy and development of Avon canyon in Benin (Dahomey) basin, southwestern Nigeria, Journal of African Earth Sciences, Volume 50, Issue 5, 2008, Pages 286–304

[5]. E.O Longe, Groundwater Resources Potential in the Coastal Plain Sands Aquifers, Lagos, Nigeria, Research Journal of Environmental and Earth Sciences 3(1), 2011, 1-7

[6]. MASTER PLAN of Lagos State Polytechnic, Works Department, Lagos State Polytechnic, Ikorodu

[7]. British Standard Institutions, Methods of Test for soils for Civil Engineering Purposes. B.S 1377: Part 2, 1990. pp 8 – 200

[8]. K Terzaghi, Theoretical soil mechanics (John Willey, 1943)

[9]. M A Stroud, and F G Butler, The standard penetration test and the engineering properties of Glacial materials. In: Proceedings of the Symposium of glacial materials, University of Birmingham.1975

[10]. Transport and Road Research Laboratory, Road note 8, A user's manual for a program to analyse dynamic cone penetrometer data

[11]. A.W Skempton, and D.H MacDonald, The Allowable Settlement of Buildings, Proc. Inst. Of Civil Engineers, Part 3, Vol. 5, 1956, pp. 727-784.

Table 1a: Physical and Engineering properties of the soil

Properties	Depth (m)	Mean Value at Test Point							
		1	2	3	4	5	6	7	8
Natural Moisture content (%)	1.00	19	21	19	19	20	20	18	20
	1.50	21	20	19	20	22	19	21	19
	2.50	20	20	18	18	22	20	21	19
Bulk Unit Weight (kN/m³)	1.00	19.60	19.40	19.70	19.94	19.12	19.80	18.98	19.81
	1.50	19.82	19.90	19.50	19.60	19.65	19.84	19.24	20.04
	2.50	19.98	20.01	19.88	19.75	19.78	20.15	19.62	20.08
Dry Unit Weight (kN/m³)	1.00	16.47	16.03	16.55	16.76	15.93	16.50	16.08	16.51
	1.50	16.38	16.58	16.39	16.33	16.11	16.67	15.90	16.84
	2.50	16.65	16.68	16.85	16.74	16.21	16.79	16.21	16.87
Effective Unit Weight (kN/m³)	1.00	9.79	9.59	9.89	10.13	9.31	9.99	9.17	10.00
	1.50	10.01	10.09	9.69	9.79	9.84	10.03	9.43	10.23
	2.50	10.17	10.20	10.07	9.94	9.97	10.34	9.81	10.27
Liquid Limit (%)	1.00	42	44	43	41	42	44	41	40
	1.50	42	42	41	40	39	41	42	39
	2.50	44	41	43	42	44	44	43	40
Plastic Limit (%)	1.00	19	21	22	20	21	22	21	20
	1.50	19	20	19	21	19	21	19	19
	2.50	20	21	20	22	20	22	20	21
Plasticity Index (%)	1.00	23	23	21	21	21	22	20	20
	1.50	23	22	22	19	20	20	23	20
	2.50	24	20	23	20	24	22	23	19
Undrained Frictional Angle, (°)	1.00	9	10	9	11	10	9	10	11
	1.50	11	11	11	11	11	10	12	9
	2.50	12	12	13	13	11	12	12	11

Undrained cohesion (kN/m²)	1.00 1.50 2.50	28 29 31	27 29 30	28 30 31	25 27 30	28 29 32	26 29 31	28 30 31	28 29 33
Void ratio	1.00 1.50 2.50	0.56 0.57 0.54	0.60 0.55 0.54	0.55 0.57 0.53	0.53 0.57 0.54	0.61 0.60 0.59	0.56 0.54 0.53	0.60 0.62 0.59	0.56 0.53 0.52
Specific Gravity		2.66	2.63	2.64	2.65	2.65	2.62	2.64	2.65
Effective particle size, d_{10} (mm)		0.221	0.230	0.206	0.186	0.222	0.209	0.213	0.229
Effective particle size, d_{10} (mm)		0.515	0.500	0.495	0.510	0.520	0.522	0.500	0.516
Effective particle size, d_{10} (mm)		0.340	0.380	0.330	0.350	0.390	0.360	0.345	0.365
Coefficient of uniformity, C_u		1.54	1.65	1.60	1.88	1.76	1.72	1.62	1.59
Coefficient of curvature, C_c		4.33	3.33	3.99	3.11	3.10	3.46	3.79	3.79
Classification (Unified)		CL	CL	CL	CL	CL	CL	CL	CL

Table 2: Estimated Bearing Capacity

BH No.	Depth of Sample (m)	Unit Weight, γ (kN/m³)	Frictional Angle, φ (°)	Undrained cohesion, c	Breadth/ Length ratio	Allowable Bearing Capacity, qu, (kN/m²)
1	1.00	19.6	9	28	0.50 0.75 1.00	138 145 153
	1.50	19.81	11	29	0.50 0.75 1.00	173 182 191
	2.50	19.83	12	31	0.50 0.75 1.00	222 232 241
2	1.00	19.6	10	27	0.50 0.75 1.00	142 150 158
	1.50	19.81	11	29	0.50 0.75 1.00	173 182 191
	2.50	19.83	12	30	0.50 0.75 1.00	217 226 236
3	1.00	19.6	9	28	0.50 0.75 1.00	138 145 153
	1.50	19.81	11	30	0.50 0.75 1.00	178 187 196
	2.50	19.83	13	31	0.50 0.75 1.00	238 249 259
4	1.00	19.6	11	25	0.50 0.75 1.00	143 150 158
	1.50	19.81	11	27	0.50 0.75 1.00	164 172 180
	2.50	19.83	13	30	0.50 0.75 1.00	233 243 253
5	1.00	19.6	10	28	0.50 0.75 1.00	147 155 163
	1.50	19.81	11	29	0.50 0.75 1.00	173 182 191
	2.50	19.83	11	32	0.50 0.75 1.00	211 221 230
6	1.00	19.6	9	26	0.50 0.75 1.00	129 136 143
	1.50	19.81	10	29	0.50 0.75 1.00	162 170 179
	2.50	19.83	12	31	0.50	222

					0.75 1.00	232 241
7	1.00	19.6	10	28	0.50 0.75 1.00	147 155 163
	1.50	19.81	12	30	0.50 0.75 1.00	191 200 210
	2.50	19.83	12	31	0.50 0.75 1.00	222 232 241
8	1.00	19.6	11	28	0.50 0.75 1.00	157 165 173
	1.50	19.81	9	29	0.50 0.75 1.00	152 160 167
	2.50	19.83	11	33	0.50 0.75 1.00	216 226 236

Table 3: Total Consolidation Settlement

Foundation breadth, B (m)	Settlement ρv (mm)
1.0	9.28
1.5	13.92
2.0	18.56
2.5	23.20
3.0	27.84

Table 4a: Average CBR value at Section 1

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	7	28.57	0.94	8.73
400	8	25.00	1.00	10.05
600	8	25.00	1.00	10.05
800	7	28.57	0.94	8.73
1000	8	25.00	1.00	10.05
1200	10	20.00	1.10	12.73
1400	10	20.00	1.10	12.73
1600	11	18.18	1.15	14.08
1800	12	16.67	1.19	15.43
2000	12	16.67	1.19	15.43

Table 4b: Average CBR value at Section 2

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	11	18.18	1.15	14.08
400	8	25.00	1.00	10.05
600	8	25.00	1.00	10.05
800	8	25.00	1.00	10.05
1000	9	22.22	1.06	11.39
1200	9	22.22	1.06	11.39
1400	12	16.67	1.19	15.43
1600	13	15.38	1.23	16.80
1800	13	15.38	1.23	16.80
2000	14	14.29	1.26	18.17

Table 4c: Average CBR value at Section 3

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	5	40.00	0.79	6.12
400	7	28.57	0.94	8.73
600	8	25.00	1.00	10.05

Table 4d: Average CBR value at Section 4

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	7	28.57	0.94	8.73
400	7	28.57	0.94	8.73
600	8	25.00	1.00	10.05

800	8	25.00	1.00	10.05
1000	6	33.33	0.87	7.42
1200	6	33.33	0.87	7.42
1400	10	20.00	1.10	12.73
1600	11	18.18	1.15	14.08
1800	11	18.18	1.15	14.08
2000	12	16.67	1.19	15.43

800	9	22.22	1.06	11.39
1000	9	22.22	1.06	11.39
1200	11	18.18	1.15	14.08
1400	12	16.67	1.19	15.43
1600	12	16.67	1.19	15.43
1800	13	15.38	1.23	16.80
2000	13	15.38	1.23	16.80

Table 4e: Average CBR value at Section 5

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	5	40.00	0.79	6.12
400	6	33.33	0.87	7.42
600	5	40.00	0.79	6.12
800	5	40.00	0.79	6.12
1000	6	33.33	0.87	7.42
1200	5	40.00	0.79	6.12
1400	6	33.33	0.87	7.42
1600	7	28.57	0.94	8.73
1800	7	28.57	0.94	8.73
2000	7	28.57	0.94	8.73

Table 4f: Average CBR value at Section 6

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	6	33.33	0.87	7.42
400	9	22.22	1.06	11.39
600	10	20.00	1.10	12.73
800	11	18.18	1.15	14.08
1000	12	16.67	1.19	15.43
1200	13	15.38	1.23	16.80
1400	14	14.29	1.26	18.17
1600	14	14.29	1.26	18.17
1800	14	14.29	1.26	18.17
2000	14	14.29	1.26	18.17

Table 4g: Average CBR value at Section 7

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	6	33.33	0.87	7.42
400	5	40.00	0.79	6.12
600	6	33.33	0.87	7.42
800	6	33.33	0.87	7.42
1000	7	28.57	0.94	8.73
1200	8	25.00	1.00	10.05
1400	9	22.22	1.06	11.39
1600	11	18.18	1.15	14.08
1800	12	16.67	1.19	15.43
2000	12	16.67	1.19	15.43

Table 4h: Average CBR value at Section 8

Depth (mm)	No. of Blow	Penetration Index (mm/Blow)	Log(CBR)	CBR (%)
0	0			
200	6	33.33	0.87	7.42
400	7	28.57	0.94	8.73
600	8	25.00	1.00	10.05
800	9	22.22	1.06	11.39
1000	9	22.22	1.06	11.39
1200	11	18.18	1.15	14.08
1400	11	18.18	1.15	14.08
1600	12	16.67	1.19	15.43
1800	11	18.18	1.15	14.08
2000	12	16.67	1.19	15.43