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ASSESSMENT OF BEARING CAPACITY OF SOILS IN ILE-IFE

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ABSTRACT: *In foundation design, there is the need to determine the bearing capacity of the underlying soil on which the foundations will be laid. This study therefore investigated the bearing capacity of selected soils in Ile-Ife, Osun State, Southwestern Nigeria. This was with a view to assessing the suitability of the soils as foundation materials. Twenty soil samples were collected from identified twenty construction sites within the study area. Preliminary and engineering properties tests such as natural moisture content, particle size analysis, specific gravity, Atterberg limits, compaction and triaxial were conducted on the soil samples. Shear strength parameters were determined from the results of the triaxial test, and the parameters were subsequently imputed into the Terzaghi's bearing capacity equations to obtain the bearing capacity of the soils. Results showed that, 80 % of the soil samples fell into A-2-7, using American Association of State Highway and Transportation Officials (AASHTO) classification. Using Unified Soil Classification System (USCS), 70 % of the soil samples were well-graded sand (SW). For strip footings, the bearing capacity values ranged from 83.15 kN/m^2 to 2697.08 kN/m^2; for circular footings, the values ranged from 105.14 kN/m^2 to 2791.83 kN/m^2; and for square footings, the values ranged from 105.20 kN/m^2 to 2932.06 kN/m^2. It was concluded that all the samples were c-ϕ soils, and they could be described as excellent to good foundation materials.*

KEYWORDS: Bearing Capacity, Construction, Foundation, Shear Strength, Strength Test

INTRODUCTION

Detailed geotechnical investigations of foundation soils are very important in order to guide against building collapse. Material study of foundation soils serves, to a large extent, as preventive measure for foundation failures (Owoyemi and Awojobi, 2016).

According to Aghamelu *et al.* (2011), bearing capacity analytical procedures for foundation stability abound; however, most existing procedures require that series of field and laboratory tests be conducted in order to generate most components of the adopted bearing capacity equation(s).

The bearing capacity for both strip and circular footings on undrained clay has already been one of the common topics in geotechnical engineering for researchers and engineers. In offshore regions, foundations in soft marine deposits as deeply penetrated spudcan, skirted and caisson foundations are often considered as circular embedded foundations. In most cases, it is important to take into account the embedment depth which often exceeds the footing diameter. Undrained vertical bearing capacity of embedded foundations has been first extensively studied through experimental and analytical methods for a wide range of foundation-soil interface conditions (Terzaghi, 1943; Tani and Craig, 1995), while numerical investigations of the bearing capacity of embedded footings have been reported by Hu *et al.* (1999), Salgado *et al.* (2004), Edwards *et al.* (2005), Gourvenec and Mana (2011).

Chen (1975) used limit analysis approach and employed Prandtl (1921) mechanism for the evaluation of bearing capacity factors for rough and smooth footings respectively. Finite element analysis has also been used by different investigators in conjunction with plasticity theory, to predict bearing capacity of strip footings.

By using the finite element method, Hu *et al.* (1999) investigated the undrained bearing capacities of skirted circular rigid foundations with embedment depth up to five times the foundation diameter (D) for a displacement of 0.3D in a non-homogeneous soil. The results showed a difference in bearing capacity between rough and smooth side cases, which increases with the increasing depth ratio.

Salgado *et al.* (2004) studied the two- and three-dimensional bearing capacities of embedded strip, square, circular and rectangular foundations in clay with embedment ratios up to five using the finite element limit analysis approach. The footing base was assumed rough. The collapse load was expressed in terms of the vertical load transmitted to the soil at the base of the footing.

Edwards *et al.* (2005) reported the finite element analyses of rough circular foundation with embedment ratios up to four for a displacement of 0.3D in undrained homogeneous soil. The footing base was assumed rough but the vertical side of the footing was represented to be both rough and smooth.

Adunoye and Agbede (2013a) modelled the relationship between fines content and bearing capacity (square footing) of selected soils, using non-linear regression. They found that the bearing capacity of soil samples generally reduced with increase in fines content.

Nwankwoala and Warmate (2014) studied the foundation geotechnical properties of a selected site in Port Harcourt. They recommended that pile foundation should be used to take imposed load from the cellar to the underlying sand stratum in the construction of structures in the study area.

Using multiple linear regression statistical analysis tool, Adunoye and Agbede (2014) developed predictive models for generating the bearing capacity of soils. The study revealed a low level of variance between experimental values and model values of bearing capacity.

Adunoye and Agbede (2017) modelled the bearing capacity of circular footing on lateritic soil. This was done by studying the relationship between bearing capacity of circular footing and fines content of selected lateritic soils. The developed regression model was found to be valid for the selected locations.

Researchers (Ola, 1988; Ige and Ogunsanwo, 2009) have worked on the geotechnical properties of foundation soils in Nigeria. However, there is presently no documented work on the assessment of bearing capacity of the soils in the study area.

Also, failure of structures is imminent as a result of the use of wrong foundation type corresponding to the engineering strength (bearing capacity) of the soil present on the site. Therefore, determination of the most suitable and economical foundation type for different soil

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types in building construction is of utmost importance. The results of this study will therefore serve as aid for engineers and contractors working in the study area. It will also add to the existing body of knowledge on bearing capacity in relation to soil characteristics.

The specific objectives of the study were to: (i) characterise the selected soil samples; (ii) determine the strength parameters of selected soils; and (iii) determine and analyse the bearing capacity of the soils.

Description and Geology of the Study Area

The study area is Ile-Ife, located within Latitude 7°26'N and 7°32'N and Longitude 4°29'E and 4°35'E and covering an area of about 1,894 km^2 in Ife Central Local Government Area, Osun State, Southwestern Nigeria (Ajala and Olayiwola, 2013; Udama *et al*., 2017) (Figure 1). The study area falls within the basement complex of Southwestern Nigeria (Figure 2). It forms part of the African crystalline shield which consists predominantly of migmatised and undifferentiated gneisses and quartzite (Akintola, 1982; Areola, 1982; Adunoye and Agbede, 2013b; Bankole and Adeoye, 2014; Adunoye *et al*., 2018; Abdulazeez *et al*., 2020).

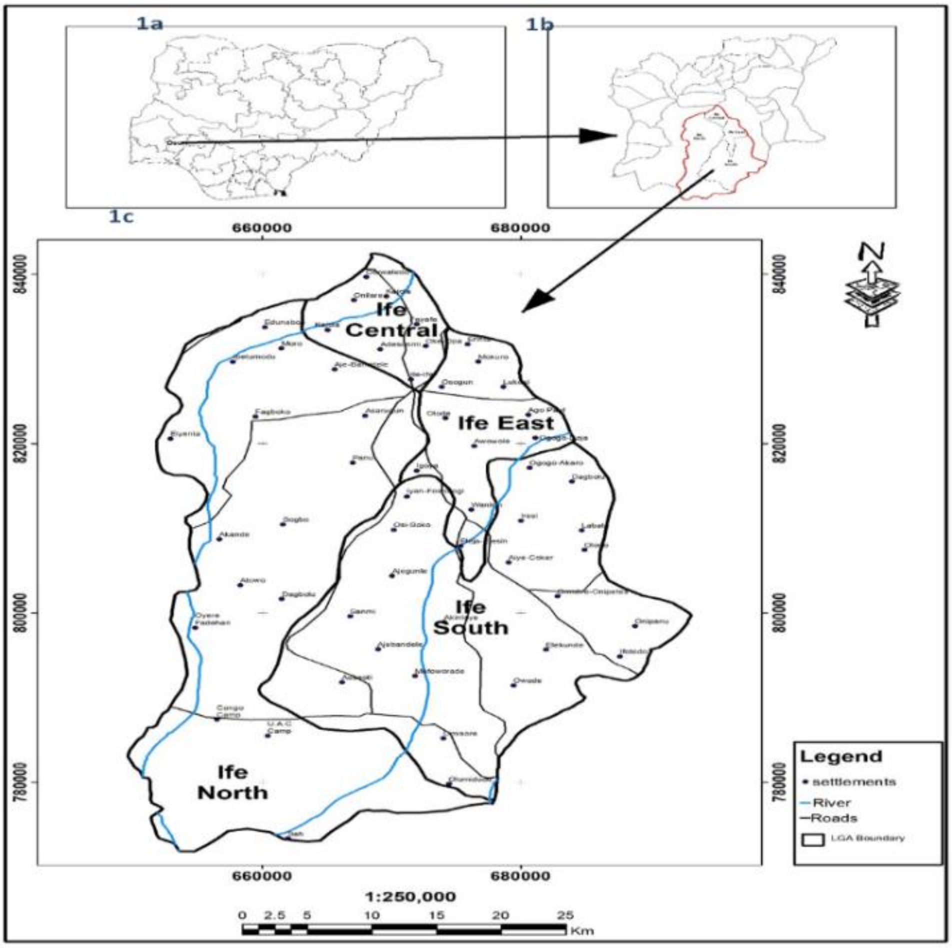

Figure 1: Map of the study area: (a) Map of Nigeria showing Osun State; (b) Map of Osun State showing Ile-Ife; (c) Map of Ile-Ife

Source: Adapted from Udama et al., 2017

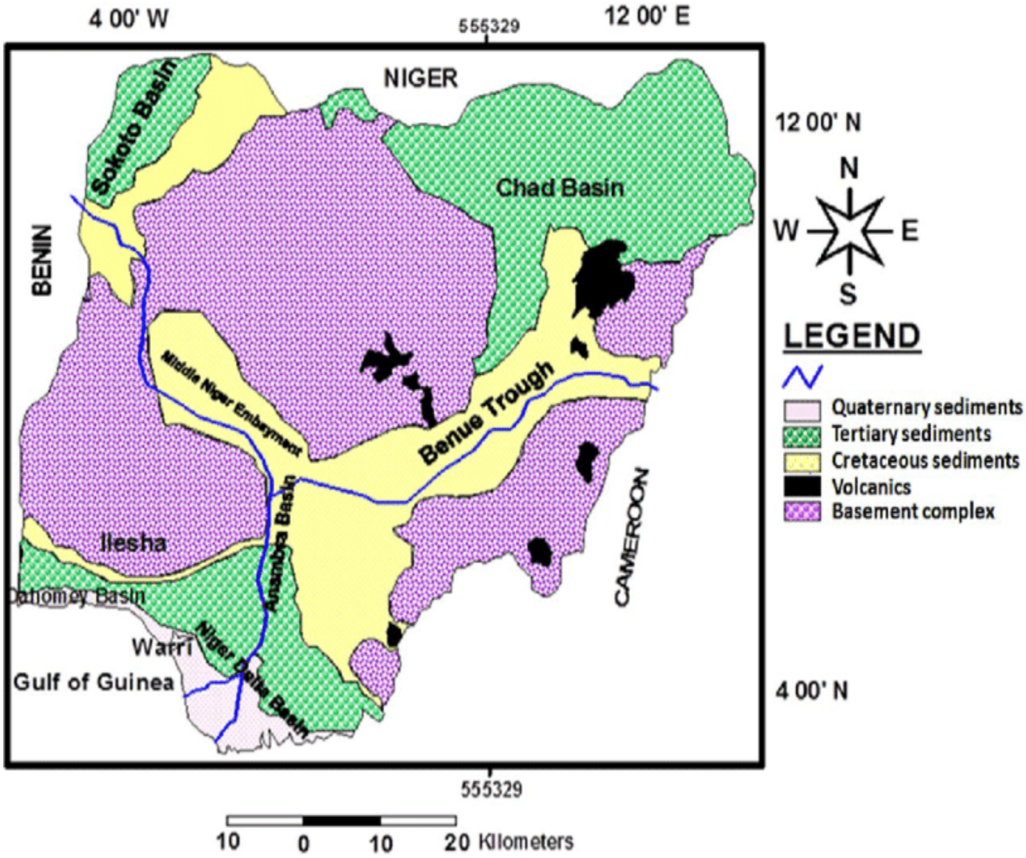

Figure 2: Map of the Major Geological Formations of Nigeria

Source: Bankole and Adeoye, 2014

MATERIALS AND METHODS

Materials and equipment

The main material used for this study was soil samples collected from twenty (20) different construction sites within the study area. The equipment used for laboratory tests were: set of British Standard (BS) sieves, moisture cans and tray, weighing balance, drying oven, Casssagrande apparatus, compaction apparatus, and triaxial machine.

Sample Collection and Preparation

A total of twenty (20) disturbed soil samples (one from each location) were collected. The depth of collection varied between 0.5 m and 1 m (Arora, 1988). About 25 kg of soil was collected from each of the sampling points, with the aid of hand auger. The samples were placed inside polythene bag, well sealed and immediately taken to the Geotechnical Engineering Laboratory, Department of Civil Engineering, Obafemi Awolowo Univerisiy (OAU), Ile-Ife, Nigeria. At the laboratory, representative samples were taken for natural moisture content determination (using oven method), while the remaining soils were air-dried for subsequent laboratory tests/analyses.

Preliminary and Geotechnical Analyses of Soil Samples

Having determined the natural moisture contents of the soil samples, the following tests were conducted on the air-dried samples, following standard methods as outlined in BS 1377 (1990): Particle size analysis, specific gravity, Atterberg limits (liquid limit, plastic limit, plasticity index – the arithmetic difference between liquid limit and plastic limit).

Engineering Tests

Compaction test and triaxial test were also conducted on the soil samples using standard procedures as outlined in BS 1377 (1990). The compacted method adopted was Standard Proctor. The optimum moisture content (OMC) obtained from the compaction tests were used to remould the soil samples for unconsolidated undrained triaxial test, for the determination of shear strength parameter – cohesion (*c*) and angle of internal friction (ϕ).

Determination of Bearing Capacity

The shear strength parameters obtained from the triaxial test were employed in the Terzaghi's bearing capacity equations (1) to (3) to obtain the bearing capacity of the soils, for different footing types/geometry – square footing, circular footing and strip footing, respectively. The values of corresponding bearing capacity factors were obtained from Das (2006). A factor of safety value of 3 was adopted and unit width and unit depth were also adopted for each of the footings.

$$\mathrm{Q_u} = c\mathrm{N_c} + \gamma \mathrm{D N_q} + 0.5\gamma \mathrm{B N_\gamma} \quad (1)$$
$$\mathrm{Q_u} = 1.3c\mathrm{N_c} + \gamma \mathrm{D N_q} + 0.4\gamma \mathrm{B N_\gamma} \quad (2)$$

$$\mathrm{Q_u} = 1.3c\mathrm{N_c} + \gamma \mathrm{D N_q} + 0.3\gamma \mathrm{B N_\gamma} \quad (3)$$

Where Q_u = ultimate bearing capacity (kN/m^2);

c = cohesion (kN/m^2);

γ = effective unit Weight of soil (kN/m^3);

D = depth of footing (m);

B = width of footing (m);

N_c, N_q and N_γ are bearing capacity factors, which depend on the values of angle of internal friction ϕ.

RESULTS AND DISCUSSION

Description of Sample Locations

The Global Positioning System (GPS), altitude and depth of excavation of each sampling point are presented in Table 1.

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Results of Preliminary and Index Property Tests

The results of preliminary and index property tests conducted on the soil samples are presented in Table 2 and discussed as follows.

Natural Moisture Content

As evident in Table 2, Sample 7 had the highest natural moisture content of 33.20 %, while Sample 19 had the lowest natural moisture content of 10.38 %. 12 soil samples, representing 60% of the samples, had natural moisture content higher than 20 % while the remaining 40 % had theirs less than 20 %. This trend can be attributed to the prevailing climatic condition when the soil samples were collected. The samples were collected during the rainy season (between July and August 2018)

Table 1: Description of Sampling Locations

Sample ID	Latitude	Longitude	Altitude above sea level (m) (m)	Depth of Excavation (m)
Sample 1	N7^0 28' 1728.95"	E4^0 28' 1728.95"	292	0.8
Sample 2	N7^0 29' 1784.74"	E4^0 28' 1724.44"	250	0.7
Sample 3	N7^0 29' 1789.54"	E4^0 28' 1725.15"	255	0.7
Sample 4	N7^0 29' 1790.24"	E4^0 28' 1724.50"	263	1.0
Sample 5	N7^0 29' 1789.75"	E4^0 28' 1724.94"	265	1.0
Sample 6	N7^0 29' 1784.80"	E4^0 30' 1846.77"	245	1.0
Sample 7	N7^0 29' 1787.57"	E4^0 30' 1856.44"	242	0.9
Sample 8	N7^0 29' 1779.55"	E4^0 31' 1914.98"	263	0.8
Sample 9	N7^0 29' 1772.07"	E4^0 31' 1901.07"	293	0.8
Sample 10	N7^0 29' 1785.28"	E4^0 31' 1888.58"	282	1.0
Sample 11	N7^0 30' 1836.04"	E4^0 33' 2032.89"	267	0.7
Sample 12	N7^0 30' 1838.14"	E4^0 34' 2040.16"	266	0.6
Sample 13	N7^0 30' 1854.56"	E4^0 35' 2105.13"	270	0.8
Sample 14	N7^0 30' 1855.01"	E4^0 35' 2103.10"	274	0.8
Sample 15	N7^0 30' 1848.97"	E4^0 35' 2104.13"	270	0.9
Sample 16	N7^0 30' 1800.82"	E4^0 30' 1809.65"	261	0.7
Sample 17	N7^0 30' 1805.60"	E4^0 30' 1835.50"	264	0.8
Sample 18	N7^0 30' 1803.07"	E4^0 30' 1809.38"	257	0.7
Sample 19	N7^0 29' 1799.70"	E4^0 30' 1800.01"	245	0.6
Sample 20	N7^0 30' 1805.65"	E4^0 30' 1816.60"	267	0.7

Specific Gravity

Samples 6 and 19 had the highest specific gravity of 2.80, while sample 20 had the lowest specific gravity of 2.43 (see Table 2). Also, 65% of the soil samples had their specific gravity greater than 2.60, while the rest had theirs less than 2.60. According to Das (2006), the specific gravity of clayey and silty soils may vary from 2.6 to 2.9. It could therefore be deduced that majority of the soil samples collected were silty-clayey in nature.

Particle Size Analysis

The results of particle size analysis (Table 2) showed that the highest percentage of fine content was 4.55 % (Sample 7), while the lowest was 1.00 % (Samples 16, 17 and 19). The average percentage fines in the soils was 1.78 %, which indicates that the soil samples were coarse in nature.

Atterberg Limits

As presented in Table 2, the highest value of liquid limit was 61.46 % (Sample 11) and the least value was 33.64 % (Sample 10). The highest value of plastic limit was 48.05 % (Sample 17) and the minimum value was 14.50 % (Sample 3). Also 37.26 % (Sample 5) was the highest plasticity index, while the lowest plasticity index was 13.05 % (Sample 17). According to Whitlow (1995), a soil having liquid limit less than 35 % has low plasticity; between 35 % and 50 % has intermediate plasticity; while 50 % - 70 % liquid limit indicates high plasticity and 70 % - 90 % very high plasticity. On this basis, 5 % of the soil samples has low plasticity; 65 % has intermediate plasticity and 30 % has high plasticity.

Table 2: Results of Preliminary and Index Properties Tests of Soil Samples

Sample ID	w (%)	G_s	Fine Content (%)	LL (%)	PL (%)	PI (%)	AASHTO Classification	USCS Classification
Sample 1	24.59	2.54	3.03	45.50	26.27	19.23	A-2-7	SW
Sample 2	15.14	2.75	1.02	48.53	22.65	25.68	A-2-7	SW
Sample 3	15.05	2.62	1.02	43.39	14.50	28.89	A-2-7	SW
Sample 4	31.45	2.64	1.01	44.42	27.27	17.15	A-2-7	SW
Sample 5	23.51	2.50	1.00	56.77	19.51	37.26	A-2-7	SW
Sample 6	19.87	2.80	3.05	41.83	20.77	21.06	A-2-7	SW
Sample 7	33.20	2.59	4.55	52.31	33.28	18.93	A-2-7	SP
Sample 8	26.61	2.46	1.01	53.89	21.43	32.46	A-2-7	SW
Sample 9	20.98	2.62	2.00	48.00	26.26	21.74	A-2-7	SW
Sample 10	24.26	2.73	3.03	33.64	19.55	14.09	A-2-6	SP
Sample 11	27.80	2.64	2.00	61.46	40.98	20.48	A-2-7	SW
Sample 12	13.89	2.55	4.06	35.66	22.44	13.22	A-2-6	SW
Sample 13	24.98	2.70	2.51	47.27	18.17	29.10	A-2-7	SP
Sample 14	23.45	2.65	2.02	43.90	19.94	23.96	A-2-7	SW
Sample 15	25.38	2.50	2.01	56.53	31.85	24.68	A-2-7	SP
Sample 16	15.43	2.61	1.00	36.97	22.90	14.70	A-2-6	SW
Sample 17	17.73	2.32	1.00	61.10	48.05	13.05	A-2-7	SP
Sample 18	20.38	2.71	1.02	44.16	22.06	27.10	A-2-7	SW
Sample 19	10.88	2.80	1.01	38.00	17.03	20.97	A-2-6	SP
Sample 20	12.83	2.43	1.51	47.71	18.52	29.19	A-2-7	SW

Soil Classification

The soil classification using the results of the index properties according to AASHTO and USCS are also presented in Table 2. From AASHTO classification, 20 % of the soil samples

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are A-2-6, while the remaining 80 % are A-2-7. On the other hand, USCS classification shows that 70 % of the soil samples were well graded sand fine to coarse (SW), while 30 % were poorly graded sand (SP). The soils can therefore be regarded as excellent to good foundation materials.

Results of Engineering Properties Tests

The results of compaction and triaxial tests are discussed below.

Compaction Test

The variation of the MDD and OMC is presented in Figure 3. The highest value of OMC of the soils was 29.67 % (Sample 11), while the lowest OMC value was 12.64 % (Sample 10). The highest value of the MDD was 1975.33 kg/m^3 (Sample 12), while the lowest MDD value was 1522.99 kg/m^3 (Sample 11). Sixty-five (65) % of the soil samples had OMC within the range 10 % - 20 % while the remaining 35 % had OMC within 20 % - 30 %. Only 5 % of the soil samples had MDD within the range 1000 kg/m^3 – 1600 kg/m^3, while the remaining 95 % had MDD values within 1600 kg/m^3 – 2000 kg/m^3. According to Murthy (2002), the greater the degree of compaction the greater the value of cohesion and the angle of shearing resistance. Thus, soils compacted with high moisture become saturated with a consequent loss of strength; that is, the greatest shear strength is attained at a moisture content lower than the OMC. Therefore, considering the fact that most of the soil samples had lower moisture content before their MDD were obtained, it could be concluded that majority of the soil samples are likely to have high bearing capacity values.

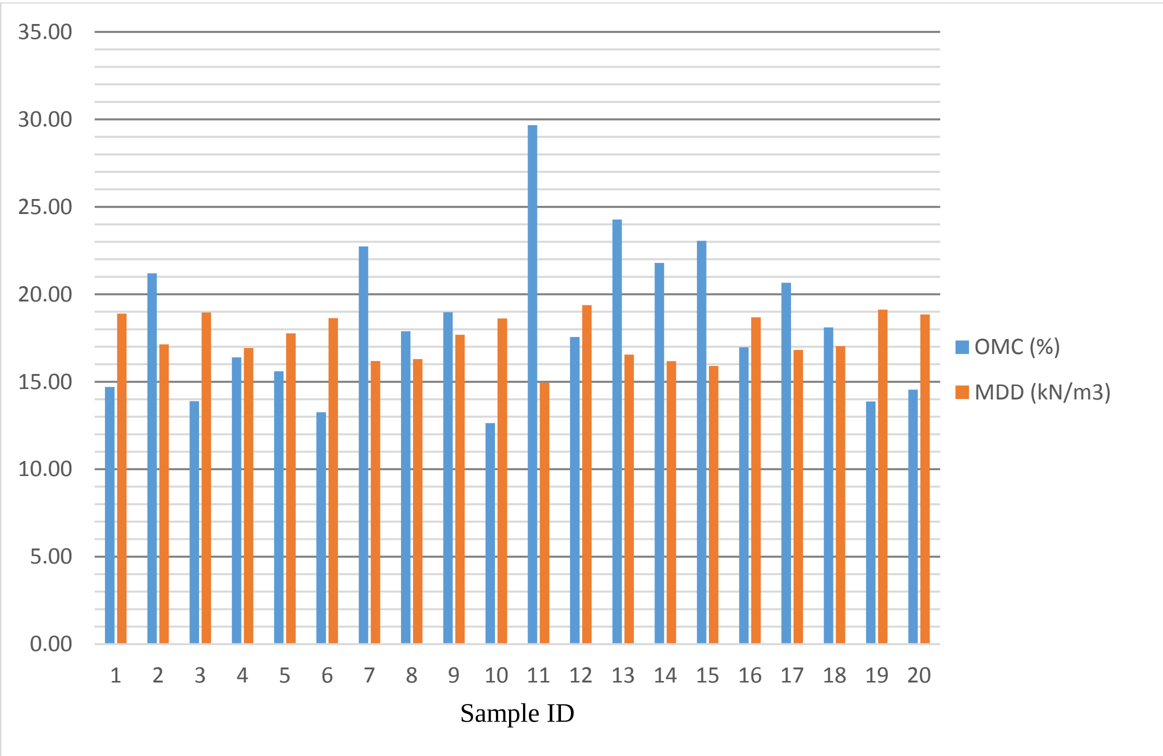

Figure 3: Variation of OMC and MDD of Soil Samples

Unconsolidated Undrained Triaxial Test

The values of shear strength parameters (*c* and ϕ) obtained from the triaxial test are presented in Table 3. Sample 4 had the highest cohesion of 96 kN/m^2, while Sample 10 had the lowest cohesion of 16 kN/m^2. The highest internal friction angle was 44^o (Sample 14), while the lowest internal friction angle was 3^0. According to Murthy (2002), the internal friction angle is within 26^o and 48^0 for granular soils, while internal friction angle less than 26^0 is for fine soils. As evident, 60 % of the soil samples had internal friction angle between 26^o and 48^o and could therefore be described as granular, while the remaining 40 % could be described as fine soils.

Bearing Capacity of Soil

The results of bearing capacity computation are presented in Table 3. The results showed that a higher value of *c* or ϕ does not necessarily imply a high bearing capacity for the samples for any of the footing shapes. The shape of footing was found to be an important factor in the variation of bearing capacity values.

For all the soil samples, the square footing was found to have the highest bearing capacity, followed by circular footing, while strip footing had the lowest bearing capacity. This could be attributed to the combined effects of different values of bearing capacity factors and coefficients of the applied bearing capacity equations. For each of the footing geometry, it was observed that Sample 2 had the least bearing capacity value, while Sample 14 had the highest value.

Table 3: Values of Shear Strength Parameters and Bearing Capacity of Soil Samples

Sample ID	Cohesion, *c* (kN/m^2)	Angle of internal friction, ϕ (^{o})	Bearing capacity (kN/m^2)		
			Strip footing	Circular footing	Square footing
Sample 1	35	30	522.61	604.85	616.23
Sample 2	34	7	83.15	105.14	105.20
Sample 3	28	24	287.16	334.80	339.90
Sample 4	96	30	1115.13	1382.67	1392.90
Sample 5	64	28	802.57	973.13	983.85
Sample 6	72	39	2508.54	2927.87	2989.29
Sample 7	39	10	124.11	156.33	156.60
Sample 8	40	30	549.30	649.61	659.46
Sample 9	18	20	136.99	159.49	161.56
Sample 10	16	28	329.75	355.36	366.55
Sample 11	59	20	335.00	418.54	420.43
Sample 12	23	31	408.98	454.38	466.18
Sample 13	55	14	227.46	286.42	287.19
Sample 14	28	44	2697.08	2791.83	2932.06
Sample 15	88	10	260.95	334.34	334.61
Sample 16	52	3	122.93	156.61	156.67

Sample 17	48	38	1850.18	2098.60	2155.11
Sample 18	92	28	1075.58	1331.06	1341.32
Sample 19	24	36	714.54	772.04	798.49
Sample 20	20	33	230.51	261.75	266.83

CONCLUSION

Assessment of bearing capacity of soils in selected sites had been done. All the soil samples were heterogeneous. Majority (80 %) of the soil samples were A-2-7 (by AASHTO classification); 70 % of the samples were well-graded sand – SW (by USCS classification). All the samples were c-ϕ soils. It could also be concluded that all the soil samples were excellent to good foundation materials; and that the values of bearing capacity were largely influenced by the nature of foundation soil and footing geometry, with the strip footings having the least values and the square footings having the highest values for each location.

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