Geotechnical Subsoil Investigation for the Design of Water Tank Foundation

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Abstract- Subsoil investigation was conducted at Unyeada in Andoni Local Government Area, in the Niger Delta for the purpose of designing a suitable foundation for a water tank structure. Field and Laboratory investigations show that the topsoil is underlain by a Loose clayey sandy Layer (about 6m thick) with Phi < 28^0, overlying a soft clay with c_u of about $12KN/m^2$. Underneath the soft clay is medium dense sandy layer with Phi between 30^0 and 32^0. The allowable bearing capacity profile of the sub-surface shows a minimum bearing Capacity of $79.8KN/m^2$. Settlement predictions based on a loading of $300KN/m^2$ indicated a settlement of 26mm. The bearing capacity analysis for the underlying soils is limited to the near surface sandy clay. In general, the sandy clay is partially saturated and when tested in unconsolidated and undrained conditions, exhibits both cohesion of 0.00kPa and angle of internal friction of $28^0$for its shear strength characteristics. However, the frictional component of shear strength is neglected for the clay encountered within normal founding depths for shallow foundations when estimating ultimate bearing pressures for the clay. The placement of a Raft foundation on a compacted granular material is suggested for the water tank stand.

Index Terms- Cone Penetrometer Test (CPT), Standard Penetration Test (SPT), Compressibility, Bearing Capacity, Shallow Foundation, Settlement

I. Introduction

The bearing capacity of shallow foundations on granular material has been studied for years by many different investigators. Although many approaches and additional considerations to the governing criteria of bearing capacity have been presented, the calculation of the ultimate bearing capacity of a footing has changed very little since Terzaghi (1943) presented his general equation for ultimate bearing capacity (qult). However, current design of shallow foundations on granular soils does not account for the absolute size of the footing, or the scale effect between the soil and the foundation. This may result in an overly conservative design, which in turn results in excessive costs of foundations. Unlike the values of Nq and Nc, the bearing capacity factor, $N\gamma$, is not a unique value, but depends on both the unit weight, γ, and the friction angle, φ of the soil. In addition to these elements, there appears to be considerable evidence that for granular materials, the bearing capacity factor ($N\gamma$) is also dependent on the absolute width of the foundation, B; that is, there appears to be a scale effect such that the value of $N\gamma$ decreases as the footing width increases, all other variables being constant. Some researchers have suggested that this phenomenon may be related to grain-size characteristics of the soil.

However, since $N\gamma$ appears to have a scale effect, and is theoretically related only to the unit weight and friction angle of the soil, it is possible that the actual scale effect may be, at least in part, related to the method of determining the friction angle. That is, it is possible that the scale effect observed between $N\gamma$ and the footing size is directly related to the confining stress felt underneath a footing, i.e., the larger the footing, the higher the confining stress and the lower the friction angle. This can be related to the curvature of the Mohr Coulomb failure envelope.

Bearing capacity failure occurs as the soil supporting the foundation fails in shear, which may involve either a general, local or punching shear failure mechanism (Bowles, 1988). For these different failure mechanisms, different methods of analyses are used. Estimation and prediction of the ultimate bearing capacity of a foundation is one of the most significant and complicated problems in geotechnical engineering (Coduto, 2001). A list of the principal contributors to the study of bearing capacity failure mechanism may include Terzaghi (1943), Hansen (1970), Vesic (1973), Chen and McCarron (1991), Lutenegger and Adams (2003) and Erickson and Drescher (2002). The focus of this work is on the estimation of soil and depth of foundation on the ultimate bearing capacity of the footing of a water tank.

II. Materials and Methods

Two methods, Cone Penetration Test and the conventional shell and auger boring were applied for this soil investigation.

Cone Penetration Test (CPT)

Hydraulically operated GMF type of static penetrometer, 100KN capacity was used in the cone resistance soundings. Mechanical mantle cone with friction jacket and discontinuous sounding procedure was adopted in the test. The cone in its retracted position is first forced into the ground a distance of 10cm by the application of force to the outer sounding tubes. The cone is then pushed out a distance of about 4cm by the application of force to the inner rods only and the magnitude of the force required to achieve this is measured on the pressure gauges (cone resistance) and recorded. With this, two CPT probes were made each to a depth of 20m as shown in figures 1a &1b.

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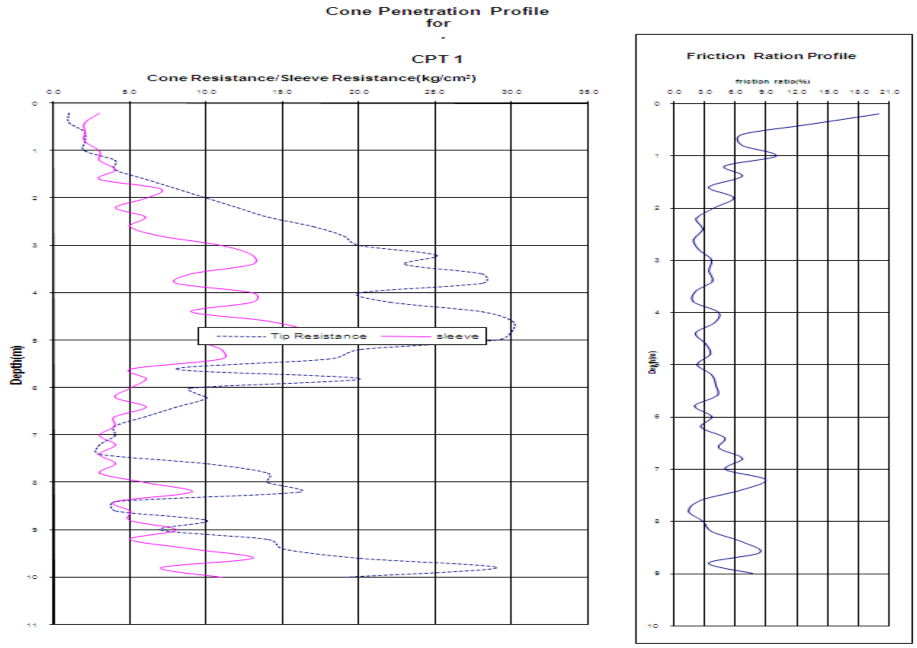

Figure 1a: CPT Probe through the soil at the site

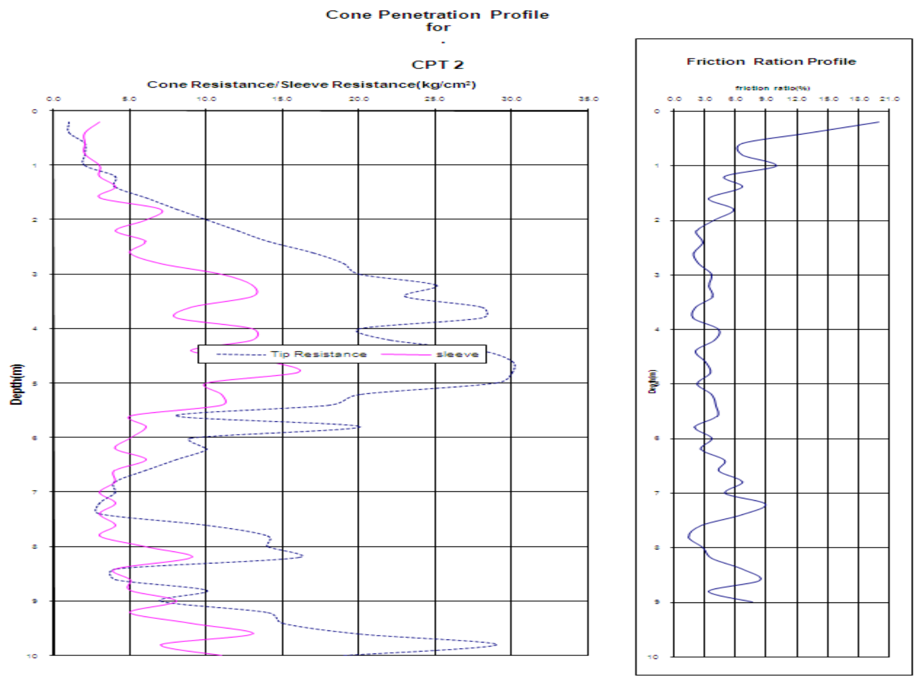

Figure 1b: CPT Probe through the soil at the site

Soil Borings

Conventional boring method which consists of the use of the light shell and auger hand rig was used in the boring operations. During the boring operations, disturbed samples were regularly collected at depths of 0.75m intervals and also when change of soil type is noticed. Undisturbed cohesive soil samples were retrieved from the boreholes with conventional open-tube sampler, 100mm in diameter and 450mm in length. The open-tube sampler consists essentially of a lower end and upper end screwed into a drive head which is attached to the rods of the rig. The head has an overdrive space and incorporates a non-return valve to permit the escape of air or water as the samples enters the tube. The sampler is driven into the soil by dynamic means using a drop hammer. On withdrawal of the sampler, the non-return valve assists in retaining the sample in the tube. All samples recovered from the boreholes were examined, identified and roughly classified in the field.

Standard Penetration Tests (SPT) is performed at every 1.5m advance through cohesionless soils. The main objective of this test is to assess the relative densities of the cohesionless soils penetrated. In this test, a 50mm diameter split spoon sampler is driven 450mm into the soil with a 63.5kg hammer falling freely at a distance of 760mm. The sampler is driven into the soil in three stages. The initial 150mm penetration of the sampler is regarded as the seating drive, while the first 300mm and the last 300mm penetration are the test drives. The number of blows required to effect the last 300mm penetration below the seating drive provides an indication of the relative density of the cohesionless soil stratum being tested. This is also referred to as the N-value. The penetration resistances in blow counts with depth are indicated on the borehole logs in figure 2.

The SPT blow count is correlated to the friction angle based on Schmertmann (1975) and Hatanaka and Uchida (1996). These methods are based on actual data and taken into account the effect of the overburden stress.

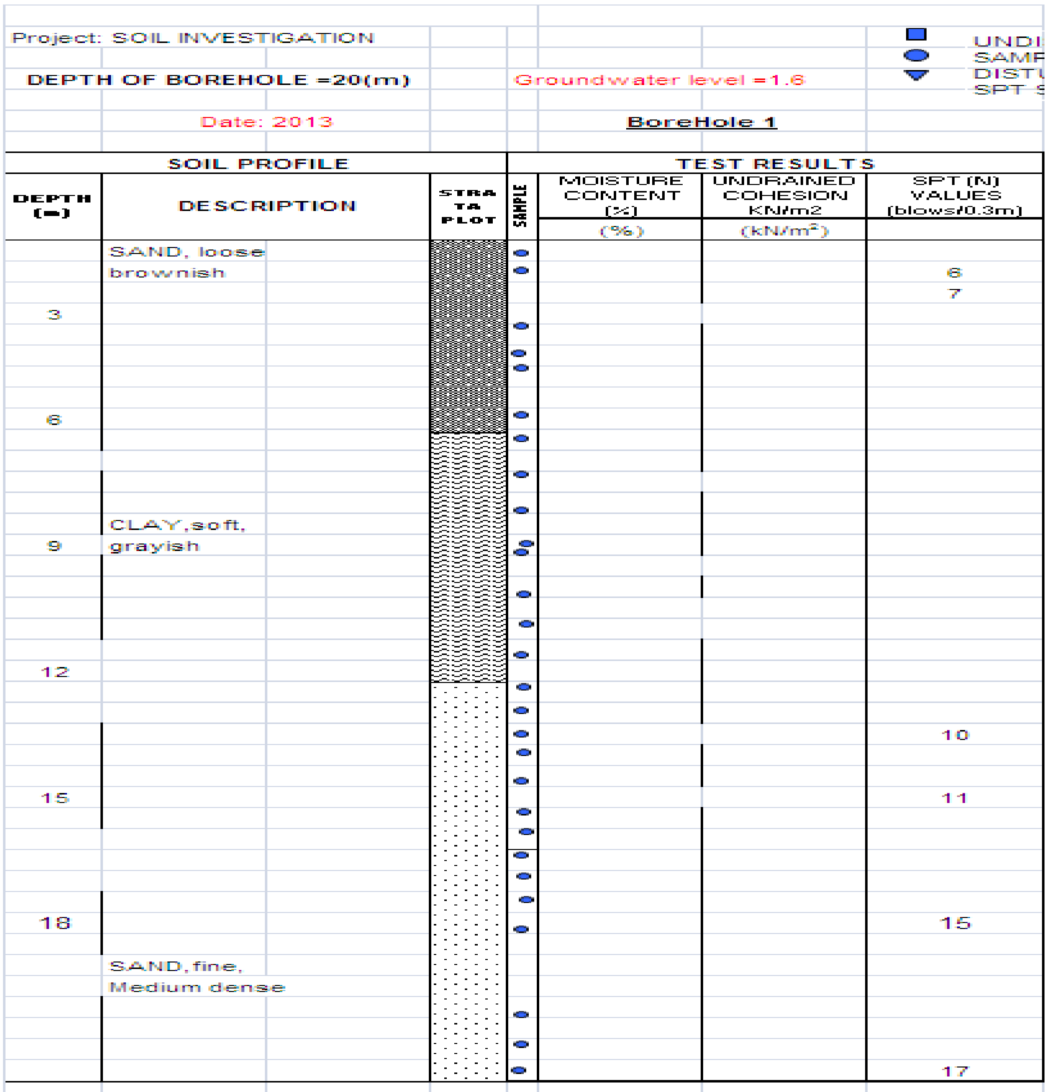

Figure 2: Borehole Logs and SPT blow counts

III. RESULTS AND DISCUSSION

Stratigraphy and Engineering Properties of the Soil

Data from the soil sampling, standard penetration tests, cone resistance soundings and laboratory tests were carefully evaluated for the determination of the stratification of the underlying soils. The evaluation uncovered three primary soil profiles characterizing the site as follows:

(i) clayey Sandy Layer encountered from surface to 6.0m depth
(ii) Soft grayish clay encountered from 6.0m to 12.0m depth
(iii) Medium dense sandy layer encountered from 12.0m to 20.0m depth where the investigation ended.

The stratigraphy beneath the site showed significant uniformity in both the two borings and the two cone penetrometer tests with water table at 1.6m below ground level. The ranges of thicknesses of the different strata are shown in the strata logs in figure 2.

Classification, strength and compressibility characteristics of the soils were determined from the laboratory and in-situ tests. The relevant index and engineering parameters of the soils are summarized below. Details of these are presented in tables 1, 2, 3, 4, 5 and figures 3, 4 and 5 of this report.

Clayey Sandy Layer

The loose clayey sandy layer is found to have low compressibilty and brownish in colour with average CPT value of 8.0kg/cm^2. The ranges of variations in the relevant index and engineering parameters of the sandy layer are summarized below:-

Average effective particle size, d_{10} (mm) is 0.13; Mean particle size, d_{50} (mm) is 0.21; Coefficient of uniformity, $Cu=d_{60}/D_{60}$ is 1.8; Coefficient of curvature, $Cc= d^2_{30}/ D_{10}$-d_{60} is 1.0.

Soft Clayey Layer

The clay has a property of high compressibility with M_v values >0.4m^2/MN and grayish in colour. The ranges of variations in the relevant index and engineering parameters of the clay are summarized in tables 3a and 3b.

Medium Dense Sandy Layer

Underlying the lower soft clay is a layer of predominantly well sorted, medium dense sand. The ranges of variations in the relevant engineering parameters of this layer are given as follows: Effective particle size, d_{10} (mm) is 0.17; Mean particle size, d_{50} (mm) is 0.41; Coefficient of uniformity, $Cu=d_{60}/D_{60}$ is 2.9; Coefficient of curvature, $Cc= d^2_{30}/ D_{10}$-d_{60} is1.0.

Table 1: Allowable Bearing Capacities for shallow foundations (Water depth < foundation Depth)

Foundation Depth (m)	Width (m)	Undrained Shear Strength (KN/m^2)	Ultimate Bearing Pressure (KN/m^2)			Allowable Bearing Pressure (KN/m^2)		
			L/B=1	L/B=1.5	L/B=5	L/B=1	L/B=1.5	L/B=5
1.5	1	0	239.4	243.6	249.48	79.80	81.20	83.16
1.5	1.5	0	264.6	270.9	279.72	88.20	90.30	93.24
1.5	2	0	289.8	298.2	309.96	96.60	99.40	103.32
1.5	2.5	0	315	325.5	340.2	105.00	108.50	113.40
1.5	5	0	441	462	491.4	147.00	154.00	163.80
1.5	10	0	693	735	793.8	231.00	245.00	264.60
2	1	0	302.4	306.6	312.48	100.80	102.20	104.16
2	1.5	0	327.6	333.9	342.72	109.20	111.30	114.24
2	2	0	352.8	361.2	372.96	117.60	120.40	124.32
2	2.5	0	378	388.5	403.2	126.00	129.50	134.40
2	5	0	504	525	554.4	168.00	175.00	184.80
2	10	0	756	798	856.8	252.00	266.00	285.60

Bearing Capacity

The conventional method of foundation design is based on the concept of bearing capacity or allowable bearing pressure of the soil. The bearing capacity is defined as the load or pressure developed under the foundation without introducing damaging movements in the foundation and in the superstructure overlying the foundation.

Damaging movements may result from foundation failure or excessive settlement. The two criteria used in the design of foundation are therefore:

(a) Determination of bearing capacity of soil and the selection of adequate factor of safety, usually not less than 2.5

(b) Estimating the settlement under the expected load and comparison with the permissible settlement

Choice of Parameters:

In clays, the ultimate bearing capacity of spread foundation is calculated using total stress parameters. This gives the end-of-construction case, which is the worst condition, and allows the design to be based on undrained shear strength tests.

The bearing capacity analysis for the underlying soils is limited to the near surface sandy clay. In general, the sandy clay is partially saturated and when tested in unconsolidated and undrained conditions, exhibits both cohesion and angle of internal friction for its shear strength characteristics (tables 3a and 4). However, the frictional component of shear strength is neglected for the clay encountered within normal founding depths for shallow foundations when estimating ultimate bearing pressures for the clay. Undrained cohesion of 0.00kPa and angle of internal friction of 28^0 are adopted for the bearing capacity analysis. Therefore, the placement of a Raft foundation on a compacted granular material is suggested for the water tank stand.

Settlement of Shallow Foundation

The Burland and Burbdige (1984) method for settlement criteria was considered in which the average N value over the depth of influence below the footing, approximately, 1.5 times the width of the foundation was used. Considerations were taken to accommodate corrections made in the SPT blow counts according to the recommendations of Youd et al (2001) and Mayne (2001), for the type of sampler, the rod length, the borehole diameter, the energy transmitted to the sampler and the overburden stress.

Settlement of shallow foundation for net foundation load of $\Delta\sigma$ **=100**kPa was then calculated.

Table 2a: Settlements Parameter

Clay	**6.0m (normally consolidated)**
e_o	**1.183**
Preconsolidation Pressure	**28KPa**
Cc	**0.17**
Soil Compressibility based on Cc and e_o	**0.077**

Table 2b: Computed Rate of Settlements

Rate of Settlements	**Years**
T_{50}	**0.3**
T_{90}	**1.4**

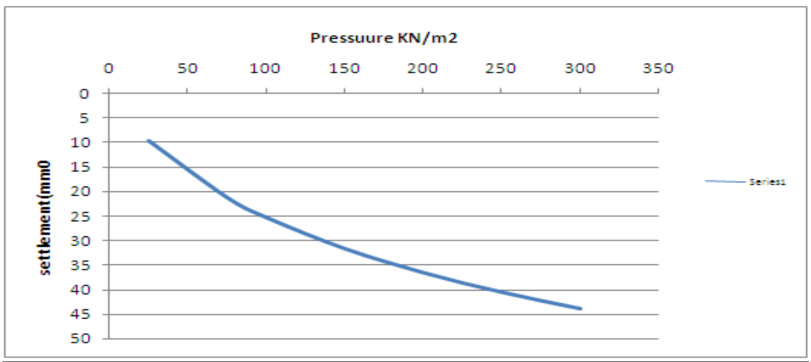

Figure 3a: Load Settlement Curve for Clay from 6.0m depth

Table 2c: Settlements Parameter

Clay	**10.0m (normally consolidated)**
e_o	**1.89**
Preconsolidation Pressure	**60KPa**
Cc	**0.5**
Soil Compressibility based on Cc and e_o	**0.173**

Table 2d: Computed Rate of Settlements

Rate of Settlements	**Years**
T_{50}	**0.173**
T_{90}	**1.0**

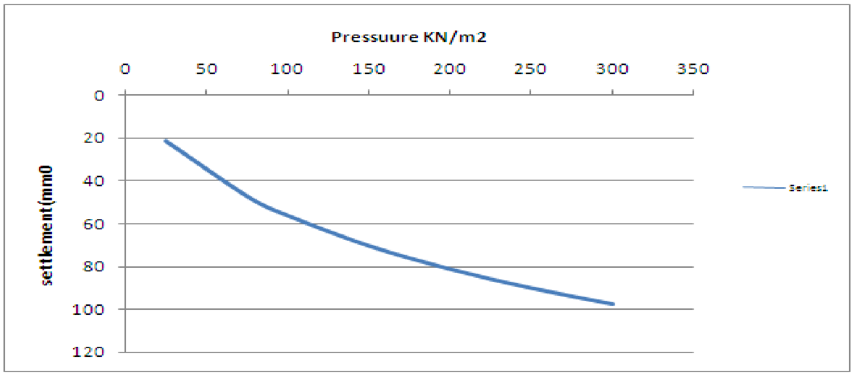

Figure 3b: Load Settlement Curve for Clay from 10.0m depth

Table 2e: Showing Variation of Settlement with foundation Pressure

Sand	**1.5m**
Load(KN/m^2)	**300**
Elastic Modulus	**12000KPa**
Influence factor	**0.5**
Cone Value (Kg/cm^2)	**6**
P_i (elastic)	**26**

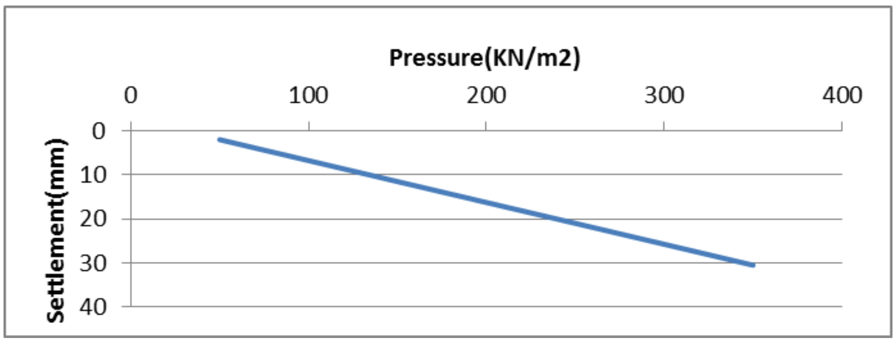

Figure 3c: Load Settlement Curve for Sand from 1.5m depth

TABLE 3a: **CLASSIFICATION TEST (ATTERBERG LIMITS)**

Borehole No.	**Depth sampled (m)**	**Liquid Limit (%)**	**Plastic Limit (%)**	**Plasticity Index**	**Sat. Unit Weight\ γ (KN/m^3)**
1	6	63	33	30	14
1	6	47	17	30	14

TABLE 3b: CONSOLIDATION (ONE–DIMENSIONAL) COMPRESSIBILITY TEST

Bore-Hole Nos	**Depth (m)**	**Pressure Range (Kpa)**	**Coefficient of Consolidation Cv(m^2/yr)**	**Coefficient of Volume Compressibility Mv 10^{-4}**	**Coefficient of Permeabilty K 10^{-8}cm/s**
1	6m	0-25	1.4892	3.100000	1.43E-8
		25-50	1.4892	5.321240	2.46E-8
		50-100	1.4892	5.985394	2.77E-8
		100-200	1.441161	4.974993	2.23E-8

		200-400	1.441161	0.180055	8.06E10
		400-800		0.556019	2.49E-9

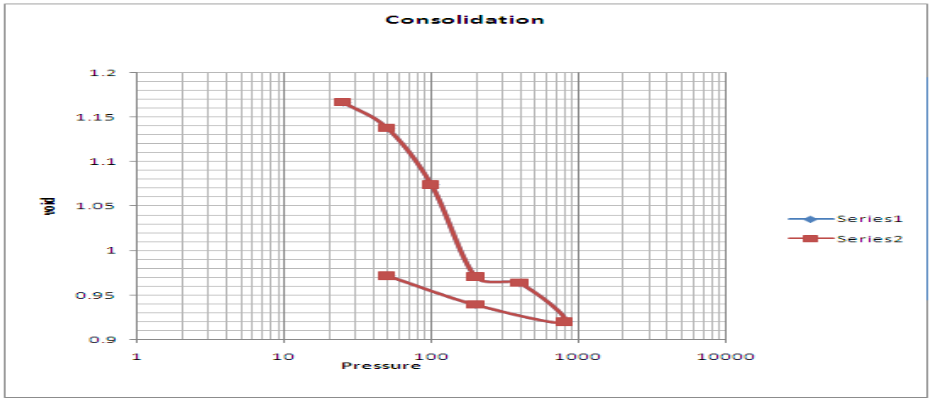

Figure 4a: Plot of Consolidation Test

TABLE 3c: CONSOLIDATION (ONE–DIMENSIONAL) COMPRESSIBILITY TEST

Bore-Hole Nos	Depth (m)	Pressure Range (Kpa)	Coefficient of Consolidation C_v (m^2/yr)	Coefficient of Volume Compressibility, M_v 10^{-4}	Coefficient of Permeabilty, K 10^{-8}cm/s
1	10m	0-25	2.127428	3.400000	2.25E-08
		25-50	2.127428	3.630862	2.4E-08
		50-100	2.127428	5.210178	3.44E-08
		100-200	1.942434	4.321746	2.61E-08
		200-400	1.942434	2.785211	1.68E-08
		400-800	1.942434	1.859349	1.12E-08

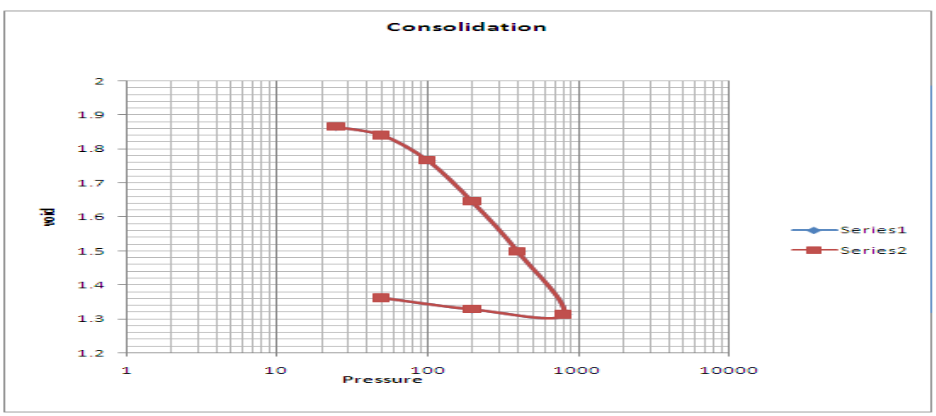

Figure 4b: Plot of Consolidation Test

TABLE 4: UNDRAINED TRIAXIAL COMPRESSION TESTS

Bore-Hole No	Depth Sampled (m)	Natural Moisture Content (%)	Undrained Cohesion (KN/m^2)	Friction angle ϕ(Degree)	USCS
1	6	53	28	2	CH
1	10	47	12	2	CH

Table 5a: Triaxial Test (BH 1, 6.0m)

Minor Principal Stress	100KN/m^2	300KN/m^2
Deviator Stress	64KN/m^2	78KN/m^2
Major Principal Stress	164KN/m^2	378KN/m^2

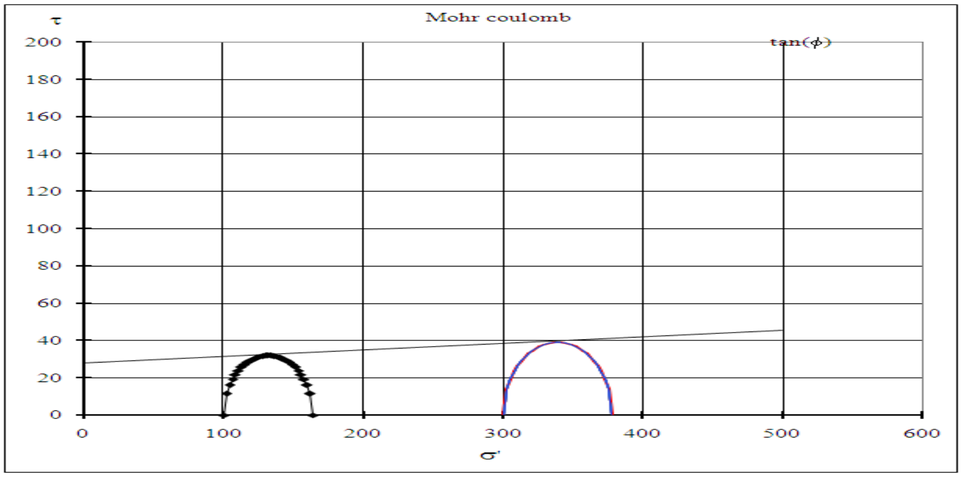

Figure 5a: Mohr –Coulomb Failure Envelop for Sample from 6m

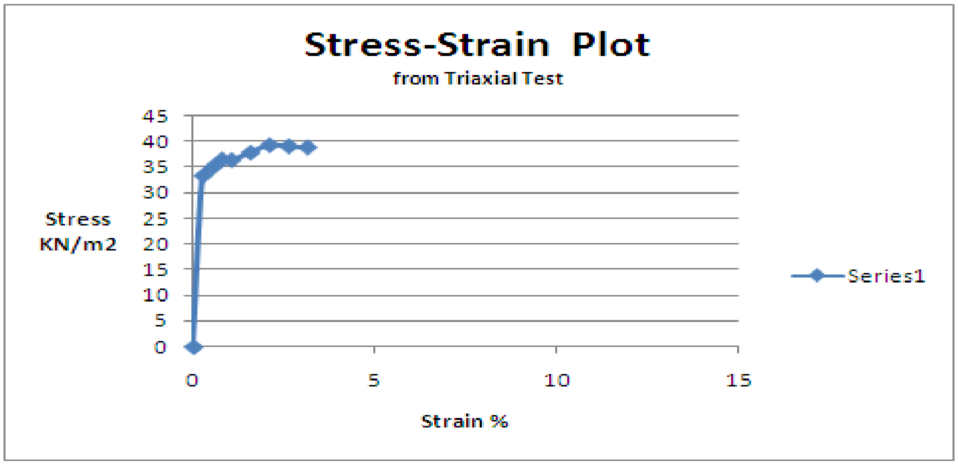

Figure 5b: Direct Shear Test Displacement Curve for Sample from 6m

Table 5b: Triaxial Test (BH 1, 10.0m)

Minor Principal Stress	100KN/m^2	300KN/m^2
Deviator Stress	31KN/m^2	47KN/m^2
Major Principal Stress	131KN/m^2	347KN/m^2

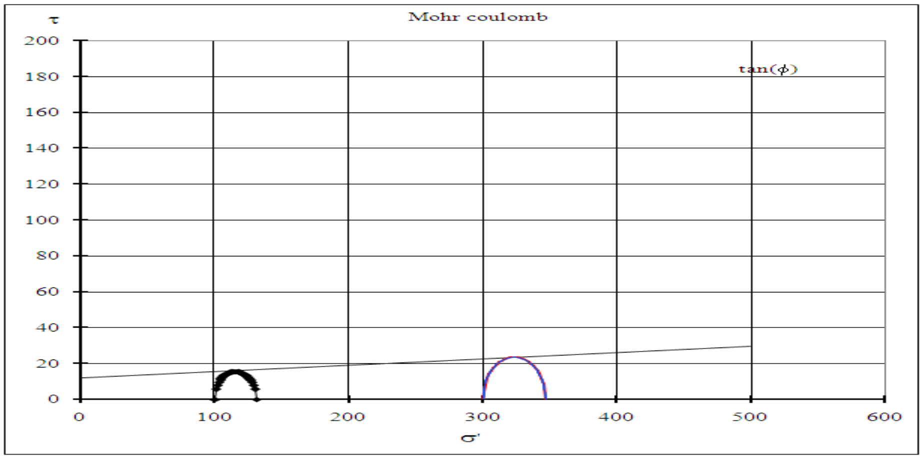

Figure 5c: Mohr –Coulomb Failure Envelop for Sample from 10m

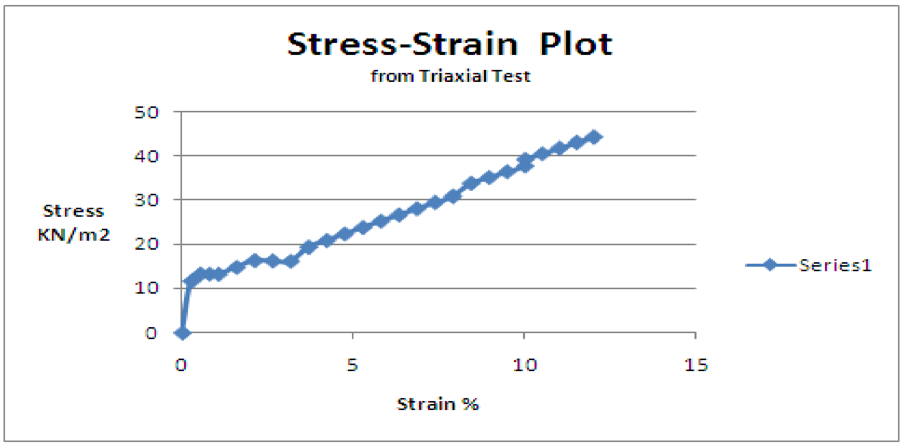

Figure 5d: Direct Shear Test Displacement Curve for Sample from 10m

IV. CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations reached in this report are based on the data obtained from the soil borings, in-situ and laboratory tests performed. It is not envisaged that soil conditions will vary significantly from those described. However, some variations are possible. The extent of the variations in the stratigraphy across the site may not be evidenced until construction commences. Should the stratigraphy vary, it will be necessary to evaluate the engineering significance of such variations that could result in further investigation and supplementary recommendations.

Field and Laboratory investigations show that the topsoil is underlain by a Loose clayey sandy Layer (about 6m thick) with Phi < 28^0, overlying a soft clay with c_u of about $12KN/m^2$. Underneath this layer is Medium Dense Sandy Layer with Phi between 30^0 and 32^0. The allowable bearing capacity profile of the sub-surface shows a minimum bearing Capacity of $79.8KN/m^2$. Settlement predictions based on a loading of $300KN/m^2$ indicated a settlement of 26mm (figure 3c).

REFERENCES

[1] Bowles, J.E. (1988): Foundation Analysis and Design. McGraw-Hill Book Co.

[2] Burland, J.B., Burbidge, M.C. (1984), Settlement of foundations on sand and gravel, Proceedings of the Institution of Civil Engineers, Part 1, 1985, 78, Dec., 1325-1381.

[3] Chen, Wai-Fah and McCarron, W.O. (1991): "Bearing Capacity of Shallow Foundations," Foundation Engineering Handbook, 2nd ed., Van Nostrand Reinhold, New York, pp. 144-165.

[4] Coduto, D.P. (2001): Chapter 6: Shallow Foundations- Bearing Capacity. Foundation Design: Principles and Practices. 2nd ed. pp. 170-206.

[5] Erickson, H.L. and Drescher, A. (2002): Bearing Capacity of Circular Footings. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 1, pp. 38-43.

[6] Hansen, J.B. (1970): A revised and extended formula for bearing capacity. Geoteknisk Inst., Bulletin 28, pp. 5-11.

[7] Hatanaka, M., Uchida, A. (1996). Empirical correlation between penetration resistance and effective friction of sandy soil. Soils & Foundations, Vol. 36 (4), 1-9, Japanese Geotechnical Society.

[8] Lutenegger, A.J. and Adams, M.T. (2003): Characteristic Load-Settlement Behavior of Shallow Foundations. Proceedings of the FONDSUP International Symposium, Paris, France. Vol. 2

[9] Mayne, P.W. (2001), Geotechnical site characterization using Cone, piezocone, SPTu, and VST, Civil and Environmental Engineering Department, Georgia Institute of Technology

[10] Schmertmann, J.H. (1975), Measurement of insitu shear strength, keynote lecture, Proceedings of the conference on in-situ measurement of soil properties, June 1-4, 1975, vol. II, American Society of Civil Engineers.

[11] Terzaghi, K. (1943): Theoretical Soil Mechanics. John Wiley & Sons, Inc., New York.

[12] Vesic, A.S. (1973): Analysis of ultimate loads of shallow foundations. ASCE Journal of theSoil Mechanics and Foundation Division, Vol. 99, No.1, pp. 45-73.

[13] Youd, T. L et al (2001): 'Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journ. of Geotech. Geoenviron. Eng., ASCE, 127(10), 817-833.

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