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Research article

Estimation of Stability and Deformation of PMS Tank Foundation Placed on Cohesionless Soil Formation: A Case Study in Lekki, Lagos of Nigeria

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Abstract

The stability and deformation of vehicular petrol (PMS) tank foundation founded on slightly silt SAND lithology in Lekki area of Lagos State, Nigeria was carried out. The study entailed field borings, standard penetration test sounding, laboratory tests and analysis of soil samples.Results of allowable bearing capacity increased with foundation depth and had a minimum value of 110kN/m^2 at 3m depth using modified Meyerhof's approach, beyond which it increased with depth. Induced vertical stress values with depth were lower than those of allowable bearing capacity.In Harr's approach, immediate settlement increased with increase in thickness of compressible stratum, with almost a constant rate of increase in settlement. In Burland and Burbidge approach, immediate settlement decreased with increase in foundation depth. A maximum immediate settlement of 27mm was evaluated in Harr'sapproach for 9m compressible thickness below metal plate-soil interface, while in Burland and Burbidge approach, 30mm immediate settlement occurred at 1m depth.Total maximum settlement was generally about 61mm for both approaches.

Key words: Bearing Pressure. Induced vertical stress.Poisson ratio.Elastic modulus. Influence factor

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Introduction

A vehicularpetrol (PMS) storage tank designed to have a floating roof type was scheduled for rehabilitation by increasing tank diameter from 32.9m diameter to 48.8m diameter and 14.4metres height after several decades of operation. Usually, these tanks are commonly designed to have flexible foundation and bearing pressure of the petroleum product is transmitted to the ground through metal plates on granular overburden layers to the underlying soil formation while the elevated tank metal sheets, most often rest on concrete ring beam. When fully operational, the PMS generates a bearing pressure of about 107kPa on the metal plates- soil interface for vehicular PMS with unit weight of 7.37kN/m^3. This tank was founded on a soil lithology consisting of slightly silty SAND up to the depth of investigation for rehabilitation purposes only. These structures are settlement controlled and the magnitude of tolerable settlement the super structure can sustain controls the design of such shallow foundations placed on sand. This makes settlement prediction very vital and many semi empirical methods have been presented by scholars in its evaluation (Nova and Montrasio,1991).Studies on crude oil tanks founded on made up granular soils underlain by cohesive soils have been reported by Akpila (2007), Akpila and Ode (2008), among others in the Niger Delta Region of Rivers State. This paper attempts to evaluate both the stability and deformation characteristics of the PMS tank placed on sand using informations obtained from borings.

Materials and Methods

Field Exploration/ Laboratory Analysis

Subsurface conditions at the site were studied through ground borings to depths of 15m using a percussion boring rig, Cone penetration tests and Resistivity tests with test points shown in Figure 1. Both disturbed and undisturbed samples from borings were collected for visual examination, laboratory testing and classification while standard penetration tests (SPT) were also conducted to determine the penetration resistance values of cohesionless formations within the boreholes. Requisite laboratory tests on soil samples to obtain input parameters for stability and deformation assessment were subsequently conducted. The static water table varied from about 1.7-1.8.0m depth below the existing ground level

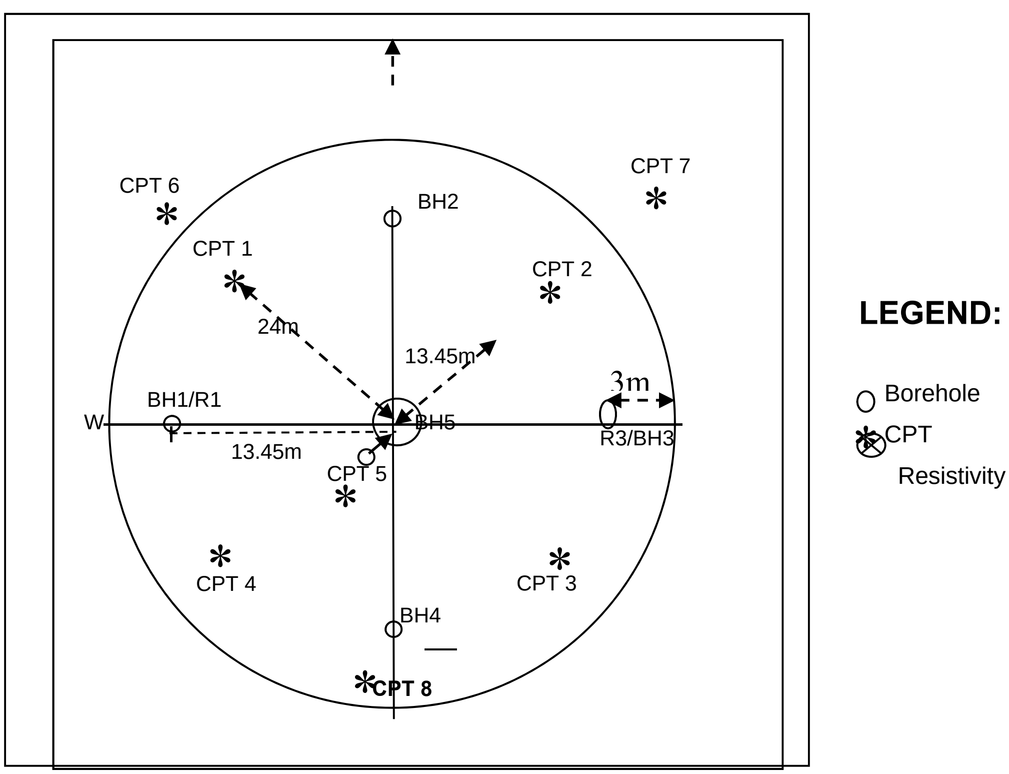

Figure 1: Borings, CPT and Resistivity points

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Bearing Capacity of Shallow Foundation

SPT Approach

In Kong and Yang (1991), they emphasized that two modes of tank foundation failures have been observed in practice, namely, base and edge shear failures. In base shear failure, the entire tank act as a unit, while in edge shear failure, local failure of a portion of the tank perimeter and contiguous portion of tank base occur. Since the site subsurface consists of cohesionless soil formation, the modified Meyerhof (1956) correlation for bearing capacity using Standard Penetration Resistance approach presented by Bowles (1977) was used to analysis the 48.8m diameter PMS circular tank foundation. The choice of modified Meyerhof method is based on the middle bound values associated with the model compared to that of Parry (1977) with higher bound values and Meyerhof (1956) with lower bound values of bearing capacity (Akpila, 2013). The modified Meyerhof expressions are given by;

$$q_{n(a)} = 19.16NF_d\left(\frac{S}{25.4}\right) \qquad for\ B \leq 1.2m \tag{1}$$

$$q_{n(a)} = 11.98N\left(\frac{3.28B+1}{3.28B}\right)^2 F_d\left(\frac{S}{25.4}\right) \quad for\ B > 1.2m \tag{2}$$

Where F_d= depth factor = 1+ 0.33 (D_f/ B) ≤ 1.33 (3)

S = tolerable settlement

B = foundation breadth

N = average penetration number

$q_{n(a)}$ = net allowable bearing capacity

Stress Analysis

The containment of the PMS of about 48.8m diameter and 14.4metres height is transmitted to the ground by metal plates placed on compacted soil. A unit weight of 7.37kN/m^3 for vehicular petrol is adopted and at full capacity under static loading, a bearing pressure of approximately 107kN/m^2 is transmitted to the ground through the metal plate. The induced vertical stress ($\Delta\sigma_z$) with depth from the PMS load is obtained from the expression (Craig, 1987);

$$\Delta\sigma_z = q\left\{1 - \frac{1}{\left[1+\left(\frac{a}{z}\right)^2\right]}3/2\right\}(4)$$

where $\Delta\sigma_z$= induced vertical stress

q = applied stress

a = radius of circular area

z = depth of interest

The variation of induced vertical stress with depth at centre of tank is shown in Figure 2.

Settlement Analysis on Sand

Immediate Settlement

Immediate settlement has been computed for a circular foundation of 48.8m diameter having an equivalent breadth of 43.24m.The immediate foundation settlement at centre of the flexible foundation on sand is obtained from the expression proposed by Harr (1966) and reported in Braja (1999) as follows;

$$s_i = \frac{q_n}{E_o} B(1 - \mu^2) I_p \tag{5}$$

where S_i is immediate settlement, B is equivalent breadth of foundation at a corner, q_n is net foundation pressure, E_o is modulus of elasticity, μ is Poisson ratio and I_p is influence factor.

The value of E_o is obtained from the expression;

$$E_o = 0.478N + 7.17MPa \tag{6}$$

and for cohesionless soils, Poisson ratio, μ, can be evaluated from;

$$\mu = \frac{1-\sin\phi}{2-\sin\phi} (7)$$

where ϕ is angle of internal friction of sand and N is average SPT blow count for sand stratum. Values of influence factor, I_p, for various length to breadth (L/B) ratios were obtained from standard curves presented in Braja (1999). In Burland and Burbidge (1985) approach, they proposed that for normally consolidated sand, the average settlement is expressed in terms of net foundation pressure, foundation breadth and compressibility index as;

$$s_i = \frac{q_n B^{0.7}}{3} \left(\frac{1.71}{N^{1.4}}\right) \tag{8}$$

where q_n is the net foundation pressure, B is foundation breadth and N is average value of standard penetration resistance.

Consolidation Settlement

While settlement on sand is generally treated as immediate, the consolidation settlement was carried out using Equations (12) and the coefficient of volume compressibility, m_v, is obtained from the following expression;

$$m_v = \frac{(1+\mu)(1-2\mu)}{E_o(1-\mu)} \tag{9}$$

where $E_o$and μ and are as defined in Equations (6 and 7) and the consolidation settlement was evaluated from Skempton and Bjerrum (1957) expression presented as follows:

$$\rho_c == \frac{\Delta e}{1+e_o}\left(\frac{1}{\Delta p}\right)\Delta\sigma_z H$$

$$= \frac{\Delta e}{1+e_o}\left(\frac{1}{\Delta p}\right)\frac{q_{nBL}}{(B+Z)(L+Z)} H, \text{ or} \tag{10}$$

$$= m_v \frac{q_n B^2}{(B+Z)^2} H \tag{11}$$

where ρ_c is consolidation settlement, q_n is net foundation pressure, B is foundation breadth, Δp is change in pressure, Δe is change in void ratio, e_o is initial void ratio, $\Delta\sigma_z$ is induced vertical stress and $\frac{\Delta e}{1+e_o}\left(\frac{1}{\Delta p}\right)$ is coefficient of volume compressibility, m_v. Substituting Equation (9) into Equation (11) yields;

$$\rho_c = \frac{(1+\mu)(1-2\mu)}{E_o(1-\mu)} \frac{q_n B^2}{(B+Z)^2} H \tag{12}$$

The total settlement from pad foundation can then be expressed as;

$$\rho_t = \frac{q_n B}{E_o}(1-\mu^2)I_p + \frac{(1+\mu)(1-2\mu)}{E_o(1-\mu)}\frac{q_n B^2}{(B+Z)^2}H \quad (13)$$

When immediate settlement is considered based on Equation (8), then for normally consolidated sand, total settlement can be expressed as;

$$\rho_t = \frac{q_n B^{0.7}}{3}\left(\frac{1.71}{N^{1.4}}\right) + \frac{(1+\mu)(1-2\mu)}{E_o(1-\mu)}\frac{q_n B^2}{(B+Z)^2}H \quad (14)$$

Limiting values for allowable settlement of different structures founded on either clay or sand have been specified by scholars including Skempton and MacDonald (1956), Polshin and Tokar (1957) and Wahls (1981).

Discussion of Results

Soil Classification/Stratification

The non-plastic (Np) or granular soil samples of Borehole1 (BH) 1 to BH 5, which were obtained from borings to a depth of 15m each, were analysed by dry sieving. Generally, the soil consists of medium- dense, brown, slightly silty SAND.

Stress Analysis

The variation of induced vertical stress from PMS product with depth at centre of tank foundation is depicted in Figure 2 and it is generally found to be within the net allowable bearing capacity of the soil. The predictive model given by Equation (15) can be used to evaluate the induced vertical stress at any desired depth beneath centre of the tank foundation.

$$\Delta\sigma_z/q = 0.285(z/a)^3 - 0.800(z/a)^2 + 0.16\,z/a + 0.991 \quad (15)$$

where $\Delta\sigma_z$= induced vertical stress, a= radius of tank, z = depth and q = net foundation pressure.

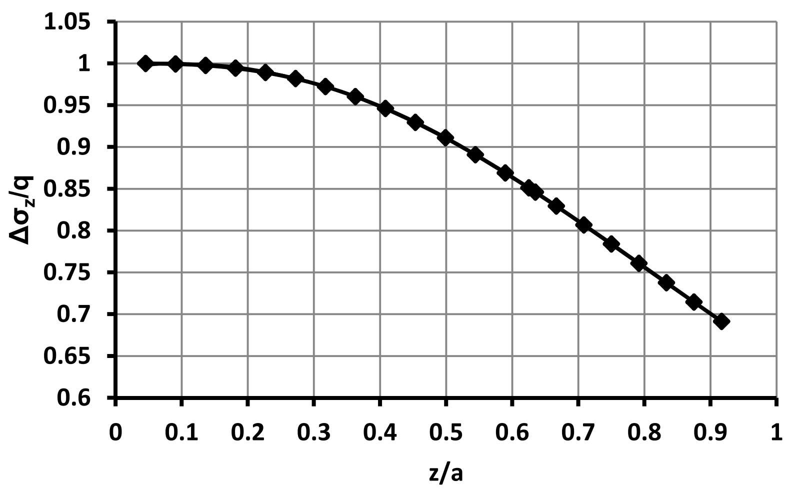

Figure 2: Induced vertical stress distribution beneath tank centre

Shear Strength Parameters

The shear strength parameter ϕ, of the cohesionless soil formations were evaluated from in-situ values of Standard Penetration Test (SPT) of the respective stratum or layer of interest. Details on SPT and ϕ values are presented in Table 1.

Table 1: SPT values with Depth

Borehole No.	Depth (m)	SPT ' N' Value
1	1	12
	3	17
	5	19
	7	23
	9	28
2	1	13
	3	16
	5	20
	7	26
	9	30
3	1	11
	3	17
	5	19
	7	23
	9	27
4	1	16
	3	2
	5	10
	7	13
	9	20
5	1	11
	3	17
	5	17
	7	21
	9	22

Bearing Capacity of Shallow Foundation

SPT Approach

The allowable bearing capacity values are presented in Table 2 and the variation of bearing capacity with foundation depth is depicted in Figure 3. Net allowable bearing capacity increased with foundation depth, but at BH4, allowable bearing capacity,$q_{n(a)}$, decreased with depth up to 3m depth, attaining a value of about 110 kN/m^2. Beyond which allowable bearing capacity increased with depth. Values of induced vertical stress were lower than allowable bearing capacity values at foundation depths of up to 9m. At a tolerable settlement of 25.4mm, bearing capacity values exceeded induced bearing pressures from PMS, hence satisfying stability requirement.

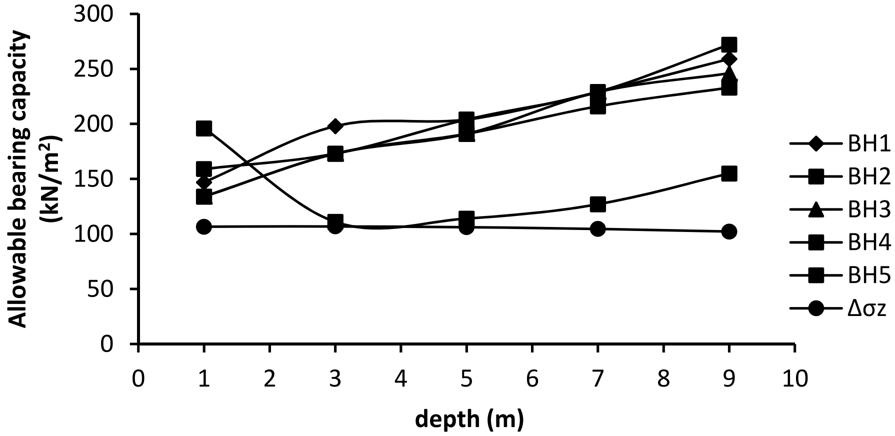

Figure 3: Variation of Allowable bearing capacity, induced vertical stress with foundation depth

Settlement Analysis

Immediate Settlement on Sand

The immediate settlement at centre of foundation with an equivalent breadth of 43.24m is obtained from Equations (5) and (8) respectively and is presented in Table 3.In Harr's approach, immediate settlement increased with increase in thickness of compressible stratum, with almost a constant rate of increase in settlement as shown in Figure 4. Soils around BH 4 had higher immediate settlement values compared to other sections within tank vicinity, portraying likelihood of localized settlement of bottom plate resulting from highly compressible formation. Whereas in Burland and Burbidge approach,, immediate settlement decreased with increase in foundation depth (Figure 5). However, the differences in settlement pattern can be attributed to application of the models; in Harr's approach foundation depth is assumed to be at 1m depth and immediate settlement is evaluated with varying compressible depths. In Burland and Burbidge approach, settlement is inversely proportional to SPT value N and the average N values were generally found to increases with depth. Presence of localized settlement around BH4 is also noticed in Burland and Burbidge approach. A maximum immediate settlement of 27mm was obtained in Harr's approach for 9m compressible thickness below metal plate-soil interface, while in Burland and Burbidge approach, 30mm immediate settlement occurred at 1m depth.

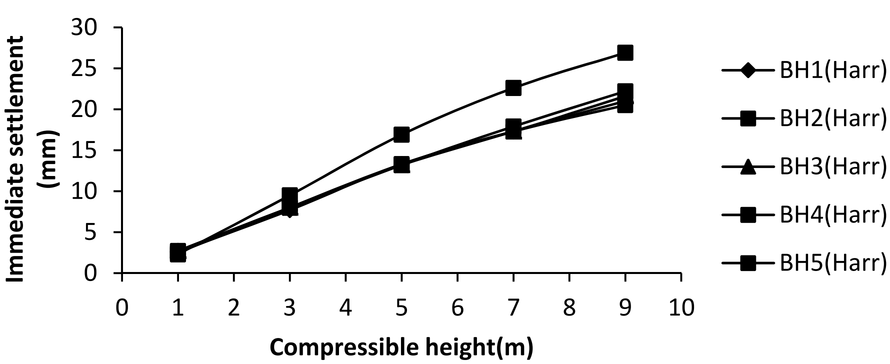

Figure 4: Variation of Immediate settlement with compressible height (Harr's Approach)

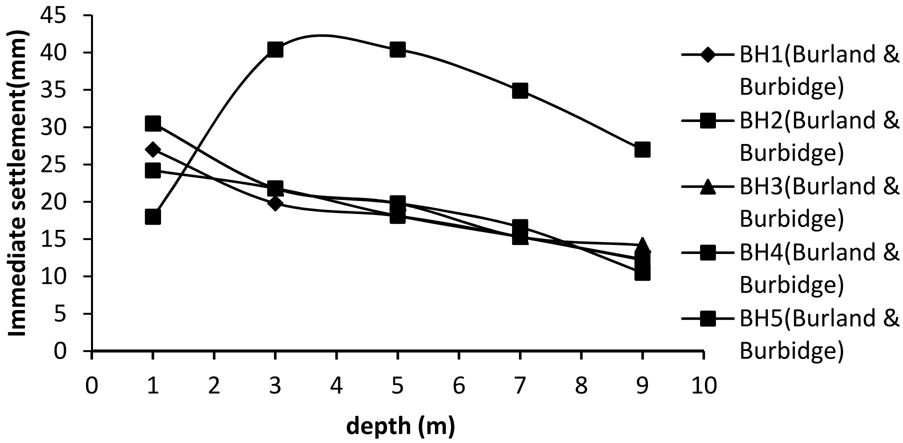

Figure 5: Variation of Immediate settlement with depth (Burland& Burbidge Approach)

Total Settlement on Sand

The maximum total settlement given by the Equations (13 and 14) is evaluated from Table 3, while variation of total settlement with depth is depicted in Figures 6 and 7. Harr'sapproach showed increase in total settlement with compressible soil thickness and soils around BH 4 had higher total settlement values beginning from 3m depth compared to other sections within tank vicinity. Total maximum settlement was generally about 61mm for both approaches.

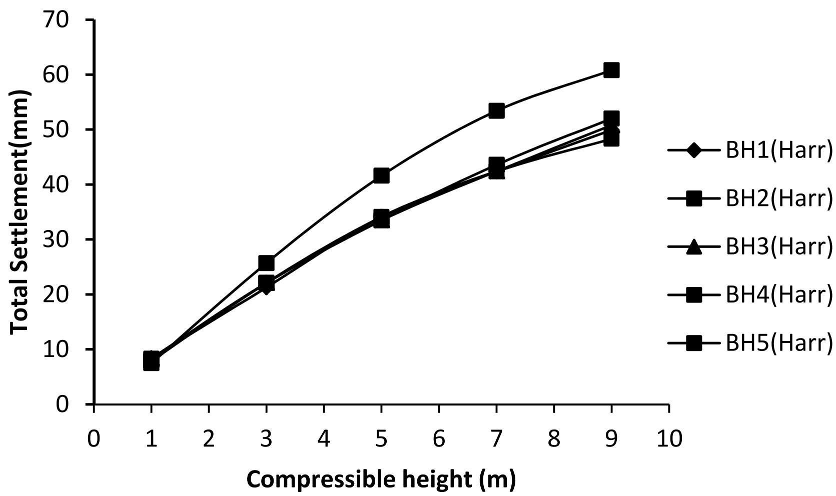

Figure 6: Variation of total settlement with compressible height (Harr's Approach)

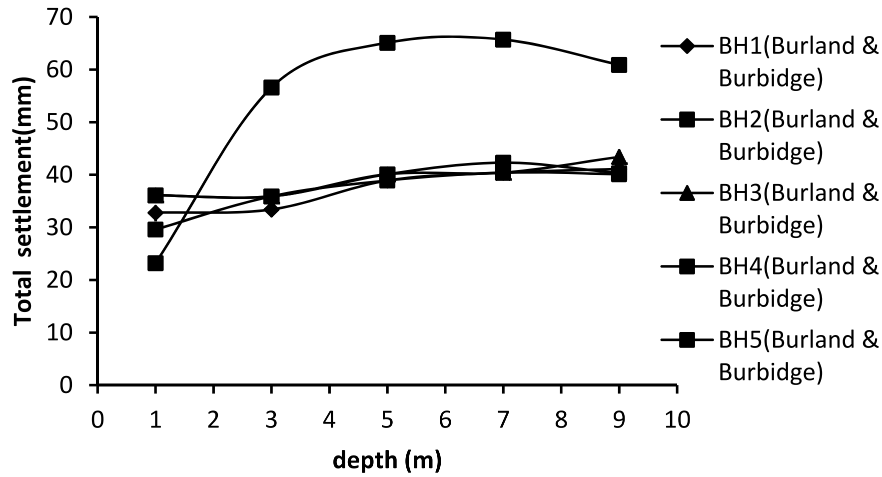

Figure 7: Variation of total settlement with depth (Burland& Burbidge Approach)

Conclusion/ Recommendation

Based on the findings, the following conclusions can be drawn;

i. The variation of induced vertical stress from PMS product with depth at centre of tank is generally found to be within the net allowable bearing capacity of the soil.

ii. Allowable bearing capacity generally increased with foundation depth, but at BH4, $q_{n(a)}$ decreased with depth up to 3m, attaining a value of about 110 kN/m^2. Beyond which allowable bearing capacity increased with depth.

iii. In Harr's approach, immediate settlement increased with increase in thickness of compressible stratum, with almost a constant rate of increase in settlement.

iv. In Burland and Burbidge approach, immediate settlement decreased with increase in foundation depth.

v. A maximum immediate settlement of 27mm was evaluated in Harr'sapproach for 9m compressible thickness below base plate, while in Burland and Burbidge approach, 30mm immediate settlement occurred at 1m depth.

vi. Total maximum settlement was generally about 61mm for both approaches.

Table 2: Bearing Capacity (SPT Approach)

BH No.	Depth of Foundation (m)	Foundation Equivalent Breadth, B (m)	D_f/ B	Average SPT value (N)	Depth Factor F_d	Allowable bearing capacity q_a (kN/m^2)
1	1	43.24	0.023	12	1.01	147
	3		0.069	15	1.02	198
	5		0.115	16	1.05	204
	7		0.161	18	1.05	229
	9		0.208	20	1.07	259

	1		0.023	13	1.01	159
2	3		0.069	14	1.02	173
	5	43.24	0.115	16	1.05	204
	7		0.161	18	1.05	229
	9		0.208	21	1.07	272
	1		0.023	11	1.01	134
3	3		0.069	14	1.02	173
	5	43.24	0.115	15	1.05	191
	7		0.161	18	1.05	229
	9		0.208	19	1.07	246
	1		0.023	16	1.01	196
4	3		0.069	9	1.02	111
	5	43.24	0.115	9	1.05	114
	7		0.161	10	1.05	127
	9		0.208	12	1.07	155
	1		0.023	11	1.01	134
5	3		0.069	14	1.02	173
	5	43.24	0.115	15	1.05	191
	7		0.161	17	1.05	216
	9		0.208	18	1.07	233

Table 3: Settlement Analysis on Sand

BH No	Depth z(m)	Average SPT value N	Poisson io, μ	Angle of friction (φ)	Elastic Modulus E(Mpa)	Coefficient of volume compressibility m_v (m^2/MN)	Immediate settlement ρ_i(mm)	Immediate settlement ρ_i(mm)	Consolidation settlement,ρ_c (mm)
							Harr's Approach	Burland& Burbidge Approach	
	1	12	0.333	30	12.09	0.055	2.6	27.0	5.8
	3	15	0.326	31	14.34	0.047	7.7	19.8	13.6
1	5	16	0.319	32	14.81	0.047	13.3	18.1	20.8
	7	18	0.319	32	15.77	0.044	17.3	15.3	25.1
	9	20	0.312	33	16.73	0.042	21.0	12.2	28.9
	1	13	0.326	31	13.38	0.051	2.5	24.2	5.4
	3	14	0.326	31	13.86	0.049	8.0	21.8	14.1
2	5	16	0.319	32	14.81	0.047	13.3	18.1	20.8
	7	18	0.319	32	15.77	0.044	17.3	15.3	25.1
	9	21	0.312	33	17.20	0.041	20.5	12.3	27.8
	1	11	0.333	30	12.42	0.053	2.7	30.5	5.6
	3	14	0.326	31	13.86	0.049	8.0	21.8	14.1
3	5	15	0.326	31	14.81	0.046	13.2	19.8	20.3
	7	18	0.319	32	15.77	0.044	17.3	15.3	25.1
	9	19	0.319	32	16.25	0.043	21.6	14.2	29.2
	1	16	0.319	32	14.81	0.047	2.3	18.0	5.2

4	3	9	0.340	29	11.47	0.056	9.5	40.4	16.2
	5	9	0.340	29	11.47	0.056	16.9	40.4	24.7
	7	10	0.340	30	11.95	0.054	22.6	34.9	30.8
	9	12	0.340	30	12.90	0.050	26.9	27.0	33.9
5	1	11	0.333	30	12.42	0.053	2.7	30.5	5.6
	3	14	0.326	31	13.86	0.049	8.0	21.8	14.1
	5	15	0.326	31	14.81	0.046	13.2	19.8	20.3
	7	17	0.319	32	15.29	0.045	17.9	16.6	25.7
	9	18	0.319	32	15.77	0.044	22.2	10.5	29.8

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