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Geotechnical Considerations for the Design and Construction of Foundations in a Marshy Stream Channel of Iwochang - Ibeno, Eastern Niger Delta, Nigeria

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Abstract: A pre-construction site investigation was carried out in a marshy stream channel and adjoining areas for a proposed building site to characterize the subsurface subgrades and recommend foundation design for which proposed structures include a 1-floor 39.7m x 33.7m hostel, a 20.5m x 10.0m 4-bedroom duplex and a 1- floor 2-bedroom block of flats measuring 28.2m x 11.5m with 1.5 factored design dead + live load data as 2700tons, 655tons and 1270 tons respectively. Field investigations include boring of 10 boreholes to a depth of 10m using auger and sounding of 6 cone penetration tests using a 2.5tons mechanical cone penetrometer. The results indicate a soft clay layer existing from ground surface to a depth varying from 1.0m – 1.1m in the stream channel and 0.60m – 0.70m on the adjoining land. These clays are extra-sensitive to sensitive high compressibility Kaolin clays (CH – OH, MH - OH) with undrained shear strength varying from 42 – 75.0KN/m^2, angle of internal friction ranging from 0 - 3^0 with cone resistance values of 3.0 – 11.0 Kg/cm^2. Swell potential ranges from 11.45 – 30.64%, swell index from 0.44 – 0.57, activity from 7.0 – 11.0 and swelling pressure 4.776KPa – 4.890KPa. Below this depth a harder clay layer occurs to a depth of between 4.5m – 5.2m and is proposed to found the structures. Pre-consolidation pressures determined from Oedometer test on undisturbed clay samples retrieved from the centre of the second clay layer varies from 125.0KPa – 162.5KPa and Overconsolidation ratios from 2.75 – 6.40 depicting overconsolidation while water table corrected bearing capacities indicates a favourable fully compensated depth of 1.2m for the building foundations. However excessive total settlement determined using Boussinesq's average vertical stress ranges from 180.1cm - 211.1cm on adjoining land and 160 -111.9cm on the stream channel under the worst case scenario for the structures necessitating further depth compensation to 2.0m. This yielded a reduction in settlement varying from 8.0% to 9.9% on the stream channel and 16.7% - 18.4% on the adjoining land. Rate of settlement depicts that it will take 6.655 and 28.65 years after construction to achieve 50% and 90% settlement under the worst case scenario. Below these clays are loose to medium density sands of varying grain sizes. Load transfer to these sands through pile foundations was considered using the cone penetrometer as a load test to derive unit toe bearing capacities of piles which embedment depth of 11.0m was recommended.

Keywords: Building Load, Vertical Stress, Water Table, Bearing Capacities, Pile Tip Capacities, Total Settlement, Rate of Settlement

1. Introduction

The recent rise in cases of building collapse in Nigeria's coastal cities has drawn attention to the importance of pre-design and construction geotechnical investigations for sustainable infrastructural development. Rather than consider the superstructure, the foundation and the subsoil as a unit and deserving thorough evaluation, characterization of the soil's strength properties has been compromised. Design is often based on assumptions. The stability of a structure founded in the earth depends on the bearing strength of the subsoil, influences due to the superstructure and time dependent displacements caused by subgrade's response to imposed load; location of the structure, erosion due to flooding, root holes, cavities, unconsolidated fills, groundwater table, presence of expansive clays and their swelling pressures etc. [1 - 2]. Foundation soils play one of the most important roles in transmitting structural load and imparting stability [3] thus for sustainable building infrastructure, design and construction must be carried out against bearing capacity failure due to compromised shear strength of the subgrades, and against time dependent displacement of the foundation soil element which could result to differential settlement when the specified maximum acceptable amount of settlement is exceeded. For any satisfactory performance whether shallow or deep, a foundation must be safe against shear in the supporting soil and must not undergo differential settlement. This is only possible when pre-design geotechnical investigation is carried out, foundation materials characterize and design based on the in situ engineering properties.

In the Eastern Niger Delta where this research was carried out, there is a paucity of data on the engineering characteristics of shallow surficial soils which found civil engineering infrastructure as against the deep subsurface. However, insight into the geotechnical properties of shallow subsoils of Western Niger Delta has been provided by [4 – 9]. [10] observed that cities bordering mangrove zones of the Niger Delta such as the study site are characterized by shallow subsurface materials of low to medium bearing capacities under structural loads of buildings and further maintained that, detailed subsurface investigations are absolute necessities for the design and construction of foundations for civil engineering infrastructures. The study area falls within this category of cities and multi-floor buildings in this kind of terrain presents challenges of building foundations. This research was carried out as a commissioned pre-design study for the construction of a skills acquisition centre to comprise of 1-floor 39.7m x 33.7m hostel complex, a 20.5m x 10.0m 4-bedroom duplex and a 1- floor 2-bedroom block of flats measuring 28.2m x 11.5m with 1.5 factored design dead + live load data as 2700tons, 655tons and 1270 tons respectively.

2. Study Location

2.1. Physiography and Climate

Iwochang - Ibeno is located on the southeastern coast of Nigeria, Eastern Niger Delta (Figure 1). It is part of the extension of coastline stretching from the Qua Iboe river estuary a distance of about 2km. Its geographical coordinates are defined by latitude 7^0 45' and 7^0 60'East and longitude $4^0$20' and 4'45' North.

The physiography of the study area is coastal plain, poorly developed with mangrove shrubs intermixed with fresh forest vegetation. It is a wetland characterized by swamps, marshes and bogs. The maximum elevation of the area is about 3m above mean sea level on the sandy barrier islands which are cut by a network of tidal channels through which oceanic waters and tides gain access to an extensive mangrove swamp. According to [11], these Niger Delta swamps are flooded for 3 to 5 months of the year due to natural and man-induced factors causing erosion, which may reach several tens of meters annually.

The climate is tropical equatorial and highest average values of humidity reach 90% in August as against an average minimum of 74% in February. Rainfall is most intense (>3500mm) between April and October, the values being 5 - 7 times higher than in November to March (500mm) [12 - 13].

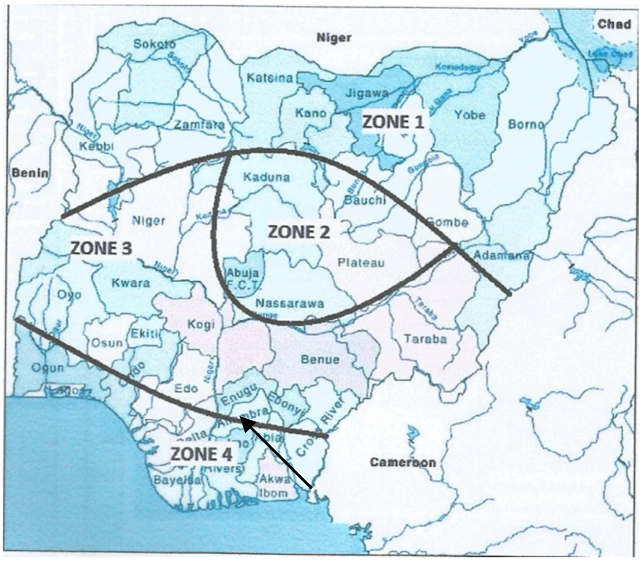


Figure 1. *Rainfall map of Nigeria showing study location.*

2.2. Geologic Setting

The Niger Delta basin evolved through triple junction rifting, opening of the continent and extension of the fracture zones into the Gulf of Guinea during the Cretaceous. The basin has three geologic formations the oldest and basal unit being the Akata shale Formation which is Paleocene to Holocene in age and the hydrocarbon source rock of the basin. It is overlain by intervening paralic sandstone/shale layers of the Agbada Formation, the sandstones forming the hydrocarbon reservoirs. The Agbada formation is overlain by Oligocene to Pleistocene continental sands, sandstones and gravels of the Benin Formation. These friable sands and gravels are of fresh water origin and have excellent aquifer properties with occasional intercalation of shales. The Benin Formation forms the regional aquifer of the Niger Delta basin and it grade into various types of quaternary alluvial deposits comprising mainly of recent deltaic sands on the surface [14 - 16]. .Hydrologically, it is controlled by freshwater rivers intricately linked with creeks that serve as receptacles for sediments from the hinterlands of the Niger River system. The water table is very close to the ground surface and varies from 0 to 4meters while the hydraulic conductivities of the sands vary from 3.82 x 10^{-3} to 9.0 x 10^{-2}cm/sec which indicates a potentially productive aquifer [18, 8].

3. Method of Study

Study method comprise of fieldwork, laboratory testing programmes and data analysis. Field investigations include site inspection and identification of tests points using global positioning system (table 1), geotechnical boring of ten boreholes using auger rigs to a depth of 10m based on the number of floors or height of the buildings [1, 18 - 19], and soil samples obtained at 1m interval for lithological description and laboratory analysis. In situ testing was equally carried out using a 2.5KN hydraulically operated mechanical cone penetrometer employing discontinuous sounding procedure. The equipment was anchored to the ground and the 60° steel cone with base area of 10cm^2 pushed into the ground at rates of 2cm/sec while the resistance (stiffness) to penetration in kg/cm^2 was measured at regular intervals of 0.2m. The recorded cone resistance readings were used to derive undrained shear strength (C_u), safe allowable bearing pressures and unit pile tip capacities of sub-soil using established empirical relations described.

Laboratory tests on retrieved soil samples include test for soil identification and classification [20] as well as shear strength and consolidation tests all carried out in accordance with [19, 21].

3.1. Data Analysis

3.1.1. Bearing Capacity

The ultimate (q_{ult}) and allowable (q_{all}) bearing capacities of the subgrades were determined for strip, square, rectangular and mat foundations using equations (1 – 4) applying a factor of safety of 3.0. Due to reduction in bearing capacity caused by water table within the limit of influence, Terzaghi water table correction factors (eqns. 5 - 6) were applied to the equations for determination of bearing capacity to account for its effect. These equations are,

Strip Footing

$$Qu = CN_c + qN_q R_{W1} + 0.5_\gamma CBN_\gamma R_{W2} \text{ --} \quad (1)$$

Square footing

$$Qu = 1.3CN_c + qN_q R_{W1} + 0.4yBN_\gamma R_{W2} . \quad (2)$$

Rectangular footing

$$Qu = CN_c(1+0.3B/L) + _\gamma D_f N_q R_{W1} + 0.5_\gamma BN_\gamma(1 - 0.2B/L)R_{W2} \quad (3)$$

Mat footing

$$Qu = 5.14C_u(1 + 0.195B/L)(1+0.4Df/B) \quad (4)$$

$$R_{W1} = 0.5(1 + D_{W1}/D_f) \quad (5)$$

$$R_{W2} = 0.5(1 + D_{W2}/B) \quad (6)$$

where,

N_c, N_q , N_γ = Bearing capacity factors
C_u = undrained shear strength
B = Foundation Breadth
L = Foundation length
R_{W1} = reduction factor for water table above the base level of the foundation
R_{W2} = reduction factor for water table below the base level of the foundation
D_{W1} = Water table below ground level
D_{W2} = Water table below foundation footing
D_f = Depth of foundation

Undrained shear strength (C_u) and allowable bearing pressures (q_p) have also been calculated using eqn. (7) [22 - 24] applying a cone factor of 20 for both normally consolidated and overconsolidated clays [25] and eqn. (8) [26] respectively based on the cone resistance (q_c) readings for the subsurface soils and equation.

$$Cu = \frac{q_c - P_o}{N_k} = \frac{q_c}{N_k} \quad (7)$$

$$q_p = 2.7q_c \quad (8)$$

where,

$q_c - N_k$ = net cone resistance
P_o = total overburden pressure
N_k = cone factor
q_p = allowable bearing pressure in KN/m^2
q_c = cone resistance in Kg/cm^2
R_{W2} = reduction factor for water table below the base level of the foundation

Table 1. *Cordinates of boreholes and CPT tests points.*

S/No	Northings	Eastings
BH1	N04^0 32.717”	E007^0 59.049”
BH2	N04^0 29.362”	E007^0 55.917”
BH3	N04^0 22.835”	E007^0 40.594”
BH4	N04^0 29.550”	E007^0 57.114”
BH5	N04^0 33.010”	E007^0 53.900”
BH6	N04^0 34.544”	E007^0 56.777”
BH7	N04^0 36.169”	E007^0 57.803”
BH8	N04^0 31.722”	E007^0 57.211”
BH9	N04^0 43.2400”	E007^0 01.851”
BH10	N04^0 38.342”	E007^0 55.597”
CPT1	N04^0 30.900”	E007^0 00.788”
CPT2	N04^0 32.080”	E007^0 58.644”

S/No	Northings	Eastings
CPT3	N04^0 33.928"	E007^0 54.526"
CPT4	N04^0 33.978"	E007^0 54.546"
CPT5	N04^0 38.364"	E007^0 48.051"
CPT6	N04^0 37.680"	E007^0 58.904"

3.1.2. Settlement Analysis

Oedometer consolidation tests [27] were conducted on the clay samples retrieved from the field. Undisturbed samples were tested in a 75mm diameter x 20mm high ring over a pressure range of 40kPa – 320kPa and data analyzed using Taylor square root of time fitting method [28] to derive the consolidation indices. Pre-consolidation pressures were determined from void ratio vs log of pressure curves using Cassagrande's method of construction [29].

In consideration of design against excessive settlement of footings, the total settlement and rates of settlement for 50% and 90% were evaluated. Total settlement was determined using eqn. (9) for overconsolidated clays applying the criteria $P_0 < P_c < (P_0 + \Delta P_{av})$ [1], while 50% and 90% rates of settlement were calculated using eqns. (10) and (11) respectively.

TOTAL SETTLEMENT (S_T)

$$S_T = \frac{C_S H_c}{1+e_o} log\frac{P_c}{P_0} + \frac{C_c H_c}{1+e_o} log(\frac{P_o+\Delta P_{av}}{P_c}) \tag{9}$$

RATE OF SETTLEMENT

$$t_{50} = \frac{0.197 T_{50} H_c^2}{C_v} \tag{10}$$

$$t_{90} = \frac{0.848 T_{90} H_c^2}{C_v} \tag{11}$$

where,

C_s = Swell index
C_c = Compression index = 0.009(LL-10)[30]
H_c = thickness of the clay layer
e_0 = initial void ratio
P_o = average effective pressure on the clay before foundation construction
P_c = Pre-consolidation pressure
ΔP_{av} = average increase of vertical stress on the Soil mass caused by the foundation.
M_v = Coefficient of volume compressibility
σ_v = vertical stress
t_{50} = time to achieve 50% settlement
t_{90} = time to achieve 90% settlement
C_v = Coefficient of consolidation

3.1.3. Determination of Average Vertical Stress

Boussinesq method [31] for determination of total average vertical stress at any point below the ground surface for a point load under a uniformly loaded rectangular area with zero eccentricity (eqn. 12) was employed to determine the total average vertical stress for settlement calculation. This is stated as,

$$\Delta P_{av} = \frac{1}{H}\int_o^H (q_o I) d_z = q_o I_a \tag{12}$$

where I_a = f(m.n) = average influence factor and m = B/z and n = L/z.

q_0 = load per unit area on the loaded area.

The average influence factors using m and n were obtained from Griffith table [32] of influence values.

3.2. Investigation of Clay Swelling

The swelling, expansion and collapsible properties of the saturated clays were investigated. According to [33], since the Liquid limit and swell characteristics of clays depend on the amount of water it can absorb, the swell potential of clays can be evaluated using simple soil property tests such as Atterberg limits, linear shrinkage, free swell and colloid content [34].This has been applied in this study using eqn. (13) proposed by [35] for swell index, eqn. (14) based on clay activity [36], eqn. (15) for swelling potential [37] and eqn. (16) for swelling pressure [38]. These are defined by the equations 13 – 16.

$$C_s = 0.0463(LL/100)G_s \tag{13}$$

$$A = I_p/(\% \text{ Clay} - n) \tag{14}$$

$$S_p = 2.16 \times 10^{-2} I_p^{2.44} \tag{15}$$

$$P_s = 3.6 \times 10^{-2} [I_p]^{1.12} [C/W_n] \tag{16}$$

where, Cs = swell index, LL = Liquid Limit, G_s = specific gravity, = A = activity, I_p = plasticity index, n = 5 for natural soils, P_s= swelling pressure, ρ_d = dry density, C = % Clay fraction and W_n = natural moisture content.

3.3. Pile Capacity from Cone Penetration Test

In view of excessive total settlement, pile foundations have been considered based on their load transfer mechanism along the pile length and as point load at the base of the pile where dense sands occur below the firm clay layer at 4.5m – 5.2m. Consequently, cone penetrometer test, as a small pile load test, its results have been used to determine the pile toe capacities according to the method proposed by [39]. In this method, unit tip bearing capacity (qt) is evaluated using the cone tip resistance (q_c).

The equations for calculation are

$$q_t = K_b q_c \tag{17}$$

$$q_s = K_s f_s \tag{18}$$

where,

q_t = Unit pile tip bearing capacity,
Q_s = pile skin frictional resistance
K_b = 0.35 for driven piles,
K_s = 0.53 for driven piles

4. Results and Discussion

4.1. Geotechnical Properties of the Clay Layers

The soft clay underlying the site from 0 - 0.6m on land and 0 – 1.1m in the stream channel is of low bearing strength with very high moisture contents, swell potential, swell index, and high compressibility. The natural moisture content varies from 76.26 – 91.3%, the liquid limit from 83.6 – 90.3%, plasticity index from 40.3 - 43.6 while the plastic limit ranges from 40.0 – 50.0% classifying as high plasticity and high compressibility inorganic clays (CH – OH, MH - OH) under the unified soil classification scheme (figure 2a). The undrained shear strength of this layer varies from 42 – 75.0KN/m^2 while the angle of internal friction ranges from 0 - 3^0 with cone resistance values of 3.0 – 5.0 Kg/cm^2 on the stream channel and 5 - 11.0 Kg/cm^2 on land rated as soft [40]. The swell potential varies from 11.45 – 30.64%, swell index from 0.44 – 0.57, activity from 7.0 – 11.0 and swelling pressure 4.776 – 4.890KPa. It is amenable to high volume changes. The second clay layer occurs at a depth range of 0.6m - 4.2m on land and 1.1m – 5.2m on the stream channel. This clay layer has natural moisture content ranging from 17.0 - 82.1.%, Liquid limit of 49.2 – 61.2 % and plasticity index 27.8 – 33.2 classifying as intermediate to high plasticity and medium to high compressibility inorganic clays and organic clays (CH – OH, MH - OH) (figure 2b). The undrained shear strength of this layer varies from 45.0 – 50.0KN/m^2 while the angle of internal friction ranges from 4.0 – 8.0^0 with cone resistance values of 5.0 – 11.0 Kg/cm^2. The swelling characteristics indicates swell index of 0.12 – 0.15, swell potential of 4.83 to 7.21%, activity of 4.0 – 7.0 and swelling pressure of 3.979 to 4.216 KPa with intermediate to high compressibility (table 2).

This layer offers a higher bearing strength and is recommended to found the structures. Water table corrected ultimate and allowable bearing capacities were 441.08KN/m^2 and 147.03KN/m^2 respectively at 1.2m depth and 450.01 KN/m^2 and 180.05 KN/m^2 at 2.0m depth for strip footings under saturated conditions. Under partially saturated conditions (on the adjoining land) the ultimate and allowable bearing capacities varies from 306.57 – 324.50KN/m^2 and 102.19 – 108.20 KN/m^2 respectively at 1.2m depth and 318.79 – 333.15KN/m^2 and 106.26 – 111.05KN/m^2 respectively at 2.0m depth. The ultimate bearing capacities range for strip footing for completely unsaturated foundation condition on the adjoining land varies from 456.15 KN/m^2 at 1.2m to 479.7KN/m^2 at 2.0m while the allowable bearing capacity varies from 152.05 KN/m^2 to 156.9KN/m^2. For square foundation footings under saturated site conditions, the' ultimate and allowable bearing capacities varies from 569.33 KN/m^2 and 189.78KN/m^2 at 1.2m depth to 578.34 KN/m^2 and 192.78 KN/m^2 at 2.0m depth. On the adjoining land under partial saturation, it was 384.54KN/m^2 and 128.18 KN/m^2 – 435.21KN/m^2 and 145.07 KN/m^2 at 1.2m and 396.78KN/m^2 and 132.26KN/m^2 – 430.17 KN/m^2 and 143.39KN/m^2 at 2.0m. For a completely unsaturated section of the adjoining land, the ultimate and allowable bearing capacity varies from 609.57KN/m^2 and 203.19KN/m^2 at 1.2m to 635.71KN/m^2 and 211.90KN/m^2 at 2.0m depths respectively.

The bearing capacity of rectangular footings varies with the foundation aspect ratio, soil strength properties and depth. For aspect ratio of 1 under saturated conditions, the ultimate and allowable bearing capacity varies from 491.63 and 168.88 at 1.2m to 580.56 and 193.45 at 2.0m. At aspect ratio of 1.5, under the same condition, it was 523.69 KN/m^2 and 174.56 KN/m^2 at 1.2m to 644.69 KN/m^2 and 214.90 KN/m^2 at 2.0m. Under partial saturation conditions, results indicates ultimate and allowable bearing capacity of 213.78 KN/m^2 and 71.26 to 435.35KN/m^2 and 145.12 KN/m^2 at 1.2m and 226.0KN/m^2 and 75.34 KN/m^2 to 429.43 KN/m^2 and 143.14KN/m^2 at 2.0m with aspect ratios 1 and1.5 respectively. For the completely unsaturated site section adjoining the stream channel, the ultimate and allowable bearing capacities varies from 493.35 KN/m^2 and 164.45 at 1.m to 603.97 KN/m^2 and 201.33 KN/m^2 at 2.0m with aspect ratio 1 and 682.40 KN/m^2 and 228.13 KN/m^2 at 1.2m to 668.47 KN/m^2 and 222.82KN/m^2 at 2.0m with aspect ratio 1.5 (table 6).

The ultimate and allowable bearing capacity of raft foundation for saturated site condition varies from 433.43 - 455.59KN/m^2 and 144.48 – 151.83 KN/m^2 respectively at 1.2m and 445.01 – 459.88KN/m^2 and 148.34 – 153.29KN/m^2 respectively at 2.0m depth. For the partially saturated site conditions, the ultimate and allowable bearing capacities range from 242.72 – 255.14KN/m^2 and 80.91 – 85.04KN/m^2 respectively at 1.2m depth and 348.07 – 257.52KN/m^2 and 82.67 – 85.84KN/m^2 respectively at 2.0m depth. For the unsaturated sections of the site, the ultimate and allowable capacity varies from 288.94 – 303.74 KN/m^2 and 96.31 – 101.25KN/m^2 respectively at 1.2m depth. At 2.0m depth, it varies from 296.89 – 306.57KN/m^2 and 98.89 – 102.19KN/m^2 respectively (table 7).

On the basis of Atterberg limits (table 2 and figure 2), both the soft and hard clay layers classify as Kaolinitic clays capable of reversal or reduction in plasticity indices when in contact with water upon consolidation, and partial cementation during burial.

Results of the Oedometer consolidation tests on the clay indicates the indices of consolidation such as coefficient of consolidation (C_v) to range 4.76 x 10^{-2} - 2.292 (Cm^2/min), coefficient of volume compressibility (M_v) from 2.097 x 10^{-4} – 4.965 x 10^{-4} KPa$^-$ and pre-consolidation pressures from 125 – 162.5 (table 3 and figures 3 and 4). When compared with the present overburden pressure across the site, the pre-consolidation pressure was higher indicating pre-consolidation of the clay layers with overconsolidation ratios varying from 2.75 – 6.40.

4.2. Geotechnical Properties of the Sand Layers

Underlying the harder clay layer is a sand layer which extends from 4.2 to 5.2m across the site to beyond 10.0m where boring terminated. These sands are loose to medium density and medium to coarse grain, non-plastic sands. Cone penetration tests values within this layer range from 19.0 – 270 Kg/Cm^2 with refusal varying from 8.0 – 9.0m on land and 11.0m – 12.0m on the stream channel. The bulk density ranges from 2.03 – 2.39 g/cm^3 while the unit weight varies from 20.01 – 22.56KN/m^3 (table 4). Undrained shear strength varies from 94.059 – 729KN/m^2 (figure 5a – f). These sand layers are in saturated condition.

Table 2. *Geotechnical properties of the clay layers.*

SOIL LAYER	SOFT CLAY LAYER			HARDER CLAY LAYER		
Property	**Minimum**	**Maximum**	**Average**	**Minimum**	**Maximum**	**Average**
Moisture Content, W_n (%)	76.26	91.3	89.4	17.0	48.17	40.7
Wet Density (g/cm^3)	1.131	1.371	1.35	1.623	1.970	1.770
Wet Unit weight (KN/m^3)	11.095	13.449	12.77	15.922	19.34	17.03
Dry Density (g/cm^3)	0.701	0.821	0.802	0.972	1.551	1.39
Dry Unit weight (KN/m^3)	6.877	8.054	7.92	9.535	15.215	13.91
Saturated Unit weight (KN/m^3)	22.64	24.81	23.98	16.092	16.336	16.271
Specific Gravity (G_s)	2.42	2.51	2.49	2.46	2.92	2.89
Porosity (n)	0.63	0.649	0.638	0.56	0.65	0.62
Void ratio (e)	1.70	1.85	1.82	1.28	1.33	1.31
Initial void ratio	1.0727	1.3321	1.2991	2.1920	3.8461	3.1173
Compression index (C_C)	0.1291	0.1863	0.1784	0.7227	0.722	0.420
Swell Index (C_S)	0.44	0.57	0.49	0.12	0.15	0.14
Swell pressure (KPa)	4.776	4.840	4.803	3.979	4.216	4.19
Swell potential (%)	11.4	30.64	27.33	4.83	7.21	6.01
Undrained shear strength C_u (KN/m^2)	42.0	75.0	69.0	45	50	47.91
Friction angle (ϕ)^0	0	3	2.91	4	8	7.0
LL (%)	83.6	90.3	89.02	49.2	61.2	59.7
PL (%)	40.0	50.0	48.71	21.4	28.0	26.9
PI	40.3	43.6	40.31	27.8	33.2	31.6
Activity (%)	7.0	11	10.37	4.0	7.0	5.1
USCS Classification	CH-OH, MH – OH			CH – OH, MH – OH		

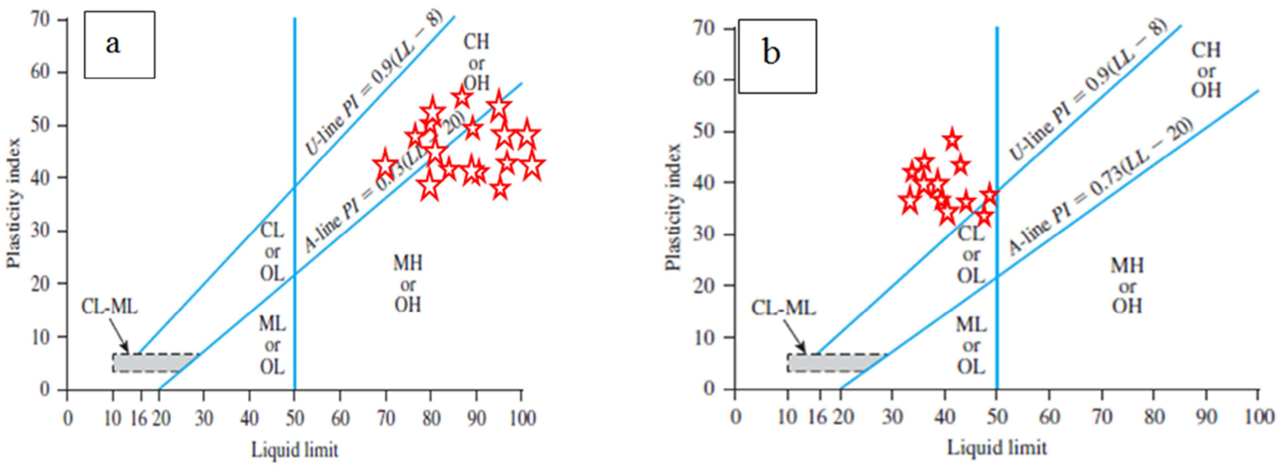


Figure 2. *Cassagrande plasticity chart (a) soft clay layer (b) Hard clay layer.*

Table 3. *Consolidation properties of the clay layers.*

Consolidation Parameter	Minimum	Maximum	Average
Coefficient of consolidation (C_v) (Cm^2/min)	4.76 x 10^{-2}	2.296	1.928
Coefficient of volume compressibility (M_v) KPa$^-$	2.097 x 10^{-4}	4.965 x 10^{-4}	3.801x10^{-4}
Pre-consolidation pressure (KPa)	125.0	162.5	160.3
Coefficient of compressibility (av) KPa$^-$	4.891 x 10^{-4}	2.406 x 10^{-3}	1.928 x 10^{-3}
Overconsolidation ratio	2.75	6.40	4.09

Table 4. *Typical Geotechnical Properties of the Sand Layers.*

Depth m.	Soil type	Colour	Texture	W_n %	Consistency			Bulk Density	Unit Weight
5.0	Sand	Brown	Loose, medium grain	11.90	NP	NP	NP	2.04	20.01
6.0	Sand	Brown	Loose, medium grain	17.96	NP	NP	NP	2.29	22.47
7.0	Sand	Brown	Loose, medium – coarse	13.88	NP	NP	NP	2.30	22.56
8.0	Sand	Grey	Loose, Medium grain	13.88	NP	NP	NP	2.30	22.56
9.0	Sand	Grey	Loose, medium grain	13.88	NP	NP	NP	2.30	22.56
10	Sand	Grey	Loose, medium grain	13.88	NP	NP	NP	2.30	22.56

Table 5. *Typical Particle Size Distribution results.*

Sample ID	Depth (m)	D_{10} Mm	D_{30} Mm	D_{60} mm	CU	CC	% Clay	% Silt	%Sand	% Gravel
BH1	1.0	0.102	0.203	0.301	2.95	1.34	19.5	18.3	61.3	0.9
	2m – 4.2	0.107	0.200	0.250	2.34	1.50	11.5	23.5	64.0	1.0
BH1	6.5 – 8	0.150	0.30	0.48	3.2	1.25	0.0	0.2	97.0	3.0
	8.0 – 10	0.150	0.30	0.48	3.2	1.25	0.1	1.2	97.0	1.3
	2m – 4.2	0.107	0.200	0.250	2.34	1.50	0.5	3.0	95.5	1.0
BH2	4.2 – 6.5	0.150	0.30	0.48	3.2	1.25	0.0	49.0	50.8	0.2
	6.5 – 10	0.150	0.30	0.48	3.2	1.25	0	38	62	0
BH3	1.0	0.101	0.33	0.411	4.1	2.62	26.1	37.3	36.5	0.1
	2 – 4.5	0.132	0.148	0.244	1.85	0.68	3.4	13.1	82.3	1.2
	4.5 – 7.	0.150	0.30	0.48	3.2	1.25	10.3	22.7	66.9	0.1
	7 – 10.	0.150	0.30	0.48	3.2	1.25	0.1	15.7	84.3	0.1
BH4	1.0	0.102	0.304	0.39	3.8	2.32	33.1	27.9	37.6	1.4
	2 – 4.5	0.101	0.217	0.230	2.3	2.03	22.9	34.3	42.2	0.6
	4.5 – 7.0	0.150	0.30	0.48	3.2	1.25	0.0	24.3	75.7	0
	7 – 10.	0.150	0.30	0.48	3.2	1.25	0.5	21.7	76.4	1.4
BH10	1.0	0.100	0.290	0.39	3.9	2.16	17.3	57.4	24.2	1.1
	2 – 4.5	0.104	0.303	0.39	3.75	2.26	11.5	25.5	61.8	1.2
	4.5 – 7.	0.150	0.30	0.48	3.2	1.25	0.8	18.7	79.8	0.7
	7 – 10.	0.150	0.30	0.48	3.2	1.25	0.2	41	58.0	0.8

4.3. Settlement Chacteristics of the Clay Layers

In consideration of design against excessive displacement and differential settlement due to vertical load transfer mechanisms for a rectangular loaded area, average vertical stress values of 4003.2 KN/m^2, 924.53 KN/m^2 and 1822.74 KN/m^2 for the hostel, 4-Bedroom duplex and 2-Bedroom block of flats respectively determined using Boussinessq equation have been applied in predicting the settlement characteristics of the clay layer. The results of the total settlement at a fully compensated depth of 1.2m was 211.1cm and 160.1cm for the hostel; 180.1cm and 111.9cm for the 4 Bedroom duplex and 190.5cm and 134.5cm for the 2-Bedroom block for the adjoining solid ground and the stream channel respectively (table 8). Based on excessive settlement results, further depth compensation aimed at reducing the net pressure on the foundation to prevent differential settlement was considered to 2.0m. Results depict 175.9cm and 146.5cm for the hostel, 146.9cm and 101.5cm for the 4-Bedroom duplex and 157.3cm and 124.0cm for the 2-Bedroom block of flats on land and stream channel respectively resulting in reduction of total settlement of 8.5% and 16.7% for hostel, 9.3% and 18.4% for the 4-Bedroom duplex and 8.0% and 17.4% for the 2-Bedroom block of flats if sited on the stream channel and adjoining land respectively. The immediate settlement varies from 0.861mm – 1.74mm (table 8). Time rate of settlement due to dissipation of excess pore water pressure as result of applied vertical stress on the clay layers accompanied by an increase in effective vertical stress shows that it will take 6.655 and 28.65 years after construction years to achieve 50% and 90% settlements under the worst case scenario. The rate of settlement for the other structures and for the different site sections (stream or land) is presented in table 8. Site layout is therefore recommended to consider setting out of the smallest loaded structure (4-Bedroom duplex) on the stream channel section and the hostel and 2-Bedroom block of flats on the adjoining land.

Table 6. *Bearing Capacity of strip, Square/Rectangular foundation*

Depth (m)	Rectangular foundation			
	B/L = 1		B/L = 1.5	
	Ultimate Bearing Capacity (KN/m^2)	Allowable Bearing Capacity (KN/m^2)	Ultimate Bearing Capacity (KN/m^2)	Allowable Bearing Capacity (KN/m^2)
Cu = 75KN/m^2, $\phi = 0^0$				
1.2	491.63	163.88	523.69	174.56
2.0	580.56	193.52	644.69	214.90
Cu = 42 KN/m^2, $\phi = 3^0$				
1.2	213.78	71.26	171.97	57.32
2.0	226.0	75.34	184.2	61.40
Cu = 50 KN/m^2, $\phi = 8^0$				
1.2	493.35	164.45	684.40	228.13
2.0	603.97	201.33	668.47	222.82
Cu = 45 KN/m^2, $\phi = 4^0$				
1.2	435.35	145.12	482.18	160.72
2.0	429.43	143.14	477.22	159.07

Table 6. *Continue.*

Depth (m)	Strip foundation		Square foundation	
	Ultimate Bearing capacity (KN/m^2)	Allowable Bearing capacity (KN/m^2)	Ultimate Bearing capacity (KN/m^2)	Allowable Bearing capacity (KN/m^2)
Cu = 75KN/m^2, $\phi = 0^0$				
1.2	441.08	147.03	569.33	189.78
2.0	450.01	150.05	578.34	192.78
Cu = 42 KN/m^2, $\phi = 3^0$				
1.2	306.57	102.19	384.54	128.18
2.0	318.79	106.26	396.78	132.26
Cu = 50 KN/m^2, $\phi = 8^0$				
1.2	456.15	152.05	609.57	203.19
2.0	479.7	156.9	635.71	211.9
Cu = 45 KN/m^2, $\phi = 4^0$				
1.2	324.5	108.2	435.21	145.07
2.0	333.15	111.05	430.17	143.39

Table 7. *Bearing capacity of Raft foundation.*

STRUCTURE	Depth (m)	Soil properties					
		Cu = 75KN/m^2, $\phi = 0^0$		Cu = 42 KN/m^2, $\phi = 3^0$		Cu = 50 KN/m^2, $\phi = 8^0$	
		Ultimate Bearing capacity	Allowable Bearing capacity	Ultimate Bearing capacity	Allowable Bearing capacity	Ultimate Bearing capacity	Allowable Bearing capacity
2-Bedroom block of flats	1.2	433.43	144.48	242.72	80.91	288.94	96.31
	2.0	445.01	148.34	248.01	82.67	296.89	98.89
4-Bedroom duplex	1.2	442.30	147.43	247.71	82.57	294.86	98.29
	2.0	455.85	151.95	255.27	85.09	303.89	101.29
1-Floor Hostel complex	1.2	455.59	151.83	255.14	85.04	303.74	101.25
	2.0	459.88	153.29	257.52	85.84	306.57	102.19

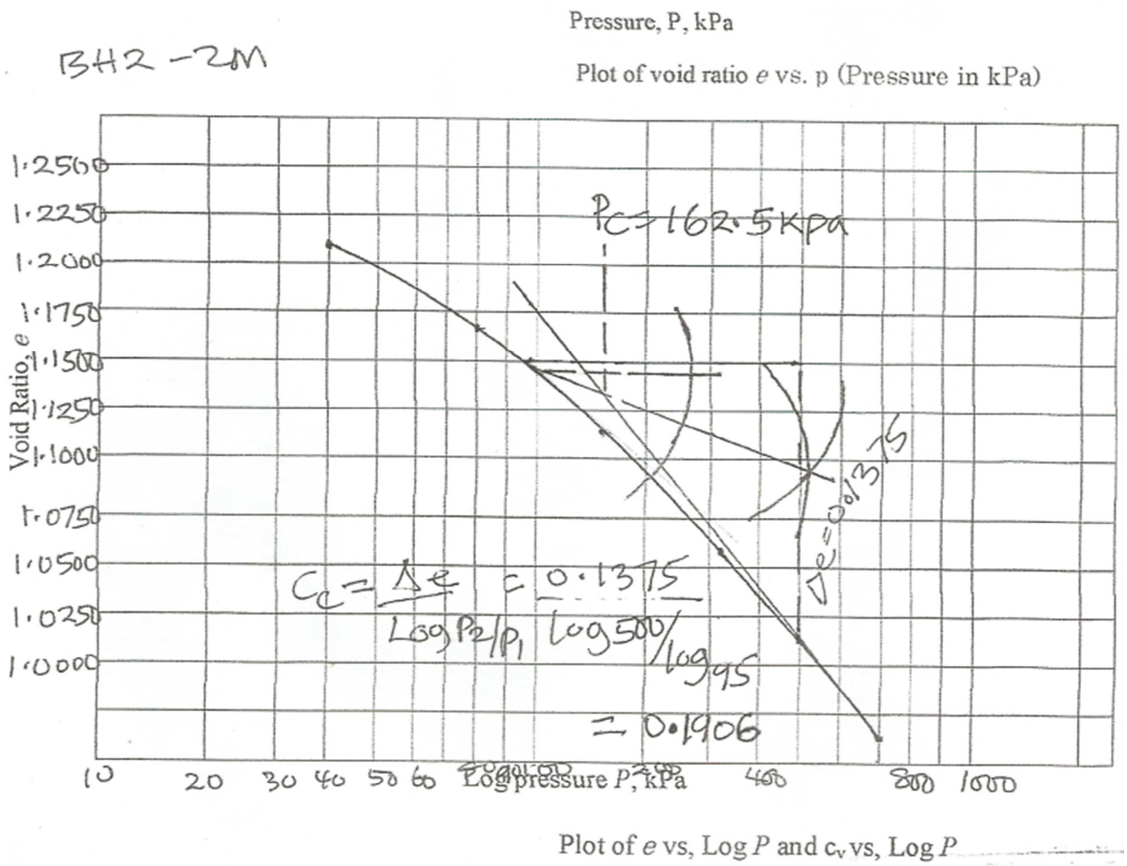


Figure 3. *Typical e log p curve for land section.*

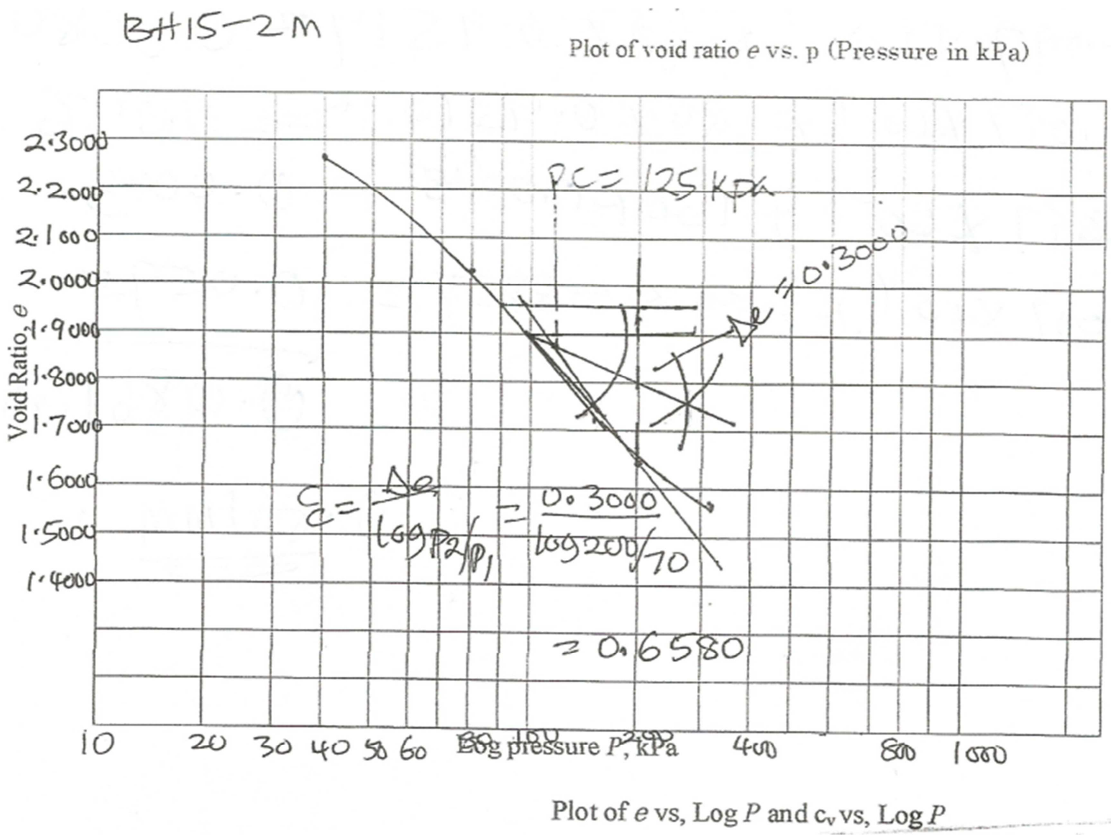
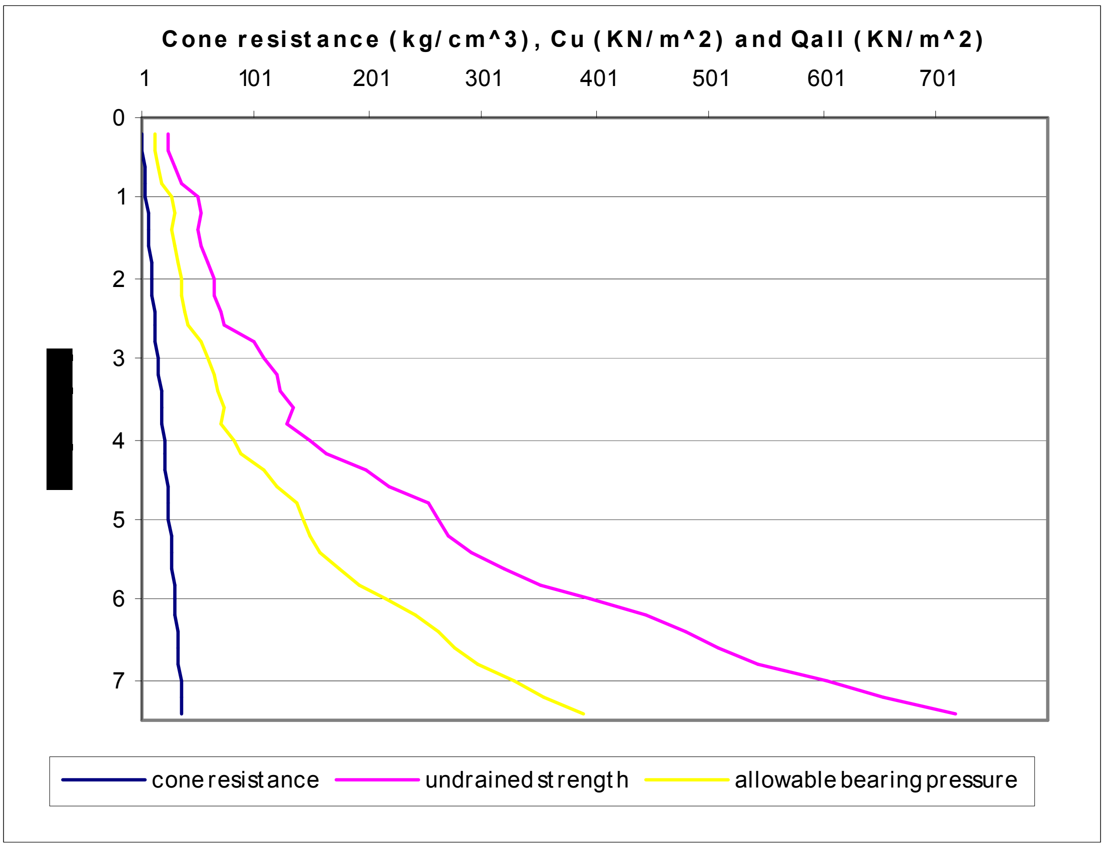
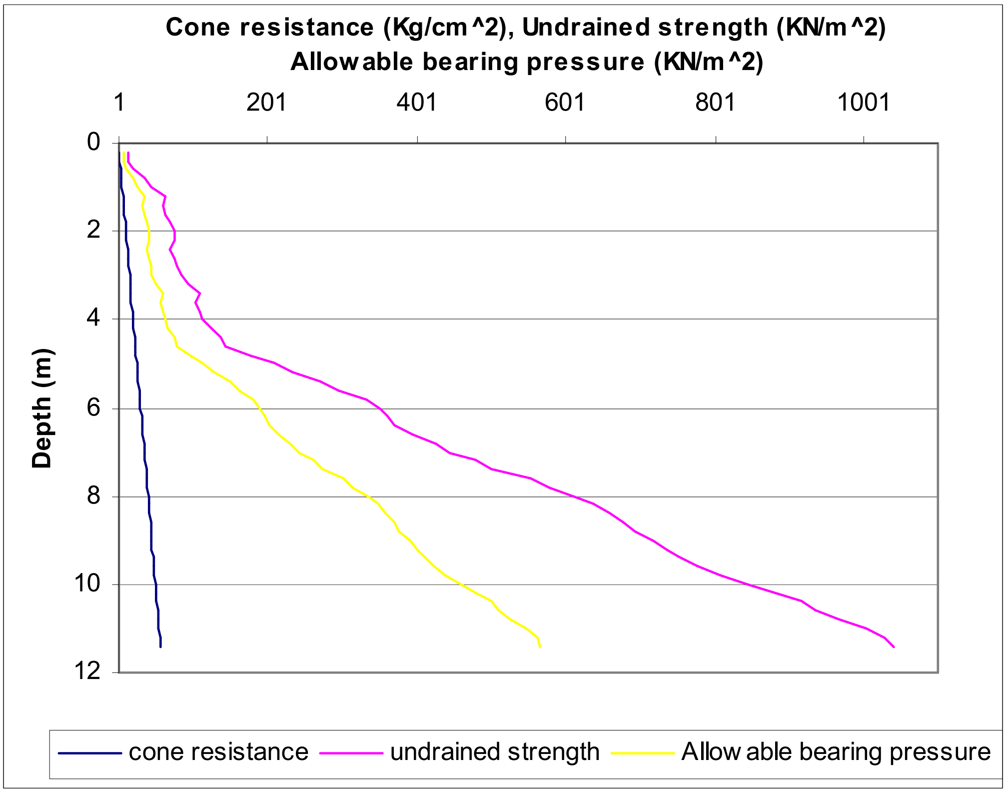


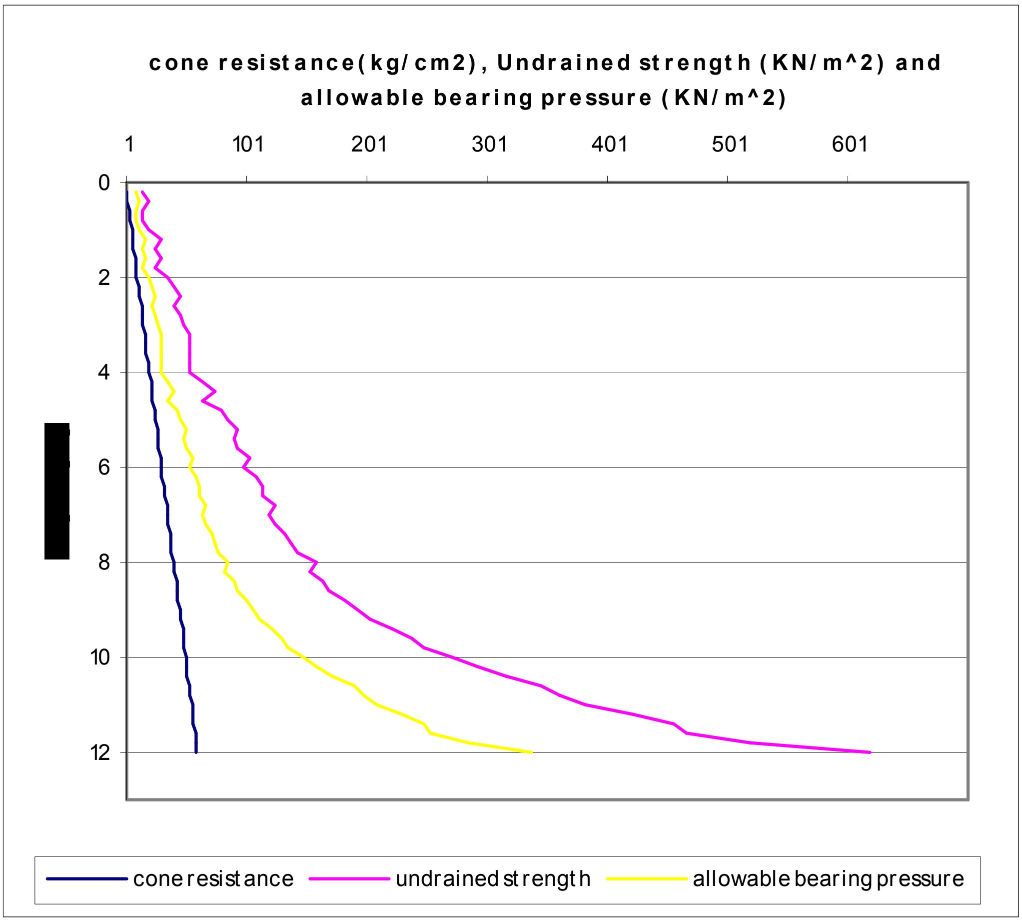
Figure 4. *Typical e log p curve for stream channel.*



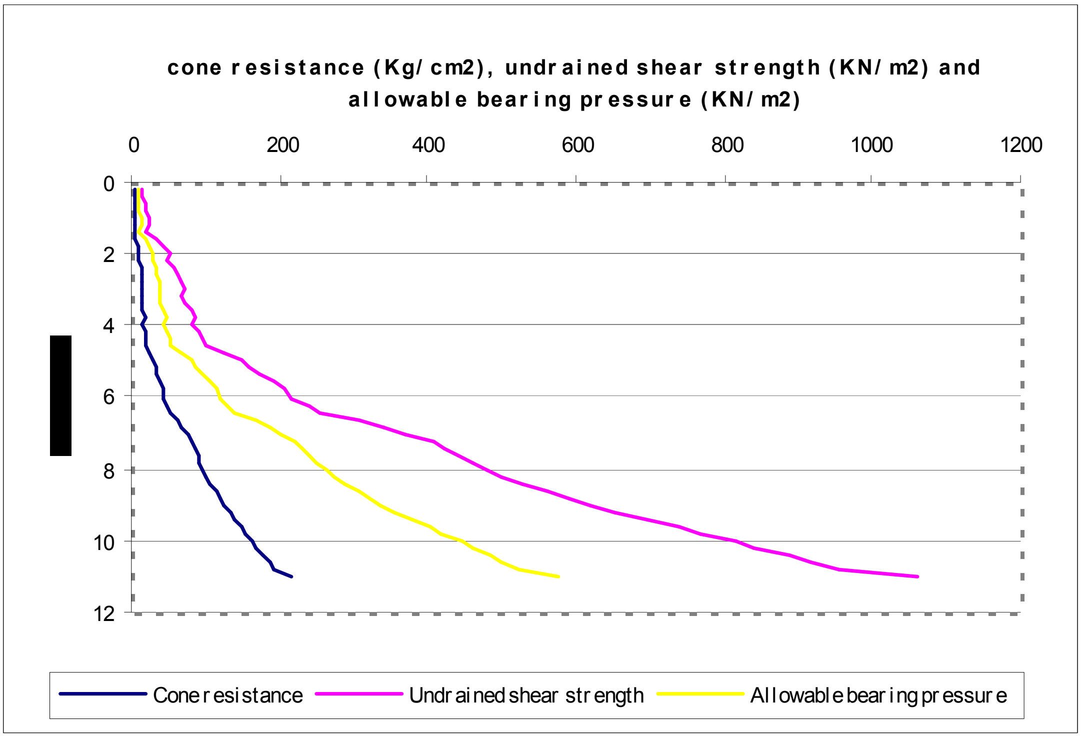
a.


Cone resistance (Kg/cm^2), Undrained strength (KN/m^2)
Allowable bearing pressure (KN/m^2)
1
201
401
601
801
1001
0
2
4
6
8
10
12
Depth (m)
cone resistance
undrained strength
Allowable bearing pressure

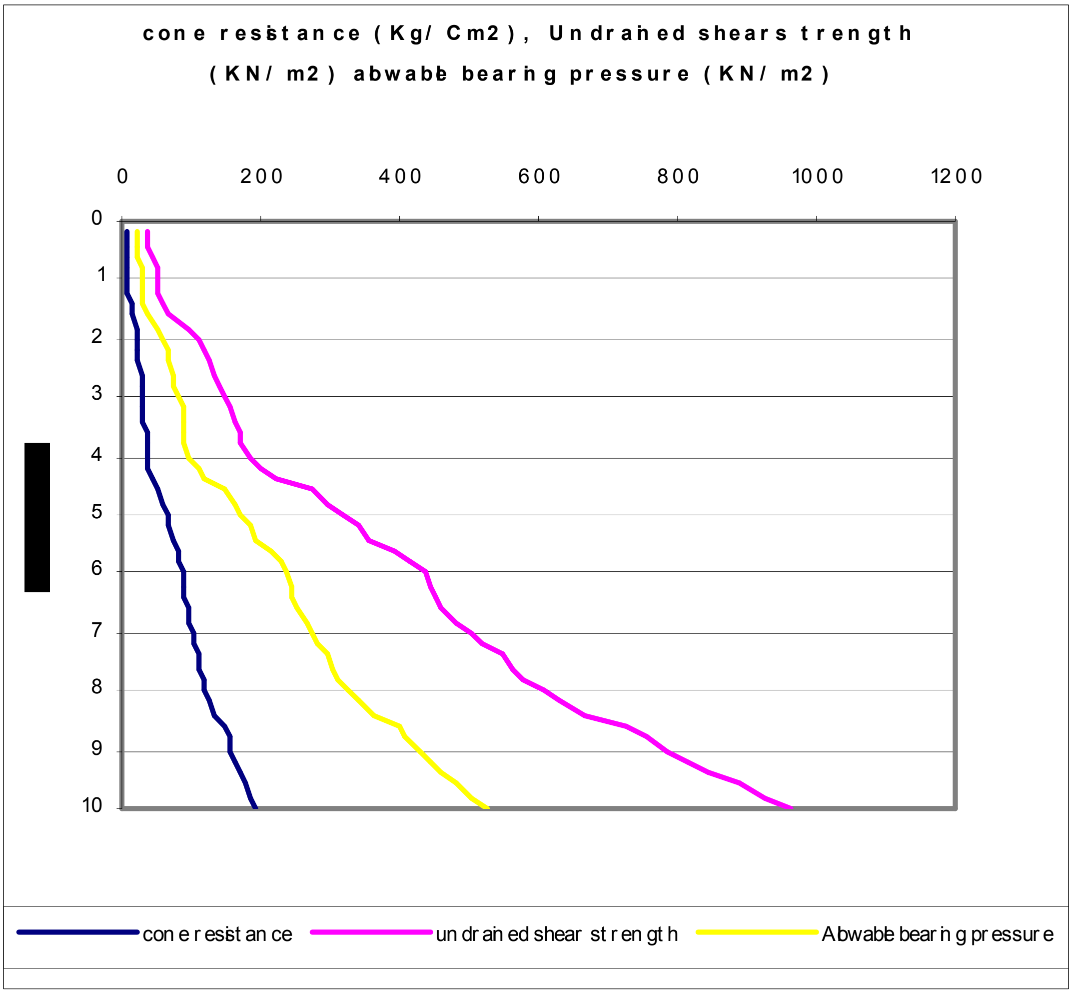
b.


cone resistance(kg/cm2), Undrained strength (KN/m^2) and
allowable bearing pressure (KN/m^2)
1
101
201
301
401
501
601
0
2
4
6
8
10
12
[redacted]
cone resistance
undrained strength
allowable bearing pressure

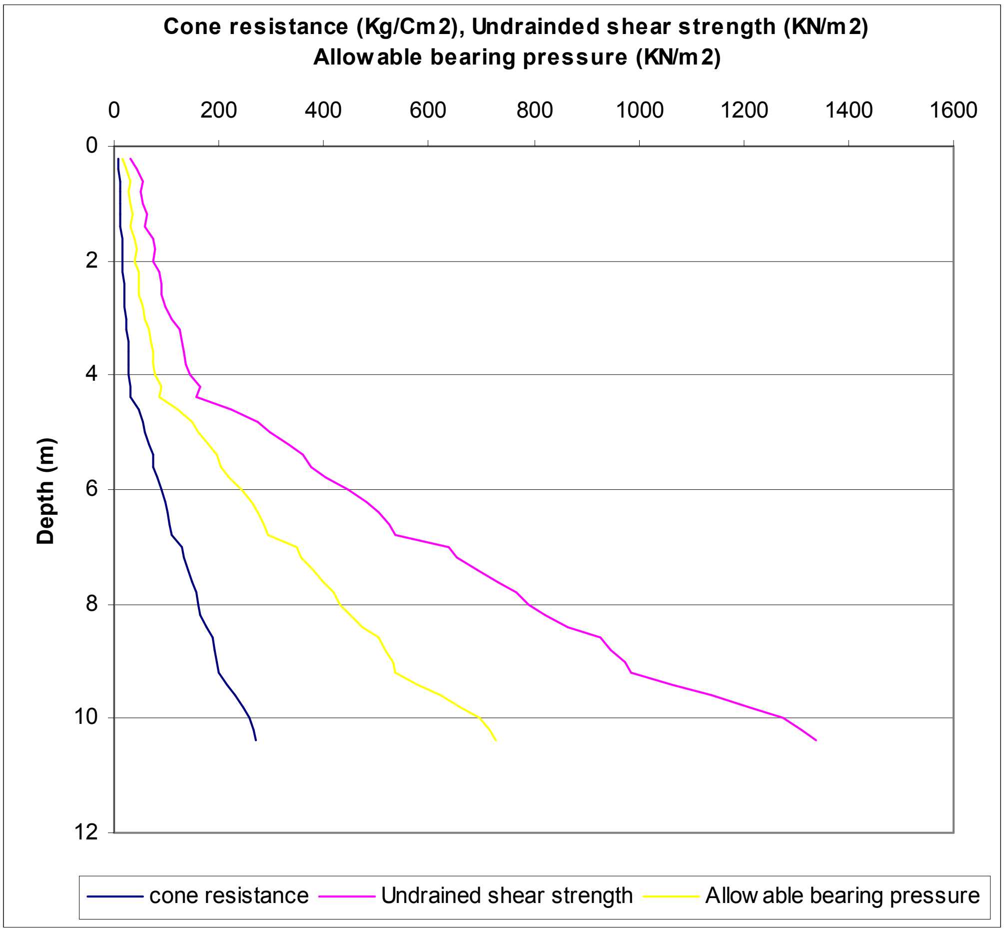
c.


cone resistance (Kg/ cm2), undrained shear strength (KN/ m2) and allowable bearing pressure (KN/ m2)
0
200
400
600
800
1000
1200
0
2
4
6
8
10
12
[redacted]
Cone resistance
Undrained shear strength
Allowable bearing pressure

d.


cone resistance (Kg/ Cm2), Undrained shearstrength (KN/ m2) abwable bearing pressure (KN/ m2)
0
200
400
600
800
1000
1200
0
1
2
3
4
5
6
7
8
9
10
[redacted]
cone resistance
undrained shear strength
Abwable bearing pressure

e.



f.

Figures 5. *Variation of cone resistance (q_c) in (Kg/Cm^2), Undrained shear strength (C_u) in (KN/m^2) and Allowable bearing pressure in (KN/m^2) with depth (m) across the site (a - CPT 1, b – CPT 2, c – CPT 3, d – CPT 4, e – CPT 5, f – CPT 6).*

Table 8. *Result of settlement prediction analysis.*

Site section	Depth (m)	L (m)	B (m)	A (m^2)	1.5 Factored D+L' Load (tons)	Unfactored Dead + Live Load (tons)
HOSTEL COMPLEX						
Stream	1.2	39.7	33.7	1337.88	2700	1800
	2.0	39.7	33.7	1337.88	2700	1800
Land	1.2	39.7	33.7	1337.88	2700	1800
	2.0	39.7	33.7	1337.88	2700	1800
4 BEDROOM DUPLEX						
Stream	1.2	20.5	10.0	205	655	436.7
	2.0	20.5	10.0	205	655	436.7
Land	1.2	20.5	10.0	205	655	436.7
	2.0	20.5	10.0	205	655	436.7
2 BEDROOM BLOCK OF FLATS						
Stream	1.2	28.2	11.5	324.3	1270	846.7
	2.0	28.2	11.5	324.3	1270	846.7
Land	1.2	28.2	11.5	324.3	1270	846.7
	2.0	28.2	11.5	324.3	1270	846.7

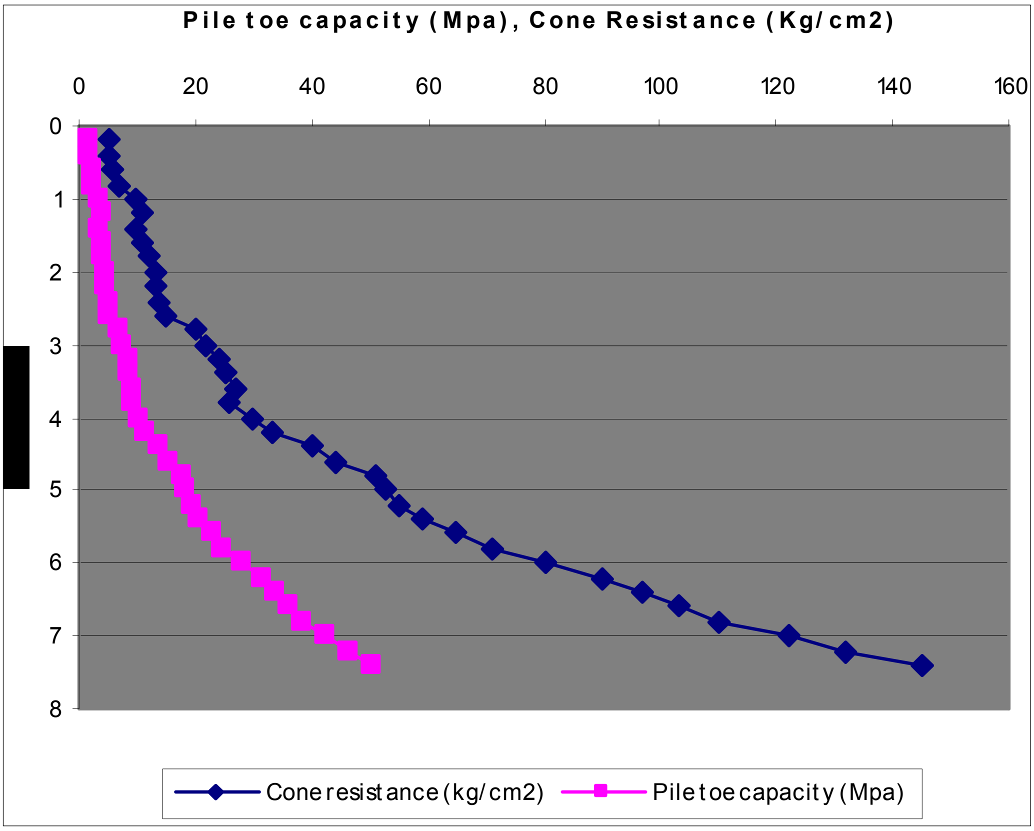
Table 8. *Continue.*

Site section	ΔPav KN/m^2	S_T (cm)	Coefficient of subgrade reaction	% Reduction in total settlement	t_{50} Years	t_{90} Years
HOSTEL COMPLEX						
Stream	4003.2	160.1	25	8.5	2,130.90	9,172.6.
	4003.2	146.5	27.3			
Land	4003.2	211.1	19	16.7	28.82	45.28
	4003.2	175.9	22.8			

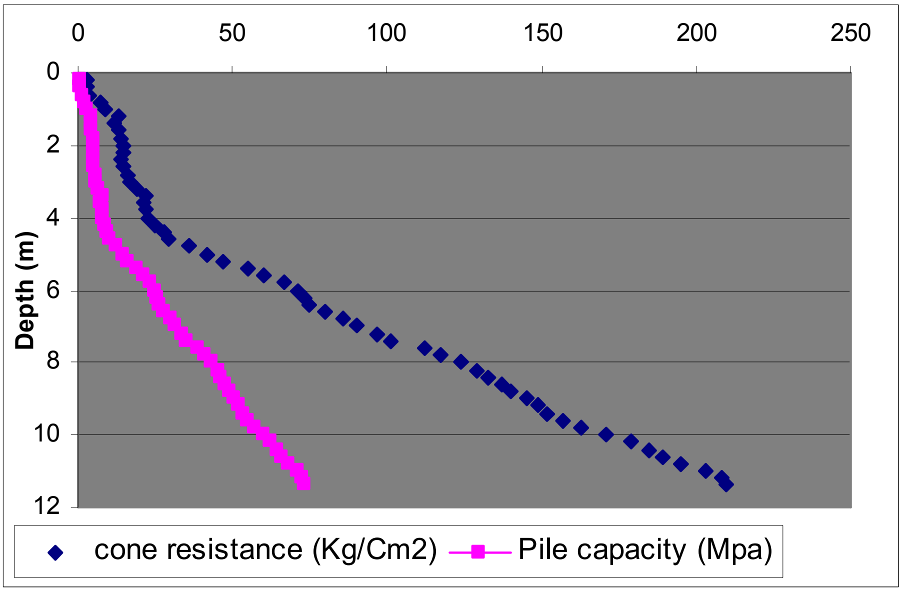
Site section	ΔPav KN/m^2	S_T (cm)	Coefficient of subgrade reaction	% Reduction in total settlement	t_{50} Years	t_{90} Years
4 BEDROOM DUPLEX						
Stream	924.53 924.53	111.9 101.5	8.3 9.1	9.3	492.12	2,118.37
Land	924.53 924.53	180.1 146.9	5.1 6.3	18.4	6.65	28.65
2 BEDROOM BLOCK OF FLATS						
Stream	1822.74 1822.74	134.5 124	13.6 14.7	8	970.2	4,178.34
Land	1822.74 1822.74	190.5 157.3	9.6 11.6	17.4	13.12	56.48

4.4. Foundation Design

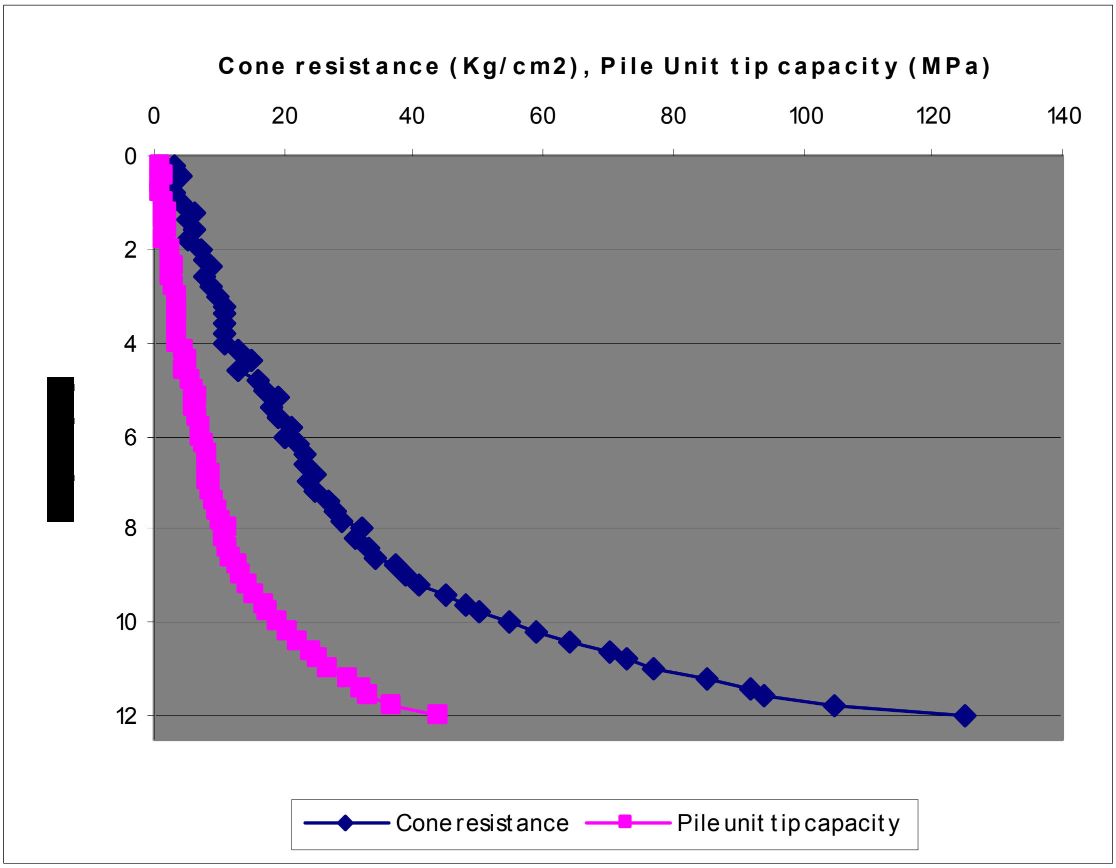
Results of this investigation depicts allowable bearing strength of the foundation subgrade conditions in both the stream channel and adjoining land to be favourable to prevent bearing capacity failure. The most significant design parameter guiding the structural stability and performance is the prevalence of excessive total settlement in both the stream channel and adjoining land. Consequently, raft foundation adopting allowable bearing capacities of 85.84KN/m^2, 82.67KN/m^2, and 85.09 KN/m^2 for the hostel complex, 2-bedroom block of flats and 4-bedroom duplex respectively with uniform top and bottom mats have been recommended. This will provide for uniform time dependent displacement and guide against differential settlement of the structures. Provision should be made for granular bed undercut and cover below and around the foundation to isolate the mat footings from the swelling effects of the clay soils to eliminate swelling in foundation [2]. In consideration of excessive total settlement and rates of settlement, pile foundation is recommended as an option for load transfer to the firm sands below the clays. Unit pile tip capacities presented in figure 6 (a – f) and ranging from 1.05 – 1.6.96MPa in the top clay layer and 6.3 – 114MPa in sand within the stream channel and 2.1MPa – 14.0MPa in clay and 10.15 – 94.5MPa in sand within the adjoining solid ground may be applied for design of straight shafted steel hollow driven piles reinforced with concrete across the site.



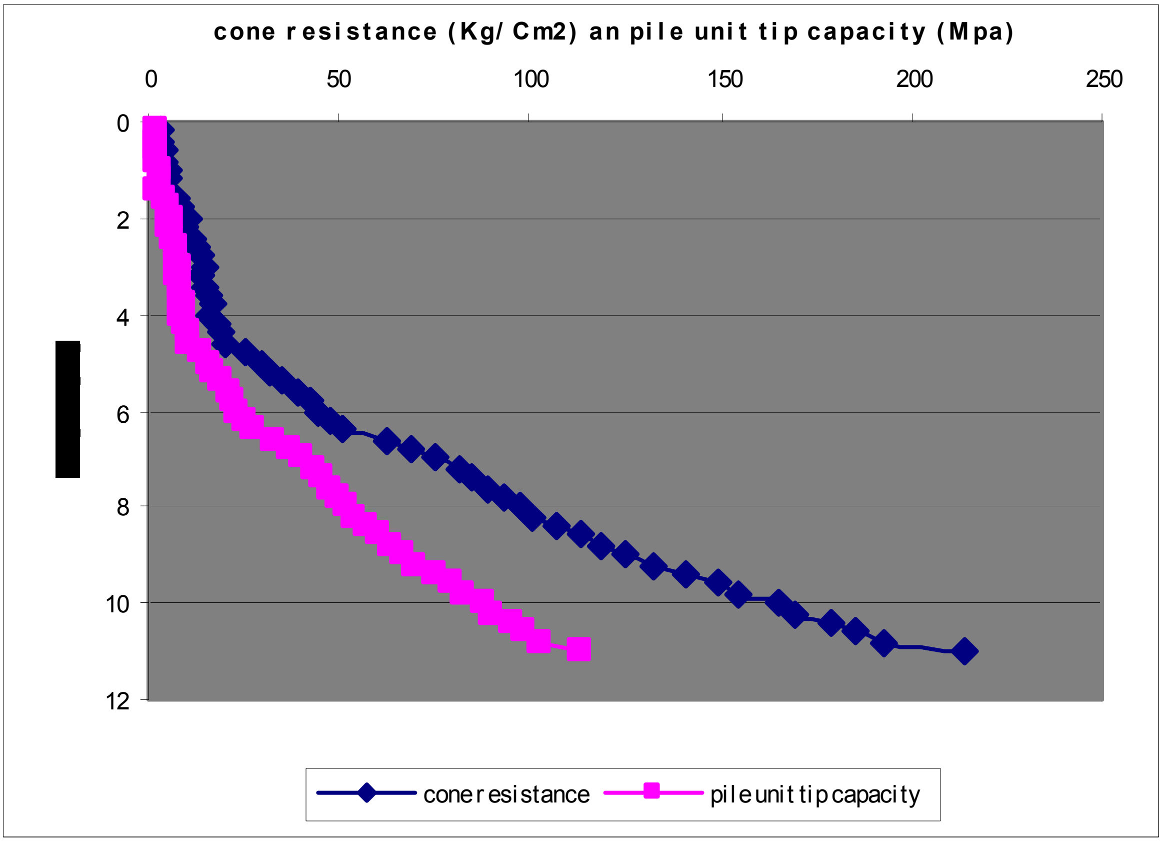
a.


0
50
100
150
200
250
Depth (m)
0
2
4
6
8
10
12
cone resistance (Kg/Cm2)
Pile capacity (Mpa)

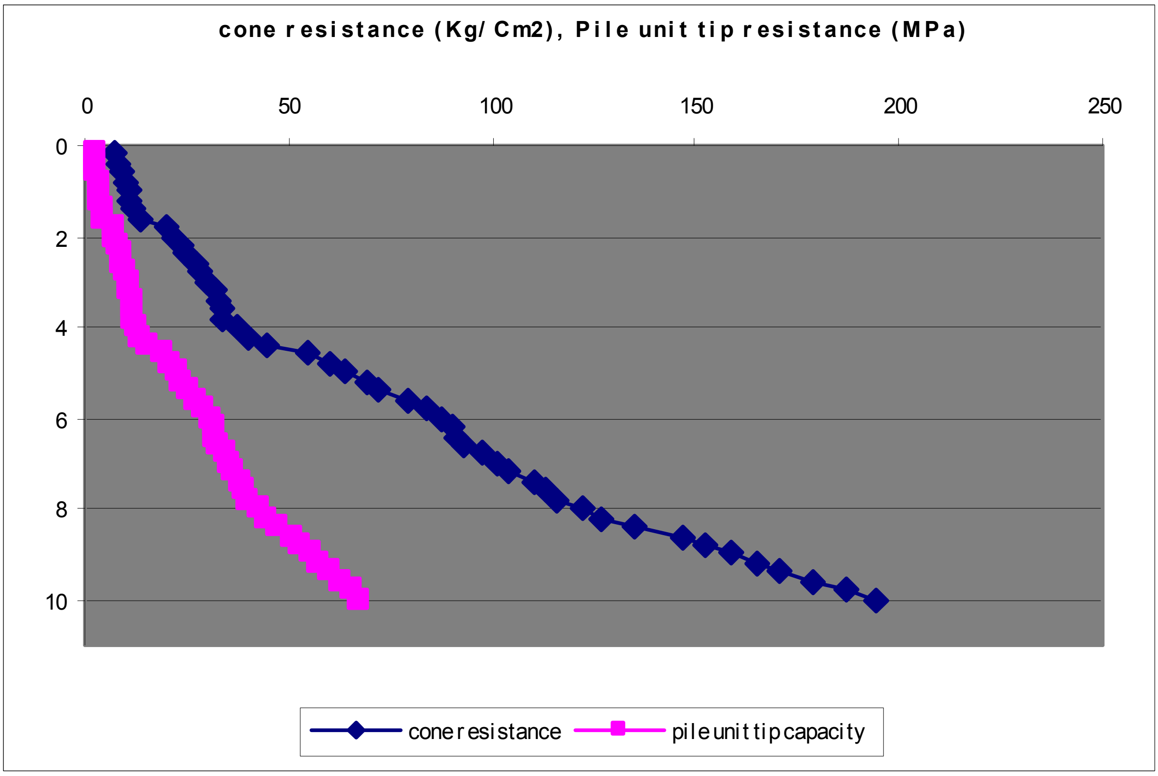
b


Cone resistance (Kg/cm2), Pile Unit tip capacity (MPa)
0
20
40
60
80
100
120
140
0
2
4
6
8
10
12
Cone resistance
Pile unit tip capacity

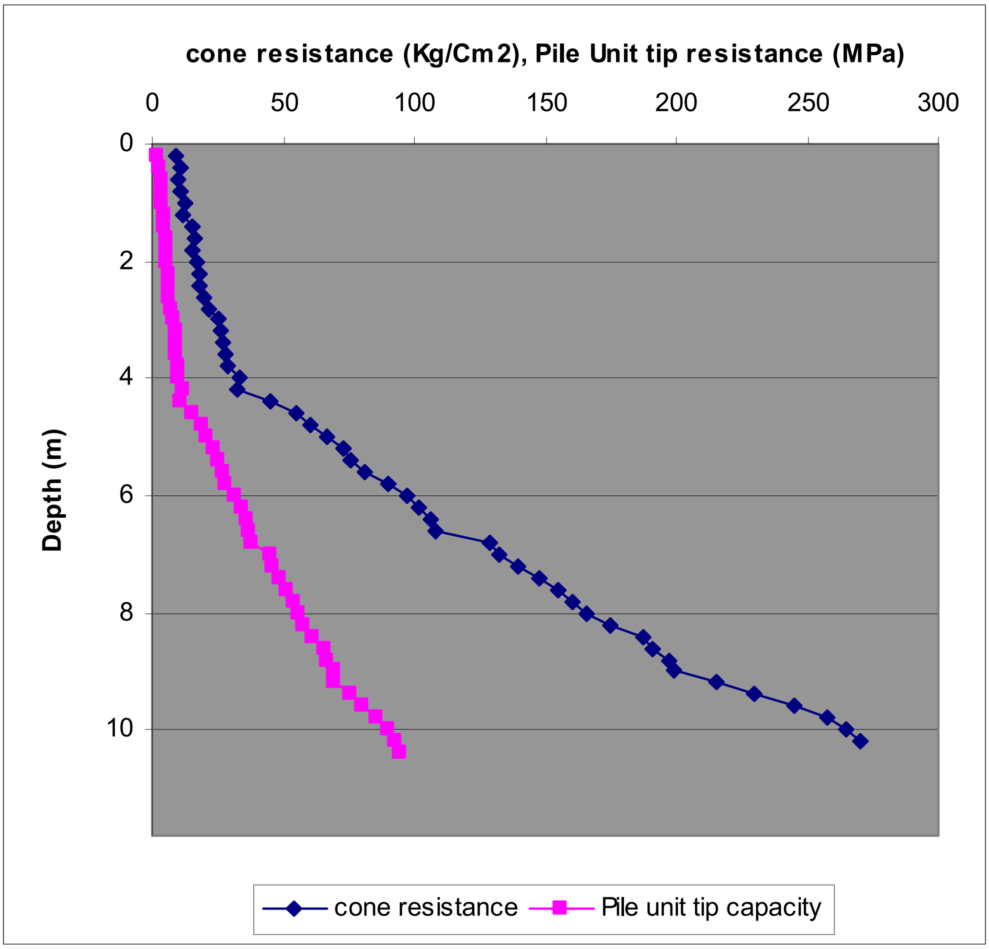
c


cone resistance (Kg/ Cm2) an pile unit tip capacity (Mpa)
0 50 100 150 200 250
0 2 4 6 8 10 12
cone resistance
pile unit tip capacity

d


cone resistance (Kg/ Cm2), Pile unit tip resistance (MPa)
0 50 100 150 200 250
0 2 4 6 8 10
cone resistance
pile unit tip capacity

e



f

Figure 6. *Variation of cone resistance (q_c) in (Kg/Cm^2) and unit pile tip capacity in MPa with depth (m) across the site (a – CPT1, b – CPT 2, c – CPT 3, d – CPT 4, e – CPT 5, f – CPT 6).*

5. Conclusion

The investigation reveals the extra-sensitive to sensitive, high compressibility, weak bearing mangrove swamp soft Kaolin clay layer existing from ground surface to a depth range of 1.0m – 1.1m with swell potential which ranges from 11.45 – 30.64%, swell index from 0.44 – 0.57, activity from 7.0 – 11.0 and swelling pressure 4.776KPa – 4.890KPa to impart excessive total settlement and consequently not suitable for founding the structures. At 2.0m depth a harder clay layer occurs and is proposed to found the structures. Pre-consolidation pressures also reveal the site to be Overconsolidated. Depth compensation of foundation is effective in reduction of total settlement. In view of the excessive total settlement, use of pile foundations has equally been recommended at embedment depth of 11.0m. This underscores the importance of pre- design and construction geotechnical characterization.

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