

## Evaluation of Sub-Soil Geotechnical Properties for Shallow Foundation Design in Onne, Rivers State, Nigeria

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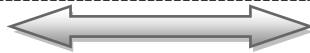
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### -----ABSTRACT-----

*This study aims at establishing the sub-soil types and profile to ascertain the engineering characteristics of the underlying soils in Onne area of Rivers State, Nigeria and recommend appropriate foundation design and construction for large scale ground and elevated water storage facilities in the area. Four borings were accomplished using a percussion rig with the aid of augers. Representative samples were analyzed in the laboratory in accordance with relevant geotechnical engineering standards. Results of the study revealed that within the maximum depth of 15m of boring, the area was found to consist of about 0.2m thick of organic topsoil followed by light brown sandy-clay underlain by a light brown to yellowish brown fine to medium grained, medium dense sand up to 15m depth of investigation. The study also reveals that superficial light brown sandy clay is suitable in quality to carry shallow or raft foundation load development and also has thickness to completely dissipate the influence of such foundations in the area. For such foundations which could be placed between 1.0m and 3.0m depth, an average allowable bearing pressure  $q_a$  of  $100\text{kN/m}^2$  is recommended for the area. The size of projects in the area in terms of loading presupposes that a shallow foundation placed at a minimum of about 900mm below existing ground level may be appropriate. For such foundation and for a maximum consolidation settlement of about 60mm, an allowable bearing pressure of 100kpa is recommended..*

**KEY WORDS:** *Sub-soils, Geotechnical Properties, foundation design, Construction, Niger Delta.*

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### I. INTRODUCTION

It has been observed in recent times, that many property developers in Nigeria ignore drastically the role of geotechnical information in the planning, design, construction, operation as well as safety of civil engineering infrastructures. This neglect results to failure of structures (Annoret *al.*, 1987; Amadiet *al.*, 2012; Youdeowei &Nwankwoala, 2013). Frequent structural failure of civil engineering infrastructures in parts of Port Harcourt Metropolis has become a source of worry to so many persons. It is for this reason that a clear understanding of the occurrence, composition, distribution, geologic history as well as the geotechnical properties of subsurface soils in the area is necessary. Considering the fact that the Eastern Niger Delta is within the coastal zone, geotechnical investigation /considerations are very desirable. The coastal zone which comprises the beach ridges and mangrove swamps is underlain by an alternating sequence of sand and clay with a high frequency of occurrence of clay within 10m below the ground surface. Because of the nearness of these compressible clays to the surface, the influence of imposed loads results to consolidation settlement. The impact of the imposed load is exacerbated by the thickness and consistency of the compressible layer. This, in addition to other intrinsic factors contributes to the failure of civil engineering structures.

The knowledge of the geotechnical characteristics of Onne, Rivers State, Nigeria is very desirable for design and construction of foundation of future civil engineering structures in order to minimize adverse effects and prevention of post construction problems. Some studies have been carried out on geotechnical properties of the sub-soils generally (Nwankwoala &Amadi, 2013; Oke &Amadi, 2008; Okeet *al.*, 2009).For the purpose of generating relevant data inputs for the design and construction of foundations for proposed structures, it is imperative that the site be geo-technically characterized through sub-soil investigation. This paper therefore, aims at establishing significant subsoil types and profile, investigation of the engineering characteristics of all such sub-soils to generate the required data relevant to the foundation design and construction of structures.

**Description of the Study Area**

The study area is located in Onne (Fig.1), Rivers State, Nigeria. The terrain is generally flat or near – flat typical of the Niger Delta region. Generally, the area lies within the flat land/plain of the Niger Delta commonly characterized by superficial stratum of deltaic lateritic materials directly overlying coastal plain sand of the Benin Formation. Generally, the Niger Delta Sedimentary Basin is made up of three rock types. Lithostratigraphically, these rocks are divided into the oldest Akata Formation (Paleocene), the Agbada Formation (Eocene) and the Youngest Benin Formation (Miocene to Recent). Generally, the present knowledge of the geology of the Niger Delta was derived from the works of the following researchers (Reyment, 1965; Short & Stauble, 1967; Murat, 1970; Merki, 1970) as well as the exploration activities of the oil and gas companies in Nigeria. The formation of the so called proto-Niger Delta occurred during the second depositional cycle (Campanian-Maastrichtian) of the southern Nigerian basin. However, the modern Niger Delta was formed during the third and last depositional cycle of the southern Nigerian basin which started in the Paleocene.



The geologic sequence of the Niger Delta consists of three main Tertiary subsurface lithostratigraphic units (Short & Stauble, 1967) which are overlain by various types of Quaternary deposits (Table 1).

Table 1: Quaternary deposits of the Niger Delta (after Etu-Efeotor & Akpokodje, 1990)

Geologic Unit	Lithology	Age
Alluvium	Gravel, Sand, clay, silt	
Freshwater Backswamp, meander belt	Sand, clay, some silt, gravel	
Saltwater Mangrove Swamp and backswamp	Medium-fine sands, clay and some silt	Quaternary
Active/abandoned beach ridges	Sand, clay, and some silt	
Sombreiro-warri deltaic plain	Sand, clay, and some silt	

**II. METHODS OF STUDY**

**Field Investigation**

The field investigation took place within the first week of December, 2012. The field activities among other things included three numbers of hand-auger soil borings distributed across the site and one deep boring at the center. Borings were accompanied by disturbed and undisturbed sampling. As at the time of field investigation, groundwater stood at 9.00m below the ground surface.

### Soil Boring

The three borings were accomplished using a 100mm diameter hand-auger, advanced to a depth of about 5.0m below the existing ground surface while the deep boring was studied by boring one (1) hole (BH4) to a depth of 15m using a Percussion boring rig with the aid of augers, clay cutter or shell. .

### Soil Sampling

During boring, disturbed soil samples were collected at regular intervals of about 1.0m or at significant changes in soil types. Undisturbed samples were also collected in U100 open tube samplers in two randomly selected boreholes at 1.0m and 1.5m depths. Shown in the borehole lithologs are the various sampling types and depths. All samples were examined, identified, classified roughly in the field and all data and information carefully recorded in the borehole logs which are presented in Fig. 2.

### Laboratory Testing

All samples obtained in the field were carefully preserved and transported to the laboratory where they were subjected to more detailed visual inspection and descriptions. Thereafter, representative samples were selected from each stratum for laboratory analysis in accordance with relevant geotechnical engineering standards including BS 1377 (1990). Disturbed samples so selected were subjected to the following laboratory classification tests: (i) Particle size analysis by sieving. (ii) Atterberg limits (liquid and plastic limits). (iii) Natural moisture content. Sieve analysis of cohesive soils particularly involved soaking oven-dried samples in water overnight and washing through sieve No. 200 (75 microns opening) while remnants retained on sieve No. 200 were oven-dried and sieved mechanically. This procedure ensures greater accuracy in determining actual proportion of fines or materials finer than sieve number 200. Such fines were then analysed using the hydrometer method based on Stoke's Law. The undisturbed and mostly cohesive samples collected in the course of boring were subjected to: (i) unconsolidated-undrained or quick triaxial tests at cell pressures of 100kPa, and 300kPa; and (ii) Oedometer consolidation test.

The shear strength parameters  $c_u$  and  $\phi_u$  of the cohesive soil samples were obtained from unconsolidated undrained triaxial test, while shear strength parameter of the granular soils were evaluated from average SPT-number of the respective stratum. The dimensionless bearing capacity factors after Vesic has been adopted in computing the bearing capacity for shallow foundation:

$$q_u = cN_c S_c + \gamma D(N_q - 1) S_q + 0.5 \gamma B N_\gamma S_\gamma$$

Where  $q_u$  = ultimate bearing capacity

$C$  = undrained cohesion

$q$  = effective overburden

$\gamma$  = effective unit weight

$N_c, N_q, N_\gamma$  = Bearing capacity factors

$S_c, S_q, S_\gamma$  = Shape factors

The net allowable,  $q_{n(a)}$ , bearing capacity of the soil has been evaluated with a factor of safety (F.S) of 3.0 and a summary of the net allowable bearing capacity are presented in Table 4. The submerged unit weight introduced in the analysis is to account for the effect of water on the Surcharge component of bearing capacity.

Ultimate pile capacities for axial loading for 406mmØ, 500mmØ and 600mmØ tubular driven steel piles have been estimated for the project. Soil profile and geotechnical data obtained from the site were used for the analysis. The following relationships were used for the calculations:

$$(i) \quad Q_u = Q_b + Q_s$$

$$(ii) \quad Q_u = 9C_u A_b + \alpha C_u A_s \quad \text{for Clay}$$

$$(iii) \quad Q_u = P'_o N_q A_b + k_s P'_o \tan \delta A_s \quad \text{for Sand}$$

Where,  $Q_u$  = Ultimate bearing capacity of pile (kN)

$Q_b$  = Total end bearing (kN)

$Q_s$  = Skin friction resistance (kN)

$C_u$  = Undrained shear strength of the soil (kN/m<sup>2</sup>)

$A_s$  = Surface area of pile shaft (m<sup>2</sup>) =  $\pi BD$

$A_b$  = Base area of pile shaft (m<sup>2</sup>) =  $\pi \frac{B^2}{4}$

$P'_o$  = Effective overburden pressure =  $\frac{\gamma - \gamma_w}{2} D$

$B$  = Diameter of pile

$D$  = Depth of penetration

$\alpha$  = Adhesion factor

$\delta$  = Friction angle between the soil and pile wall =

$k_s$  = Coefficient of horizontal soil stress =  $1.5k_o$

$k_o$  = Coefficient of earth pressure at rest =  $1 - \sin\phi$

The calculated ultimate pile capacities for borehole are shown below. Allowable axial pile capacity has been obtained by applying safety factor of 2.5 for driven piles. Plot of depth of penetration versus allowable pile capacity for the various boreholes is shown in the Appendices,

### Settlement Analysis

Total consolidation settlement ( $\rho_c$ ) has been computed for foundation breadth ( $B$ ) between 1.00-2.50 metres, subjected to a net allowable bearing capacity of 100kN/m<sup>2</sup>. Based on the soil lithology, the thickness of the consolidating layer for a square foundation is taken as the depth to the point where the induced vertical stress ( $\Delta\sigma$ ) is equal to  $0.55q_n$ . The induced vertical stress ( $\Delta\sigma$ ) at the centre of the consolidating layer has been used in computing  $\rho_c$ . The consolidation settlement has been computed from the expression.

$$\begin{aligned} \rho_c &= \mu_g P_{oed} \\ &= m_v \sigma_z H \\ &= m_v \times 0.55q_n \times 1.5B \end{aligned}$$

where  $\mu_g$  = Coefficient which depends on the type of clay

$P_{oed}$  = Settlement as calculated from oedometer test

$m_v$  = Coefficient of volume compressibility

$q_n$  = Net foundation pressure

$B$  = Breadth of foundation.

An  $m_v$  value of 0.20m<sup>2</sup>/MN, which corresponds to the adopted net allowable bearing capacity for the site, has been used in the settlement analysis.

## III. RESULTS

### 3.1. Soil Stratigraphy

The data obtained from the boring, soil sampling, field penetration tests and laboratory gradation tests were interpreted to determine the stratification of subsoils underlying the site. Within the borehole termination depth of about 15m, the site is characterized superficially by light brown sandy-clay underlain by light brown to yellowish brown fine to medium grained, medium dense soil materials.

**IV. ENGINEERING PROPERTIES OF MAJOR SUB-SOILS**

From results of field and laboratory analyses on samples of the soil superficially dominating the site up to borehole termination depth of about 15.0m, the site is characterized by the light-brown sandy- clay. Some selected engineering characteristics for this soil material is summarized in Table 2.

Table 2: Engineering Properties of the soil

Engineering Properties	Min	Max	Mean
Natural moisture content (%)	23.3	28.6	26.0
Bulk Density (kN/m <sup>3</sup> )	18.8	19.4	19.2
Liquid Limit (%)	39	58	49
Plastic Limit (%)	25	28	27
Plasticity Index (%)	14	30	22
Dry unit weight (kN/m <sup>3</sup> )	15.0	15.4	15.2
Effective unit weight (kN/m <sup>3</sup> )	9.0	9.4	9.2
Undrained cohesion (kN/m <sup>2</sup> )	40	44	42
Undrained $\phi$ (degree)	3	6	4
Void ratio	0.33	0.58	0.45
Classification (Unified)	CI	CH	CH

**Physical Properties of Sand:**

**Medium Dense Sand:** The sand is fine to medium grained, well graded, medium dense. The layers are almost of uniform gradation. The relevant engineering parameters of sands are shown in Table 3.

Table 3: Relevant Engineering Parameters

Engineering Properties	Min	Max	Mean
	Effective particle size, $d_{10}$ (mm)	0.02	0.05
Effective particle size, $d_{30}$ (mm)	0.25	0.40	0.35
Effective particle size, $d_{50}$ (mm)	0.50	0.70	0.60
Effective particle size, $d_{60}$ (mm)	0.60	0.90	0.75
Coefficient of uniformity $C_u = d_{60}/d_{10}$	18	30	24
Coefficient of curvature $C_c = d_{30}^2/(d_{60} \cdot d_{10})$	3.56	5.21	4.39
Specific Gravity	2.59	2.79	2.72
Classification (Unified)	SP		

Table 4: Bearing Capacity

BH No	Depth of Sample (m)	Unit Weight $\gamma$ (kN/m <sup>3</sup> )	Angle of friction $\phi$ (degrees)	Undrained Cohesion ' $c_u$ ' (kN/m <sup>3</sup> )	Breadth/Length ration	Allowable bearing capacity ' $q_a$ ' (kN/m <sup>2</sup> )
1	1.00	18.80	5	42	0.50	108
					0.75	116
					1.00	123
	2.00	18.80	5	42	0.50	113
					0.75	120
					1.00	126
	3.00	18.80	5	42	0.50	116
					0.75	125
					1.00	130
2	1.00	19.10	5	40	0.50	104
					0.75	111
					1.00	117
	2.00	19.10	5	40	0.50	108
					0.75	114
					1.00	121
	3.00	19.10	5	40	0.50	112
					0.75	118
					1.00	125

3	1.00	19.40	5	41	0.50 0.75 1.00	107 114 120
	2.00	19.40	5	41	0.50 0.75 1.00	111 117 124
	3.00	19.40	5	41	0.50 0.75 1.00	114 121 128
4	1.00	19.20	6	41	0.50 0.75 1.00	113 120 127
	2.00	19.20	6	41	0.50 0.75 1.00	118 124 131
	3.00	19.20	6	41	0.50 0.75 1.00	122 129 136

Table 5: Axial Pile Capacity for Pile Foundation

Pile Size (mm)	Depth of Penetration (m)	Ultimate Pile Base Resistance (kN)	Ultimate Pile Shaft Resistance (kN)	Ultimate Axial Pile Capacity (kN)
406	5.0	53	33	34
	10.0	53	186	95
	15.0	146	267	166
500	5.0	80	59	56
	10.0	80	229	123
	15.0	222	331	221
600	5.0	116	71	75
	10.0	116	274	156
	15.0	320	398	287

Table 6: Total Consolidation Settlement

Foundation breadth, B (m)	Settlement $\rho_c$ (mm)
1.0	16.50
1.5	24.75
2.0	33.00
2.5	41.25

## V. DISCUSSIONS

Results of the study revealed that the major sub-soils underlying the study area have about 0.2m thick organic topsoil followed by light brown sandy- clay which exists to about 10m depth underlain by light brown to yellowish brown colored medium dense sand up to termination depth of about 15m.

## VI. SHALLOW/RAFT FOUNDATIONS

Shallow or raft foundation is the most appropriate in the area since a soft to firm silty-clay exists up to 10m depth and underlain by another fine to medium grained sand. The calculated bearing capacities could be used in determining the foundation type for structures. An average bearing capacity of 100kN/m<sup>2</sup> is recommended.

## VII. PILE FOUNDATION

**Axial Pile Capacity:** The types of piles to be adopted in the study area are those that are driven (displacement piles) e.g. pre-cast concrete and steel cased piles. However, the versatility and cost effectiveness of a particular type of pile is the governing factor in the recommendation. The safe working load of piles are dependent on factors such as pile diameter, founding depth and method of installation which is usually best determined by a specialist-piling contractor from a pile load test. The values of total consolidation settlement are presented in Table 6.

The computed settlement values presented above for a square foundation size of between 1.00 – 2.50m is less than the allowable maximum settlements suggested by Skempton and MacDonald (1956) for isolated foundations on clays of 65mm, Hence the foundation can sustain the suggested allowable bearing pressure of 100kN/m<sup>2</sup>.

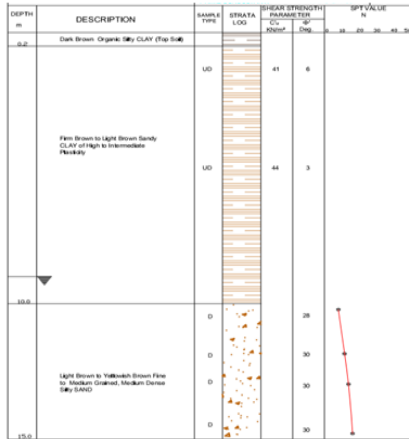


Fig.1: Borehole logs

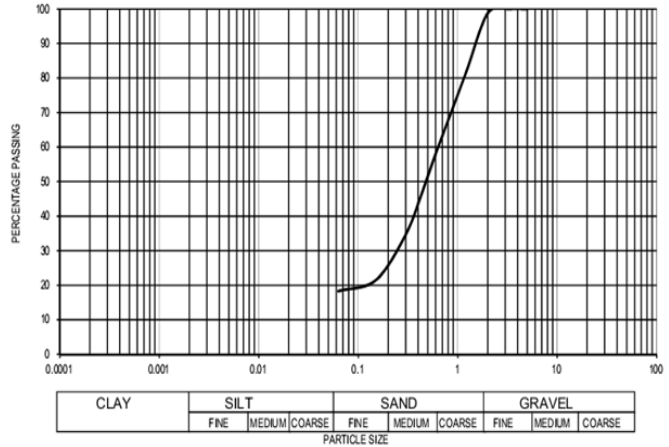


Fig. 2: Particle size Distribution Curves for BH 4 @ 13.00m

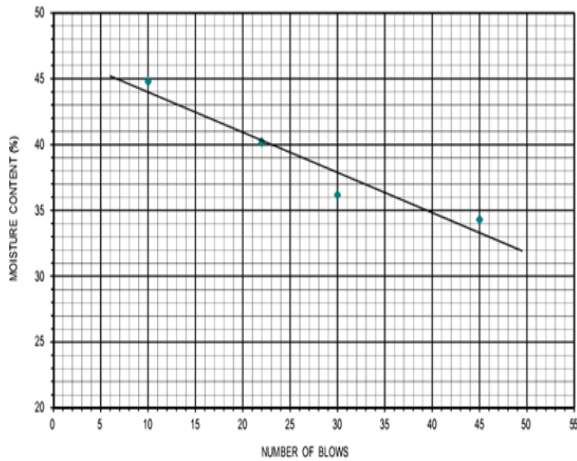


Fig.3: Liquid and Plastic Limit Curve showing intermediate Plasticity Clay @ 7.00m

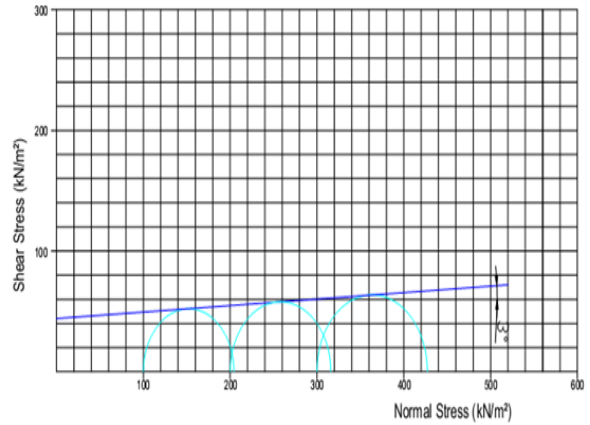
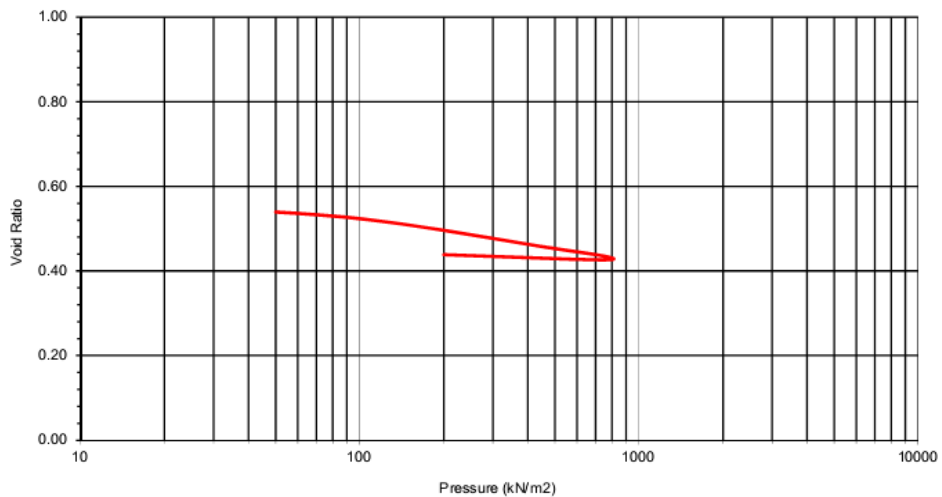


Fig.4: Mohr Circle of the Quick Undrained Triaxial Test for BH 4



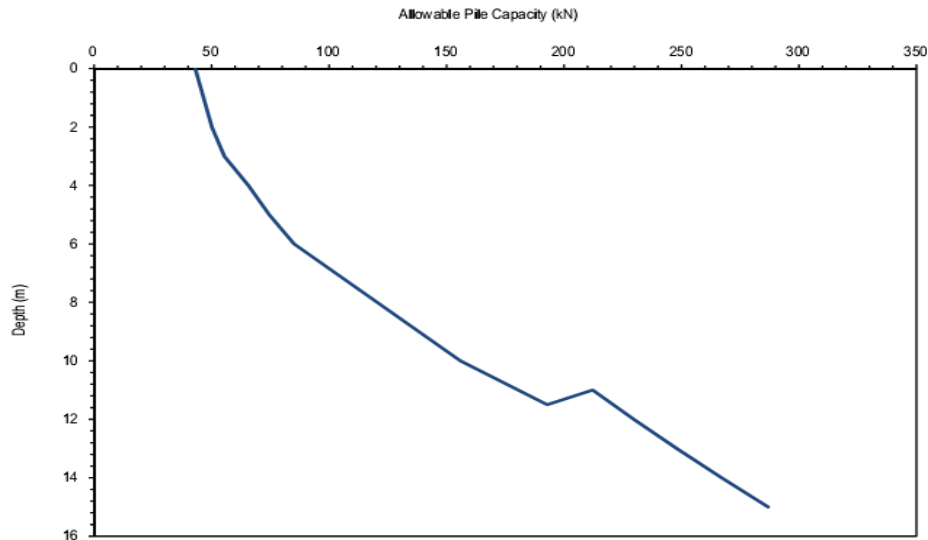


Fig.5: Consolidation Test for BH 2 @ 3.00m

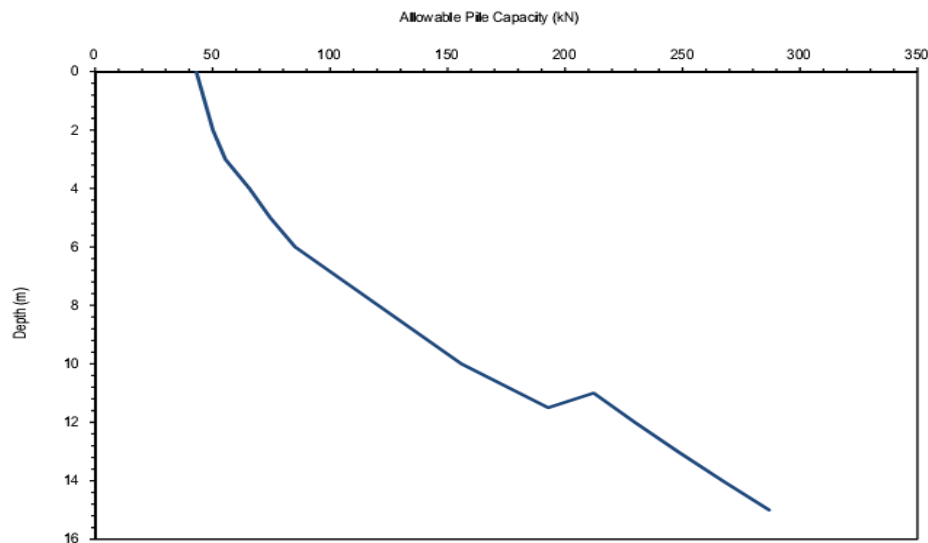


Fig. 6: Axial Pile Capacity Analysis for BH 4

### VIII. CONCLUSIONS

The superficial light brown sandy clay is suitable in quality to carry shallow or raft foundation load from proposed development and also possesses thickness to completely dissipate the influence of such foundations. For such foundations which could be placed between 1.0m and 3.0m depth, an average allowable bearing pressure  $q_a$  of  $100\text{kN/m}^2$  is recommended for the site. The safe working loads of piles are dependent on factors such as pile diameter, founding depth and method of installation which is usually best determined by a specialist-piling contractor from a pile load test. Ultimate pile capacities for axial loading for  $406\text{mm}\varnothing$ ,  $500\text{mm}\varnothing$  and  $600\text{mm}\varnothing$  tubular driven steel piles have been estimated for the project. Soil profile and geotechnical data obtained from the site were used for the analysis. It is generally accepted that pile foundation design is not complete without pile load test. Therefore it is recommended that pile load test be conducted at the site to ensure the design adopted can be safely carried.

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