DIVISION OF SOIL MECHANICS AND FOUNDATION ENGINEERING

PROGRESS THROUGH KNOWLEDGE

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GEOTECHNICAL INVESTIGATION REPORT

NAME OF WORK	:	**REPORT ON THE RECOMMENDATION OF FOUNDATION FOR THE PROPOSED CONSTRUCTION OF NEW REGIONAL OFFICE BUILDING FOR M/s. HLL LIFECARE LIMITED AT PALLIKARANAI, CHENNAI, TAMIL NADU**
CLIENT	:	**Ms KBR Deepti** **Deputy Manager (Civil)** **Infrastructure Development Division** **HLL Lifecare Limited** No 12.First Floor ,100 Feet Velachery- Taramani Road, Near Canara Bank, Vijaynagar Velachery, Chennai-600 042
REFERENCE	:	1) Lr. No.HLL/ NOB-ID-Chennai/Soil Test/2015-16/001, dated 27.03.2015, from M/s HLL Life Care Limited, Chennai 2) Lr. No.HLL/ NOB-ID-Chennai/Soil Test/2015-16/002, dated 02.04.2015, from M/s HLL Life Care Limited, Chennai
JOB No	:	SM&FE / 064 / Consultancy / HLL / 2015
DATE	:	20th April 2015

REPORT ON THE RECOMMENDATION OF FOUNDATION FOR THE PROPOSED CONSTRUCTION OF NEW REGIONAL OFFICE BUILDING FOR *M/s. HLL LIFECARE LIMITED* AT PALLIKARANAI, CHENNAI, TAMIL NADU

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1. INTRODUCTION

M/s. HLL Lifecare Limited, Infrastructure Development Division has proposed to construct a New Regional Office (G+5 framed structure) at Pallikaranai, Chennai, Tamilnadu. The officials of M/s HLL Lifecare Limited (Infrastructure Division), Chennai have approached the Division of Soil Mechanics and Foundation Engineering, Department of Civil Engineering, College of Engineering, Guindy (CEG), Anna University, Chennai - 25 to carryout soil investigation in the proposed site and to recommend the most suitable foundation system for the proposed construction of New Regional Office at Pallikaranai, Chennai, Tamilnadu, vide reference 1st and 2nd cited. Accordingly, the work was taken up and field Investigation was carried out during 07th of April 2015 to 12th of April 2015 under the supervision of Faculty members of Division of Soil Mechanics and Foundation Engineering, Department of Civil Engineering, Anna University, Chennai. This report comprises of the details of soil investigation, analysis of field and laboratory test results on soil and water, and recommendation of most suitable foundation system.

2. SITE CONDITION AND EXPERIMENTAL PROGRAMME

The proposed site is located at Pallikaranai, Chennai. There is a water of pond at 10m distance from the proposed site to the western side. The site is having Velacherry-Tambaram main road to its eastern side, Kamatchi Hospital on the southern side and residential houses on the northern side. The site is made up of 3 m to 4 m uncompacted clayey fill over the existing pond. The site seems to have been existing as pond with stagnated water few years back like the marshy swamp adjacent to the Velachery-Tambaram road to the eastern side. The total number of borehole locations has been decided as '3' numbers as agreed by the officials of M/s. HLL Life Care Limited, Chennai and the Professor & Project Co-ordinator, Division of Soil Mechanics and Foundation Engineering, Department of Civil Engineering, Anna University, Chennai. The location of bore holes is shown in figure 1. The nature of field tests includes standard penetration tests, disturbed soil sampling through split spoon sampler, identification of different soil layers, location of ground water table, complete logging of the borehole etc,. Laboratory investigation consists of classification tests such as grain size distribution, Atterberg limits, specific gravity and free swell index of soil samples, chemical analysis of soil and ground water and point load strength index and geological classification of rock core samples.

After removing the top 0.3 m soil layer, the boreholes were advanced from the existing ground level using rotary boring technique supplemented by Bentonite mud circulation. Mud circulation was used to stabilize the sides and bottom of the boreholes and then to bring the soil cuts to the surface. Bentonite slurry would also help to minimize the disturbance of the soil at the bottom of the borehole while drilling operation is in progress. In general, rotary boring technique with bentonite mud circulation is found most suitable to make exploratory boreholes of diameter 150 mm (IS 1892 : 1979). During boring operation, borehole was always kept full with the drilling mud so as to prevent any disturbance to the soil within the test zone.

Standard penetration tests were conducted at every 1.0 m interval up to 6 m depth from the existing ground level and thereafter 1.5 m interval in all the borehole locations. View of SPT in progress at the site is shown in Plate 1 of Annexure – 1. These SPT tests were carried out using Winch and Cathead device for which the expected energy level is approximately 55% to 60%. The 'N' value resulting from this procedure may be considered as N_{60}. Disturbed soil samples collected through the split spoon sampler were preserved and transported to the soil testing laboratory for detailed identification tests. View of soil sampling through split spoon sampler is shown in Plate 2 of Annexure - 1. During the progress of soil investigation at the proposed site, the site was inspected and supervised by the faculty members of Department of Civil Engineering, CEG Campus, Anna University, Chennai -25 (Plate 3 of Annexure - 1). The rock core samples collected from BH 1 to BH 3 are shown in Plates 4 to 6 of Annexure - 1. Ground water levels were recorded at the end of each boring.

All field and laboratory tests were conducted as per the Indian Standard Testing procedure (SP 36 Part I: 1987 & Part II: 1988). All the field test results recorded in the bore logs are illustrated in figures 2 to 4. The field 'N' values at different depths and samples were collected at these depths are mentioned in the bore logs. Laboratory test results (index and engineering properties) of soil samples collected through split spoon sampler are summarized in table 1 to table 3 of this report.

3. REVIEW OF FIELD AND LABORATORY TEST RESULTS OF SOIL PROFILE

3.1 Borehole Number-1 (BH 1)

The top 2 m soil is an uncompacted grayish black silty clay (CH) which is having sand and fine content (silt and clay) of 10% and 90% respectively. This silty clay is having liquid limit of 69%, plastic limit of 34% and free swell index of 65%, and is classified as clay of high plasticity (CH). Grayish black silty clay was encountered at 2 m and 3 m depths with 'N' values of 4 and 3 respectively. The sand and fine content (silt and clay) of this silty clay layer is 14% - 25% and 75% - 86% respectively (Table 1). This silty clay (CH type) is having liquid limit of 63%, plastic limit of 32% and free swell index of 60% (Table 1). Free swell index value shows that the soil is high expansive nature. At 4m depth, grayish black clayey sand was observed with 'N' value of '1'. The sand and fine content (silt and clay) of this clayey sand layer is 70% and 30% respectively. This clayey sand (SC type) is having liquid limit of 32%, plastic limit of 18% and free swell index of 25%. Grayish black clayey / silty sand was encountered at 5m and

6m depths with 'N' value of '1'. The sand and fine content (silt and clay) of this clayey / silty sand layer is 73% and 27% respectively (Table 1). This clayey / silty sand is having liquid limit of 30%, plastic limit of 16% and free swell index of 20%. Grayish black silty sand was encountered at 7.5 m and 9 m depths with 'N' values of 1 and 6 respectively for 7.5 m and 9 m depths. The sand and fine content (silt and clay) of this silty sand layer is 80% and 20% respectively (Table 1). Grayish brown silty sand (weathered rock sediments) was encountered in the depth of 10.8 m with SPT 'N' > 100 (Hammer rebounded for 54 blows with 13 cm penetration). The NX size double tube core barrel was used to drill and retrieve the rocky stratum from 10.8 m to 15.3 m depth. The observed rock layer is Hypersthene Granite (Charnockite) (Plate 5 of Annexure - 1). The weathering grade of this rock is II (slightly weathered and moderately strong) as per IS 8764:1998 of weathering grade of rock mass. The Point Load Strength Index of this hypersthene granite is 2.17 MPa (2170 kN/m²). The borehole was terminated at 15.3 m depth from the existing ground level. The index and shear strength properties of soils and rock samples collected in BH 1 at different depths are listed in table 1. The ground water table is located at a depth of 4.45 m from the existing ground level. Figure (a) shows the variation of SPT 'N' value of different soil layers with respect to depth in BH 1 location.

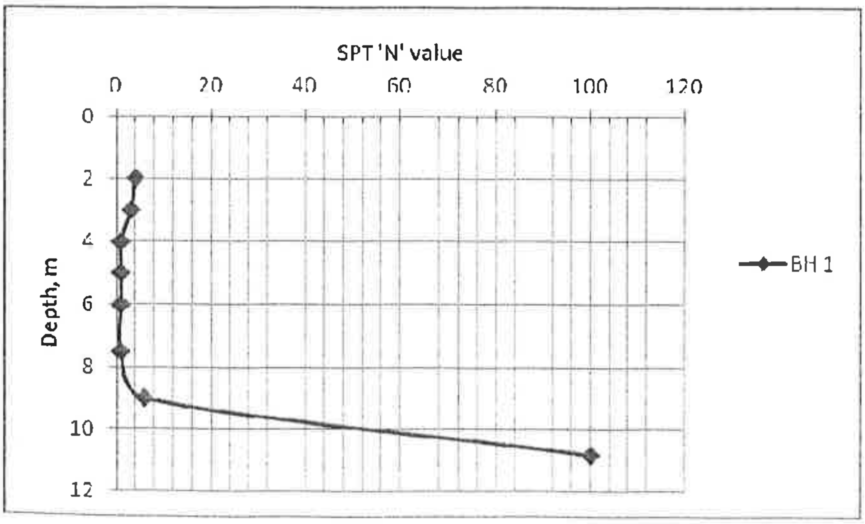

Figure (a) Variation of SPT 'N' value of different soil layers with respect to depth in BH 1

3.2 Borehole Number-2 (BH 2)

The top 3 m soil is filled with uncompacted grayish black silty clay (CH) which is having sand and fine content (silt and clay) of 12% and 88% respectively. Grayish black silty clay was found at 3 m depth with 'N' value of 1. The sand and fine content (silt and clay) of this silty clay layer is 36% and 64% respectively (Table 2). This silty clay (CH type) is having liquid limit of 65%, plastic limit of 35% and free swell index of 60% and this classified as clay of high plasticity. Free swell index value shows that the sandy clay is high swelling nature. In 4m depth, grayish black clayey sand (SC) was observed with 'N' value of 5. The sand and fine content (silt and clay) of this clayey sand layer is 68% and 32% respectively. This clayey sand (SC type) is having liquid limit of 34%, plastic limit of 18% and free swell index of 30%. At the depth of 5 m, grayish black clayey/ silty sand was observed with SPT 'N' = 0 and this clayey/silty sand layer is having sand and fine content (silt and clay) of 75% and 25% respectively and having liquid limit of 31%, plastic limit of 16% and free swell index of 25%.

Grayish black clayey / silty sand was encountered at 6 m and 7.5 m depths with 'N' value of 2 and 6 respectively. This clayey / silty sand layer is having liquid limit of 27%, plastic limit of 15% and free swell index of 20%.The sand and fine content (silt and clay) of this clayey / silty sand layer is varying 78% - 79% and 21% - 22% respectively. Grayish black silty sand was found at 9 m depths with 'N' value of 4.This silty sand is non plastic. This silty sand layer is having sand and fine content (silt and clay) of 88% and 12% respectively. Grayish brown silty sand (weathered rock sediments) was observed in the depth of 10.5 m with SPT 'N' > 100 (Hammer was rebound for 60 blows with 15 cm penetration). The NX size double tube core barrel was used to drill and retrieve the rocky stratum from 10.5 m to 15.8 m depth. The borehole was terminated at 15.8 m depth from the existing ground level. The observed rock layer is Hypersthene Granite (Charnockite) (Plate 6 of Annexure - 1). The weathering grade of this rock is II (slightly weathered and moderately strong) as per the ISI scale of weathering grade of rock mass. The Point Load Strength Index of this hypersthene granite is 3.89 MPa (3890 kN/m^2).The index and shear strength properties of soils and rock samples collected at BH 2 in different depths are listed in table 2. The ground water table is located at a depth of

4.65 m from the existing ground level. Figure (b) shows the variation of SPT 'N' value of different soil layers with respect to depth in BH 2 location.

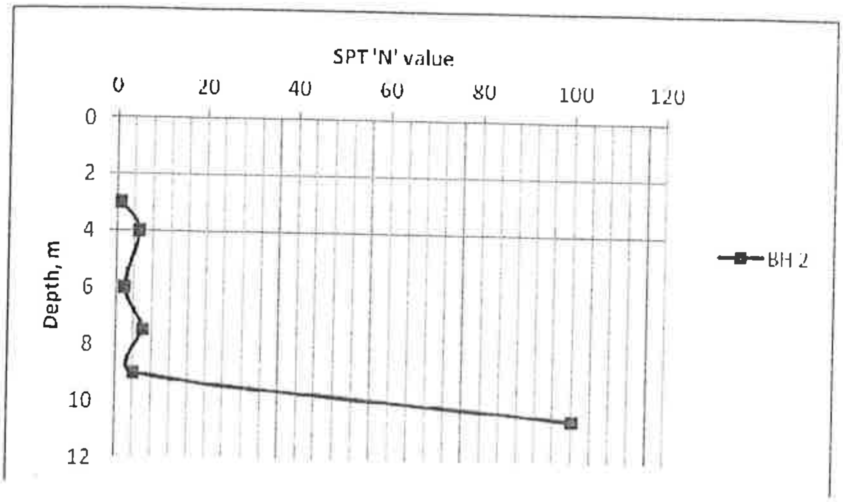

Figure (b) Variation of SPT 'N' value of different soil layers with respect to depth in BH 2

3.3 Borehole Number-3 (BH 3)

In BH 3, the top 3 m soil is an uncompacted grayish black silty clay (CH) which is having sand and fine content (silt and clay) of 10% and 90% respectively. Grayish black silty clay encountered at 3 m and 4 m depths with 'N' values of 4 and 3 respectively. The sand and fine content (silt and clay) of this silty clay layer is varying from 20% - 30% and 70% - 80% respectively. This silty clay is having liquid limit varying from 60% - 62%, plastic limit of 34% and free swell index of 60% (Table 3) and, the clay layer is classified as CH type. The free swell index value implies the high expansive nature of silty clay layer. Grayish black clayey sand was found in depths of 5 m and 6 m with 'N' values of 2 and 1 respectively. The sand and fine content (silt and clay) of this clayey sand layer is varying from 73% - 74% and 26% - 27% respectively (Table 3). This clayey sand (SC type) is having liquid limit of 34%, plastic limit of 18% and free swell index of 30%. Grayish black silty clay was found in 7.5 m and 9 m depths with 'N' values of 16 and 9 respectively. In the depth of 7.5 m, the silty clayey layer is having sand and fine content (silt and clay) of 12% and 88% respectively, and the liquid limit of 72%, plastic limit of 35% and free swell index of 70% (Table 3). At the depth of 9 m, silty clayey layer is having sand and fine content (silt and clay) of 45% and 55% respectively and liquid limit of 60%, plastic limit of 32% and free swell index of 60% (Table 3).

Brown silty sand (weathered rock sediments) was encountered in the depth range of 10.5 m to 12 m with SPT 'N' > 100 (Hammer was rebound for 89 blows with 22 cm penetration at 10.5 m depth and at 12 m depth, hammer was rebound for 51 blows without any penetration). The NX size double tube core barrel was used to drill and retrieve the rocky stratum from 12 m to 16.5 m depth. The observed rock layer is Hypersthene Granite (Charnockite) (Plate 6 of Annexure - 1). The weathering grade of this rock is II (slightly weathered and moderately strong) as per the ISI scale of weathering grade of rock mass. The Point Load Strength Index of this hypersthene granite is 2.05 MPa (2050 kN/m^2).The borehole was terminated at 16.5 m depth from the existing ground level. The index and shear strength properties of soils and rock samples collected in BH 3 at different depths are listed in table 3. The ground water table is located at a depth of 4.25 m from the existing ground level. Figure (c) shows the variation of SPT 'N' value of different soil layers with respect to depth in BH 3 location.

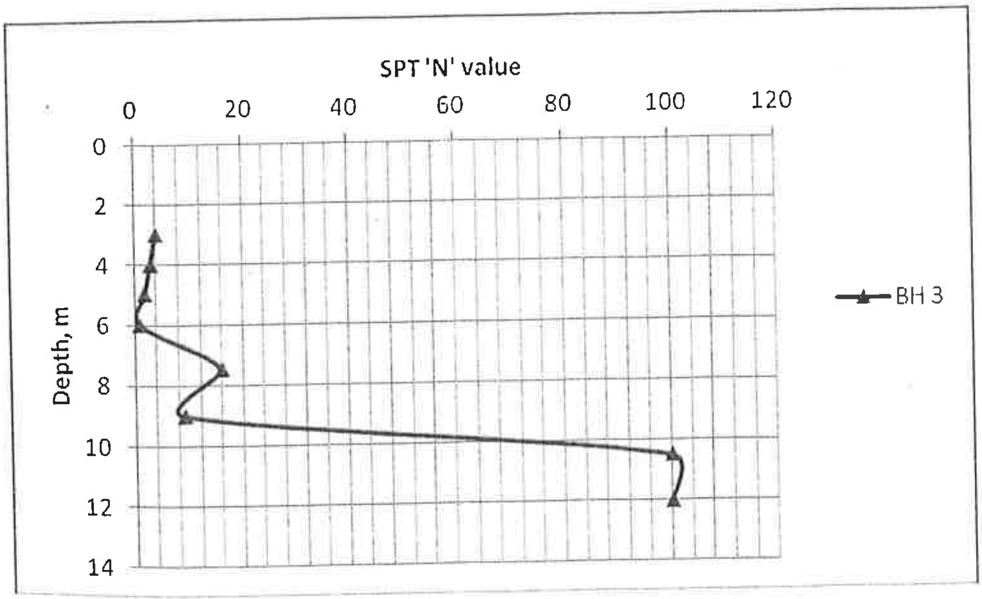

Figure (c) Variation of SPT 'N' value of different soil layers with respect to depth in BH 3

Figure (d) shows the variation of SPT 'N' value of different soil layers with respect to depth in BH 1 to BH 3 of the proposed site. It could be seen that the variations of SPT 'N' values are almost similar in all the bore holes, indicates the homogeneity of soil layer.

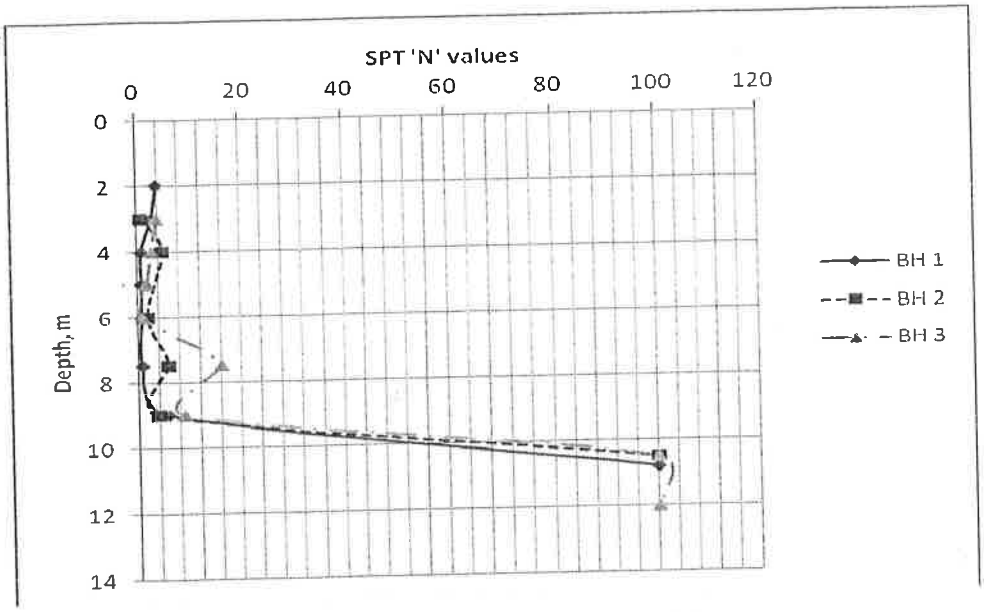

Figure (d) Variation of SPT 'N' value of different soil layers with respect to depth in BH 1, BH 2 and BH 3

4.0 SHEAR STRENGTH PARAMETERS, CPT 'q_c' VALUES AND ELASTIC MODULUS 'E_s' VALUES OF DIFFERENT DEPTHS WITH RESPECT TO SPT 'N' VALUES

Table I to Table III show the SPT 'N' value, correlated CPT 'q_c' value (IS 2911 - 2010, Part 1, Sec.1), correlated Elastic modulus 'E_s' value (Schmertmann, 1970) and shear strength parameters of the borehole 1 to 3 of the proposed site.

TABLE I SPT "N' values, Correlated CPT 'q_c' Values, Shear Strength Parameters, Elastic Modulus (E_s) for BH 1 Location

Depth (m)	SPT 'N' Value	CPT 'q_c' Value (kN/m²)	Elastic Modulus 'Es' Value(kN/m²)	Shear Strength Parameters	
				'c' (kN/m²)	Φ in Degrees
2	4	800	1600	25	0
3	3	600	1200	18	0
4	1	400	800	0	19
5	1	400	800	0	19
6	1	400	800	0	19
7.5	1	400	800	0	19
9	6	2400	4800	0	26
10.8	> 100	100000	766000	0	45
12.3	> 100	100000	766000	0	45
13.8	> 100	100000	766000	0	45
15.3	> 100	100000	766000	0	45

TABLE II SPT "N' values, Correlated CPT 'q_c' Values, Shear Strength Parameters, Elastic Modulus (E_s) for BH 2 Location

Depth (m)	SPT 'N' Value	CPT 'q_c' Value (kN/m²)	Elastic Modulus 'E_s' Value (kN/m²)	Shear Strength Parameters	
				'c' (kN/m²)	Φ in Degrees
3	1	200	766	6	0
4	5	2000	3830	0	25
5	0	0	0	0	15
6	2	800	1532	0	21
7.5	6	2400	4596	0	26
9	4	1600	3064	0	24
10.5	>100	100000	766000	0	45
12	>100	100000	766000	0	45
13.5	>100	100000	766000	0	45
15	>100	100000	766000	0	45
15.8	>100	100000	766000	0	45

TABLE III SPT "N' values, Correlated CPT 'q_c' Values, Shear Strength Parameters, Elastic Modulus (E_s) for BH 3 Location

Depth (m)	SPT 'N' Value	CPT 'q_c' Value (kN/m^2)	Elastic Modulus 'E_s' Value (kN/m^2)	Shear Strength Parameters	
				'c' (kN/m^2)	Φ in Degrees
3	4	800	3064	25	0
4	3	600	2298	18	0
5	2	800	1532	0	21
6	1	400	766	0	19
7.5	16	3200	12256	100	0
9	9	1800	6894	56	0
10.5	>100	100000	766000	0	45
12	>100	100000	766000	0	45
13.5	>100	100000	766000	0	45
15	>100	100000	766000	0	45
16.5	>100	100000	766000	0	45

In the proposed site, the top 3 m to 4 m layer is uncompacted silty clay made up soil over the existing pond. The observed SPT 'N' values and shear strength parameters are very low in shallow depth of 3 m to 7.5 m in all the three borehole locations.

5.0 SUMMARY OF INFERENCE FROM BOREHOLES

The following salient points are summarised from the discussion made in the section 3 and 4.

1) The site is located over the made up soil of silt / sandy clay of depth vary from 0 to 3m with high plasticity (CH type). The soil layer is having very poor 'N' value. In general, clay of high plasticity and high swelling nature are problematic soil and as such they are unfit to act as a good foundation material, because of its higher compressibility and expansive nature.
2) The soil in the depth range of 3m to 7.5m are very loose silty / clayey sand with average 'N' value of '3'. The very low 'N' value only indicates that the soil layer is having poor shear strength and bearing capacity.

3) The average ground water table is located at 4.45m from the existing ground level. It is expected to reach much higher level, i.e. 1 to 1.5m from the ground level during seasonal changes.
4) A pond of water is found to exist very close (less than 10m distance) to the western side of proposed site. Further, there is a marshy land swamp protected by Department of Forest, Tamil Nadu to the eastern side of the proposed site, which is less than 100m distance. These water bodies on the eastern and western side of the site would eventually raise the ground water table especially in situation of high rainfall and poor drainage characteristics of top clayey layer. In general, the presence of water table at shallow depth would further reduces the strength of the soil and thus leading excessive settlement.
5) The results of chemical analysis of ground water and soil are shown in table 4 & 5. The pH of the soil 8.07 shows the soil is mild alkaline. The chloride and sulphates are 655 mg / kg and 98.5 mg / kg respectively (table 4).
6) The TDS, sulphate and chloride of the ground water sample are on the higher side (table 5).

6.0 RECOMMENDATION OF SUITABLE FOUNDATION SYSTEM FOR THE PROPOSED G+5 FRAMED STRUCTURE OF NEW OFFICE BUILDING

On examining the discussions of section 3, 4 and 5, the provision of shallow foundation is not feasible for the proposed construction of G+5 structure for the following reasons.

- The average 'N' value is hardly '3' from the existing ground level even upto 7.5m, which only depicts that the soil is having very poor shear strength in turn low bearing capacity.
- Even though the exact loading of the G+5 proposed structure is not provided by the M/s. HLL Lifecare Limited, Chennai, the likely range of column load is expected to vary between 150 t to 200 t. The likely footing pressure for an assumed column load of 150 t and foundation width of 2m, is 75 t/m^2 and whereas the allowable bearing pressure of the soil is only 3 t / m^2 (for an allowable settlement of 25mm). Hence, isolated footing is not suitable for the proposed construction.

- Further, in an attempt to provide a raft foundation at a depth of 5.0m from the existing ground level, for an assumed raft of size 14 m x 32 m and the total column load 5400 t (or) 54,000 kN (36 columns and each column of 150 t), the settlement computed as per the IS code 8009 - 1976 and De beer method is more than 200mm which is much higher than the permissible settlement value of 75mm. Hence, the proposal of providing raft foundation is also not advisable. Further, in order to construct a raft of 14 m x 32m size at 5m depth, the soil layer of 14m x 32m x 5m to be excavated is highly uneconomical.
- Raft foundation (size of raft is 14m x 32m) at a depth of 5m may be however possible to suggest, provided if the soil below 5m to 9m depth is improved with ground improvement techniques such as installation of stone column or compaction pile. Design of compaction pile or stone column has to be arrived for the upcoming exact structural load.

Considering the very low bearing capacity and excessive settlement criteria, shallow foundation either isolated or raft is not recommended for the proposed New Regional Office (G+5 framed structure), M/s. HLL Lifecare Limited, Pallikaranai, Chennai. Instead of having raft foundation on the compaction pile or stone column, it is advisable to opt for pile foundation which would be ideal for the proposed site condition.

7.0 COMPUTATION OF SAFE PILE CARRYING CAPACITY

In the absence of shallow foundation, pile foundation can be provided. As the exact total structural load of the super structure has not been made available to this office of Division of Soil Mechanics and Foundation Engineering, CEG, Anna University, Chennai - 25, attempts are made to suggest pile foundation for varying pile diameter. For computing the load carrying capacity of pile, Indian Standard IS 2911 (Part 1/Sec. 2) 2010 method and Meyerhof 1959 formula (Based on SPT 'N' Value) are used. The diameters of piles are assumed as 400 mm, 500 mm, 600 mm and 750 mm and the length of the pile is taken as 12 m. The least shear strength parameters and least 'N' values were taken as the criteria from Table I to Table III out of the three borehole locations for the design of pile foundation to the proposed site. Accordingly, the least shear strength parameters and least 'N' values of soil layers were observed in BH - 2. Hence, safe pile carrying capacity computations were made for BH-2 borehole data.

The Indian Standard IS 2911 (2010) specifies that the base resistance should not exceed 1000 to 1100 t/m^2 for bored cast-in-situ piles and 1500 t/m^2 for precast driven piles. The end bearing capacity is also computed based on Cole and Stroud (1977) approach by providing '1 D' in Granite Rock Strata as pile socketing (where D is pile diameter in mm). The shear strength of rocky stratum (point bearing shear strength is the point load strength index of the rock samples) was used to determine the ultimate end bearing resistance of the pile.

For calculating the skin friction along the pile length of 12 m, the design 'N' values and shear strength parameters for the respective depths were used. The top 4 m depth soil is uncompacted silty clayey. The observed 'N' value is '0' at 5 m depth in BH 2 location. This uncompacted fill and very loose clayey / silty sand layer may induce settlement of surrounding soil which in turn may cause negative skin friction. Hence, the safe frictional resistance is computed beyond 6 m depth and the negative skin friction value was suitably deducted in the computation of safe bearing capacity of piles. The safe end bearing resistance and safe skin frictional resistance are together added for different pile diameter and the total safe load carrying capacity of pile thus computed is shown in Table IV. The calculations of pile carrying capacity by static formula (IS 2911 : 2010), Meyerhof formula (1959) and Cole and Stroud (1977) are presented in Annexure 2. The factor of safety of 3 has been used for determining safe end bearing and also frictional capacity of bored cast-in-situ piles.

The uplift capacity of a pile is given by sum of the frictional resistance and the weight of the pile (buoyant or total as relevant) as per Section 6.3.2 of IS 2911 (Part 1/Sec. 2) 2010. As per codal provisions, the recommended factor of safety is 3.0 in the absence of any pull out test results and 2.0 with pullout test results. Uplift capacity can be obtained from static formula by ignoring end-bearing, but adding weight of the pile (buoyant or total as relevant). The safe uplift capacity of pile varying diameter is given in Table V.

TABLE IV Safe Load Carrying Capacity of Pile for varying Diameter for the Length of 12m

Pile Diameter	Safe Pile Capacity (Tons) Meyerhof 1959	Safe Pile Capacity (Tons) (IS 2911 Part-I 2010 Static Formula)	Safe Pile Capacity (Tons) Cole and Stroud (1977) Formula) - 1 D Socketed into Granite Rock Strata
400 mm	60	63	117
500 mm	73	77	144
600 mm	103	108	205
750 mm	158	164	318

TABLE V Safe Uplift Capacity of Pile for varying Diameter for the Length of 12 m (IS 2911 (Part 1/Sec. 2) 2010

Pile Diameter	Safe Uplift Capacity (Tons) IS 2911 (Part 1/Sec. 2) 2010
400 mm	11
500 mm	13
600 mm	17
750 mm	24

8. CONCLUSIONS AND RECOMMENDATIONS

In view of the discussions made in section 4, 5, 6 and 7, the following conclusions are recommended for the proposed construction of G+5 new regional office building of HLL life care limited at Pallikaranai, Chennai, Tamilnadu

1) As observed SPT 'N' values and shear strength parameters are very low in the depth range of 0.5 m to 7.5 m from the existing ground level in all the 3 borehole locations, the shallow foundation is not recommended at this site.
2) It is recommended to provide pile foundation for the proposed construction of New Regional Office at Pallikaranai, Chennai. The recommended safe vertical pile carrying capacity and uplift capacity of the 12 m length of 'bored cast in-situ pile' for varying diameter is shown in <u>Table VI</u>.

3) The bored cast in-situ pile shall be terminated between 12 m to 13.5 m depth in the rocky stratum.

4) It is recommended to socket the bored cast in situ pile into the rock stratum for a minimum length of 1 D ('D' is the diameter of pile).

5) Pile load test shall be conducted to ensure the designed pile carrying capacity. The minimum grade of concrete for pile foundation shall be M 30 and M 40.

6) As the existing ground water exceeds the permissible value of chloride and soluble salt content the ground water shall not be used for the construction purpose.

7) As the soil layer from 0 to 4m exhibits very high plasticity characteristics, it shall not be used for backfilling purposes.

Table VI Recommended Safe Load Carrying Capacity and Uplift Capacity of Pile for varying Diameter for the Length of 12 to 13.5 m at Pallikaranai, Chennai

Pile Diameter (mm)	Pile Capacity (Tons)	Uplift Capacity (Tons)
400	61	11
500	75	13
600	105	17
750	160	24

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[4] IS 8009 (Part I) (Reaffirmed 1998), "Code of Practice for Calculation of Settlements of Foundations", Bureau of Indian Standards.

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[7] Schmertmann, J.H. (1970) "Static cone to compute static settlement over sand", Journal of the Soil Mechanics and Foundations Division, ASCE, 96(3): 1011-1043.

[8] Cole, K.M. and Stroud, M.A. (1977), Rock Socket Piles at Coventry Point – Piles in Weak Rocks, Proc. Inst. of Civil Engineers, London.

[9] IS 8764-1998 "Method for Determination of Point Load Strength Index of Rocks"

[10] De Beer, E. and Martens .A (1957), "Method of computation of an upper limit for the influence of heterogeneity of sand layers in settlement of bridges", 4th ICSMFE, London. Volume-1, pp.236 - 246

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PROJECT : Proposed Construction of HLL Regional Office Building, Pallikaranai

H NO	1		DATE OF START	07.04.2015
ITE	Pallikarani		DATE OF COMPLETION	07.04.2015
IA OF BORING	150 mm		GROUND WATER LEVEL	4.45 m
YPE OF BORING	Rotary (Calyx)		RL	-

Depth below EGL (m)	Soil / Rock Profile	Description / Classification of Soil / Rock	Standard Penetration Test (SPT) / UDS / Core Drilling				Relative Density/ Consistency
			15	30	45	N	
0 - 2.00		Grayish Black Silty Clay (CH) - Filled up Soil	Filled up Soil				
2.00		Grayish Black Silty Clay (CH)	2	2	2	4	Soft
3.00		Grayish Black Silty Clay (CH)	2	1	2	3	Soft
4.00		Grayish Black Clayey Sand (SC)	0	0	1	1	Very Loose
5.00		Grayish Black Clayey / Silty Sand	0	0	1	1	Very Loose
6.00		Grayish Black Clayey / Silty Sand	0	0	1	1	Very Loose
7.50		Grayish Black Silty Sand	0	0	1	1	Very Loose
9.00		Grayish Black Silty Sand	1	2	4	6	Loose
10.80		Grayish Brown Silty Sand - Weathered Rock sediments	40	54 (13 cm) Rebound		> 100	Hard
12.30		Brown Weathered Rock	CR = 5%, RQD = 0%			> 100	Hard
13.80		Brown Weathered Rock	CR = 10%, RQD = 0%			> 100	Hard
15.30		Brown Weathered Rock	CR = 5%, RQD = 0%			> 100	Hard

Borehole Termination Depth is 15.3 m from the Existing Ground Level

CR – Core Recovery, UDS – Undisturbed Sample, RQD – Rock Quality Designation

Figure 2 Field log profile of BH-1

PROJECT : Proposed Construction of HLL Regional Office Building, Pallikaranai

IH NO	2	DATE OF START	08.04.2015
ITE	Pallikarani	DATE OF COMPLETION	09.04.2015
IA OF BORING	150 mm	GROUND WATER LEVEL	4.65 m
YPE OF BORING	Rotary (Calyx)	RL	-

Depth below EGL (m)	Soil / Rock Profile	Description / Classification of Soil / Rock	Standard Penetration Test (SPT) / UDS / Core Drilling				Relative Density/ Consistency
			15	30	45	N	
0 - 3.00		Grayish Black Silty Clay (CH) - Filled up Soil	Filled up Soil				
3.00		Grayish Black Silty Clay (CH)	0	0	1	1	Soft
4.00		Grayish Black Clayey Sand (SC)	1	2	3	5	Loose
5.00		Grayish Black Clayey / Silty Sand	0	0	0	0	Very Loose
6.00		Grayish Black Clayey / Silty Sand	0	0	2	2	Very Loose
7.50		Grayish Black Clayey / Silty Sand	1	1	5	6	Loose
9.00		Grayish Black Silty Sand	1	1	3	4	Loose
10.50		Grayish Brown Silty Sand - Weathered Rock sediments	45	60 (15 cm) Rebound		> 100	Hard
12.00		Brown Weathered Rock	CR = 3%, RQD = 0%			> 100	Hard
13.50		Brown Weathered Rock	CR = 5%, RQD = 0%			> 100	Hard
15.00		Brown Weathered Rock	CR = 8%, RQD = 0%			> 100	Hard
15.80		Brown Weathered Rock	CR = 10%, RQD = 0%			> 100	Hard
Borehole Termination Depth is 15.8 m from the Existing Ground Level							

CR – Core Recovery, UDS – Undisturbed Sample, RQD – Rock Quality Designation

Figure 3 Field Log Profile of BH-2

PROJECT : Proposed Construction of HLL Regional Office Building, Pallikaranai

H NO	3	DATE OF START	10.04.2015
ITE	Pallikarani	DATE OF COMPLETION	12.04.2015
IA OF BORING	150 mm	GROUND WATER LEVEL	4.25 m
YPE OF BORING	Rotary (Calyx)	RL	-

Depth below EGL (m)	Soil / Rock Profile	Description / Classification of Soil / Rock	Standard Penetration Test (SPT) / UDS / Core Drilling				Relative Density/ Consistency
			15	30	45	N	
0 - 3.00		Grayish Black Silty Clay (CH) - Filled up Soil	Filled up Soil				
3.00		Grayish Black Silty Clay (CH)	2	2	2	4	Soft
4.00		Grayish Black Silty Clay (CH)	0	1	2	3	Soft
5.00		Grayish Black Clayey Sand (SC)	0	0	2	2	Very Loose
6.00		Grayish Black Clayey Sand (SC)	0	0	1	1	Very Loose
7.50		Grayish Black Silty Clay (CH)	5	7	9	16	Stiff
9.00		Grayish Black Silty Clay (CH)	3	3	6	9	Stiff
10.50		Brown Silty Sand - Weathered Rock sediments	30	35	54 (7 cm)	> 100	Hard
12.00		Brown Weathered Rock sediments	51 (0 cm) Hammer Rebound			> 100	Hard
13.50		Brown Weathered Rock	CR = 3%, RQD = 0%			> 100	Hard
15.00		Brown Weathered Rock	CR = 5%, RQD = 0%			> 100	Hard
16.50		Brown Weathered Rock	CR = 5%, RQD = 0%			> 100	Hard
Borehole Termination Depth is 16.5 m from the Existing Ground Level							

CR – Core Recovery, UDS – Undisturbed Sample, RQD – Rock Quality Designation

Figure 4 Field log profile of BH-3

Table 1

PROJECT : Proposed Construction of HLL Regional Office Building, Pallikaranai

BH 1	Ground Water Level	4.45 m from EGL

LABORATORY TEST RESULTS

SPT 'N' Value	Depth (m)	Soil Description / Classification	Index Properties						Specific Gravity	Grain Size Analysis (%)						~~Triaxial Shear / UCC Test / Direct Shear Test Results~~ / Correlated Values*		Consolidation Test Results	
			Natural Moisture Content (NMC), %	Bulk Density (kN/m³)	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %	Free Swell Index%		Gravel	Coarse Sand	Medium Sand	Fine Sand	Fines		Cohesion (c) kN/m²	Angle of Friction (Φ)	Co-efficient of Consolidation (C_v) x 10^{-3} cm²/sec	Compressibility Index, (C_c)
														Silt	Clay				
-	0.50	Grayish Black Silty Clay (CH) - Filled up Soil	17	-	69	34	35	65	2.48	0	2	4	4	90		-	-	-	-
4	2.00	Grayish Black Silty Clay (CH)	44	14	63	32	31	60	2.50	0	2	5	7	86		25	0^0	-	-
3	3.00	Grayish Black Silty Clay (CH)	48	14	63	32	31	60	2.51	0	4	7	14	75		18	0^0	-	-
1	4.00	Grayish Black Clayey Sand (SC)	32	11	32	18	14	25	2.53	0	17	21	32	30		0	19^0	-	-
1	5.00	Grayish Black Clayey / Silty Sand	30	11	30	16	14	20	2.58	0	17	25	31	27		0	19^0	-	-
1	6.00	Grayish Black Clayey / Silty Sand	30	11	30	16	14	20	2.58	0	17	24	32	27		0	19^0	-	-
1	7.50	Grayish Black Silty Sand	28	11	Non Plastic				2.61	0	24	27	29	20		0	19^0	-	-
6	9.00	Grayish Black Silty Sand	18	16	Non Plastic				2.61	0	37	24	19	20		0	26^0	-	-
> 100	10.8	Grayish Brown Silty Sand - Weathered Rock sediments	10	22	Non Plastic				2.66	25	24	10	23	18		0	45^0	-	-

Note: * Shear strength parameters were derived as per Terzaghi and Peck Correlation (1974).

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Table 2

PROJECT : Proposed Construction of HLL Regional Office Building, Pallikaranai

BH 2 | **Ground Water Level** | **4.65 m from EGL**

LABORATORY TEST RESULTS

SPT 'N' Value	Depth (m)	Soil Description / Classification	Index Properties						Specific Gravity	Grain Size Analysis (%)						~~Triaxial Shear / UCC Test / Direct Shear Test Results~~ / Correlated Values*		Consolidation Test Results	
			Natural Moisture Content (NMC), %	Bulk Density (kN/m³)	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %	Free Swell Index%		Gravel	Coarse Sand	Medium Sand	Fine Sand	Fines		Cohesion (c) kN/m²	Angle of Friction (Φ)	Co-efficient of Consolidation (C_v) x 10^{-3} cm²/sec	Compressibility Index, (C_c)
														Silt	Clay				
-	0.50	Grayish Black Silty Clay (CH) - Filled up Soil	16	-	66	32	34	60	2.46	0	3	5	4	88		-	-	-	-
1	3.00	Grayish Black Silty Clay (CH)	38	11	65	35	30	60	2.46	0	6	9	21	64		6	0^0	-	-
5	4.00	Grayish Black Clayey Sand (SC)	36	15	34	18	16	30	2.52	0	15	20	33	32		0	25^0	-	-
0	5.00	Grayish Black Clayey / Silty Sand	28	11	31	16	15	25	2.58	0	21	30	24	25		0	15^0	-	-
2	6.00	Grayish Black Clayey / Silty Sand	27	13	27	15	12	20	2.58	0	24	30	24	22		0	21^0	-	-
6	7.50	Grayish Black Clayey / Silty Sand	27	16	27	15	12	20	2.60	0	33	27	19	21		0	26^0	-	-
4	9.00	Grayish Black Silty Sand	20	14	Non Plastic				2.60	0	46	23	19	12		0	24^0	-	-
> 100	10.5	Grayish Brown Silty Sand - Weathered Rock sediments	12	22	Non Plastic				2.66	18	21	18	26	17		0	45^0	-	-

Note: * Shear strength parameters were derived as per Terzaghi and Peck Correlation (1974).

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Table 3

PROJECT : Proposed Construction of HLL Regional Office Building, Pallikaranai

BH 3

Ground Water Level: 4.25 m from EGL

LABORATORY TEST RESULTS

SPT 'N' Value	Depth (m)	Soil Description / Classification	Index Properties						Specific Gravity	Grain Size Analysis (%)						~~Triaxial Shear / UCC Test / Direct Shear Test Results~~ / Correlated Values*		Consolidation Test Results	
			Natural Moisture Content (NMC), %	Bulk Density (kN/m³)	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %	Free Swell Index %		Gravel	Coarse Sand	Medium Sand	Fine Sand	Fines		Cohesion (c) kN/m²	Angle of Friction (Φ)	Co-efficient of Consolidation (C_v) x 10^{-3} cm²/sec	Compressibility Index, (C_c)
														Silt	Clay				
-	0.50	Grayish Black Silty Clay (CH) - Filled up Soil	18	-	63	31	32	60	2.44	0	2	4	4	90		-	-	-	-
4	3.00	Grayish Black Silty Clay (CH)	42	14	62	34	28	60	2.44	0	5	5	10	80		25	0^0	-	-
3	4.00	Grayish Black Silty Clay (CH)	41	14	60	34	26	60	2.50	0	5	7	18	70		18	0^0	-	-
2	5.00	Grayish Black Clayey Sand (SC)	32	13	34	18	16	30	2.54	0	13	18	43	26		0	21^0	-	-
1	6.00	Grayish Black Clayey Sand (SC)	31	11	34	18	16	30	2.54	0	14	16	43	27		0	19^0	-	-
16	7.50	Grayish Black Silty Clay (CH)	39	19	72	35	37	70	2.58	0	3	3	6	88		100	0^0	-	-
9	9.00	Grayish Black Silty Clay (CH)	38	17	60	32	28	60	2.58	0	10	15	20	55		56	0^0	-	-
> 100	10.5	Brown Silty Sand - Weathered Rock sediments	15	22	Non Plastic				2.67	10	17	16	28	29		0	45^0	-	-
> 100	12.0	Brown Weathered Rock sediments	-	22	Washed Sample											0	45^0	-	-

Note: * Shear strength parameters were derived as per Terzaghi and Peck Correlation (1974).

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Table 4 a: *Chemical analysis of soil sample at 0.3m depth in the proposed construction site*

Sl.No.	Parameters	Protocol	Unit	Values
1	Total Organic Carbon	IS 10158 : 1982 (Reaff:2003)	%	0.08
2	Nitrate as NO3	IS 10158 : 1982 (Reaff:2003)	mg / kg	207
3	Chloride as Cl	APHA 22nd EDI : 2012	mg / kg	650
4	Sulphate as SO4	IS 2720 (Part-27) : 1977 (Reaff:2006)	mg / kg	98.0
5	Soluble Salt Content	IS 2720 (Part-21) : 1977 (Reaff:2006)	mg / kg	400
6	Carbonates as CO3	APHA 22nd EDI : 2012	mg / kg	Nil
7	pH	IS 2720 (Part-26) : 1987 (Reaff:2007)	-	8.06

Table 4 b: Chemical analysis of soil sample at 2m depth in the proposed construction site

Sl.No.	Parameters	Protocol	Unit	Values
1	Total Organic Carbon	IS 10158 : 1982 (Reaff:2003)	%	0.09
2	Nitrate as NO3	IS 10158 : 1982 (Reaff:2003)	mg / kg	209
3	Chloride as Cl	APHA 22nd EDI : 2012	mg / kg	655
4	Sulphate as SO4	IS 2720 (Part-27) : 1977 (Reaff:2005)	mg / kg	98.5
5	Soluble Salt Content	IS 2720 (Part-21) : 1977 (Reaff:2005)	mg / kg	408
6	Carbonates as CO3	APHA 22nd EDI : 2012	mg / kg	Nil
7	pH	IS 2720 (Part-26) : 1987 (Reaff:2005)	-	8.07

APHA – American Public Health Association

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Table 5 : Chemical analysis of water sample collected the proposed construction site

Sl.No.	Parameters	Protocol	Unit	Values
1	Carbonates as CO3	APHA 22nd EDI : 2012	mg / L	Nil
2	Nitrate as NO3	APHA 22nd EDI : 2012	mg / L	107
3	Soluble Salt Content	APHA 22nd EDI : 2012	mg / L	3081
4	Calcium as Ca	APHA 22nd EDI : 2012	mg / L	529
5	Magnesium as Mg	APHA 22nd EDI : 2012	mg / L	83
6	pH @ 25^{o} C	APHA 22nd EDI : 2012	-	7.40
7	Total Dissolved Solids (Inorganic) @ 180^{o} C	APHA 22nd EDI : 2012	mg / L	3081
8	Chloride as Cl	APHA 22nd EDI : 2012	mg / L	1200
9	Sulphate as SO4	APHA 22nd EDI : 2012	mg / L	485

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Annexure - 1 (Site Photographs)

Plate 1 View of SPT in Progress at the proposed Site

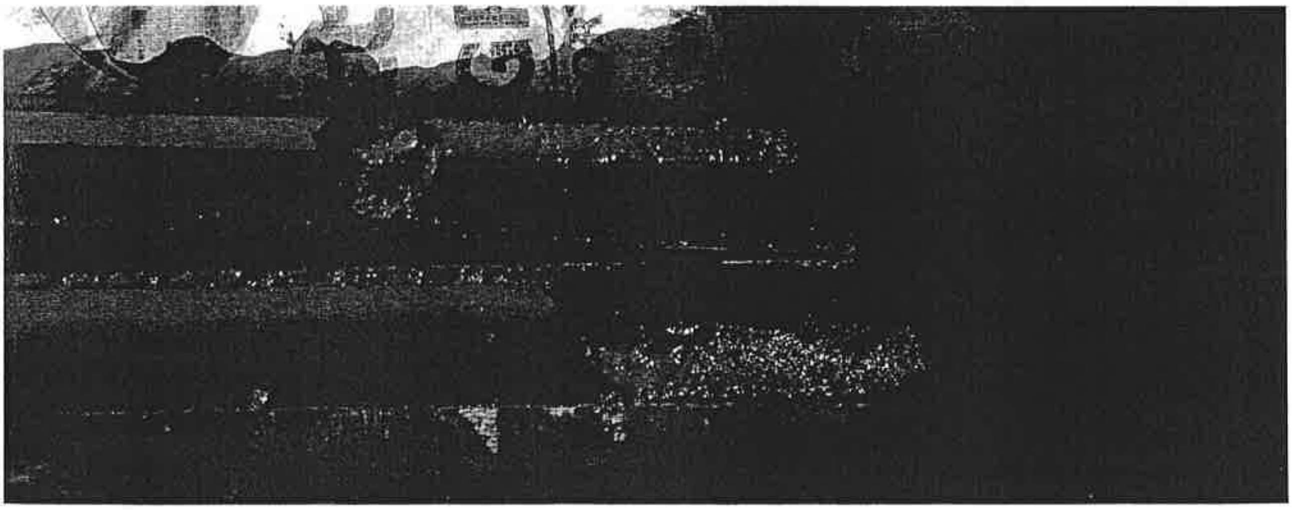
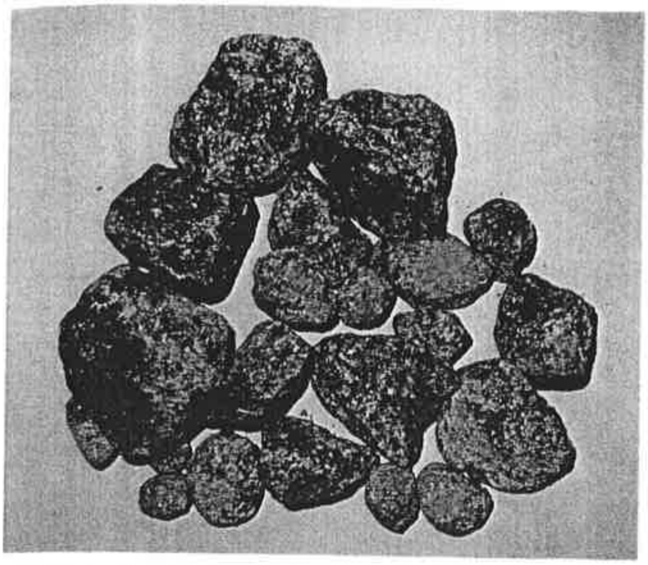
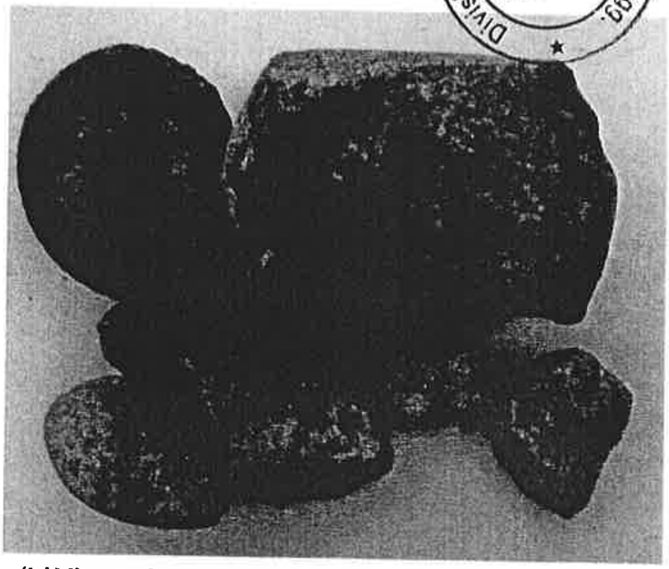


Plate 2 View of Soil Sampling collected through Split Spoon Sampler at the proposed Site

Plate 3 Inspection by the Faculty Member of Department of Civil Engineering, CEG, Anna University, Chennai on 07.04.2015 at proposed Site along with Officials of M/S. HLL Life care Limited



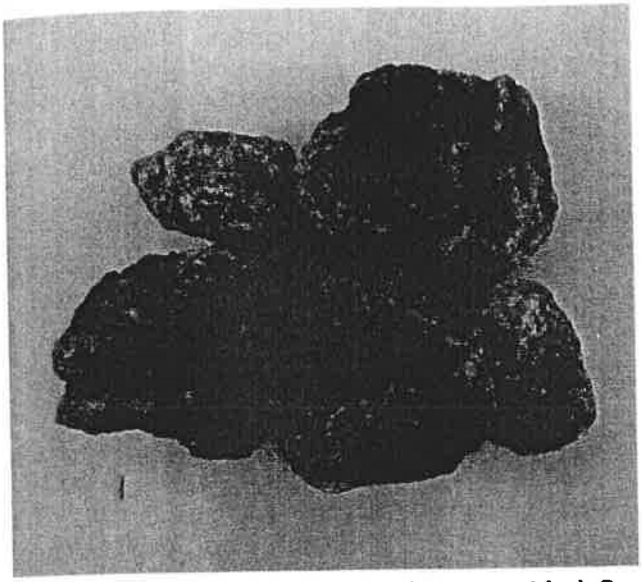
(a)View of Hypersthene Granite (Charnockite) Core Sample found in BH 1 at 10.8 m to 12.3 m Depth



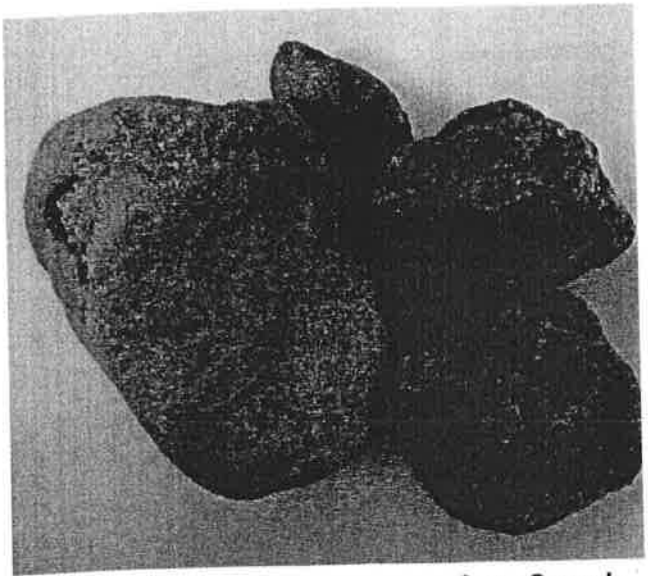
(b)View of Hypersthene Granite (Charnockite) Core Sample found in BH 1 at 12.3 m to 13.8 m Depth

(c)View of Hypersthene Granite (Charnockite) Core Sample found in BH 1 at 13.8 m to 15.3 m Depth

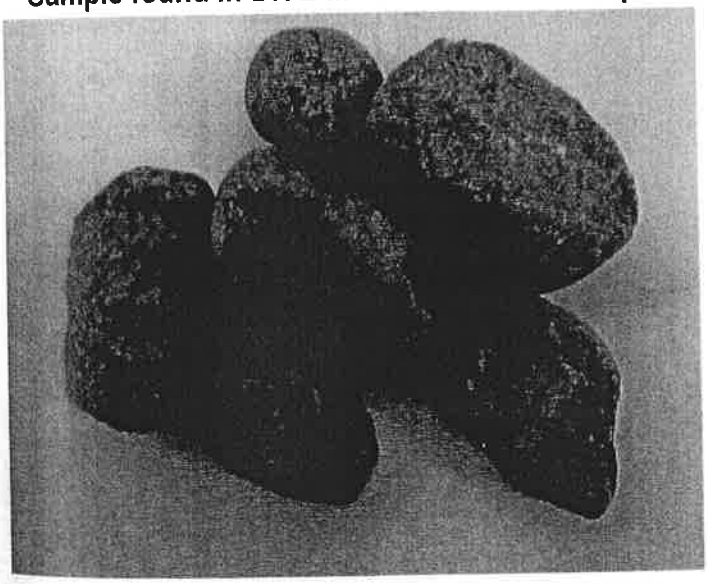
Plate 4 A top view of Hypersthene Granite (Charnockite) Core Sample found in BH 1 at 10.80 m to 15.30 m Depth



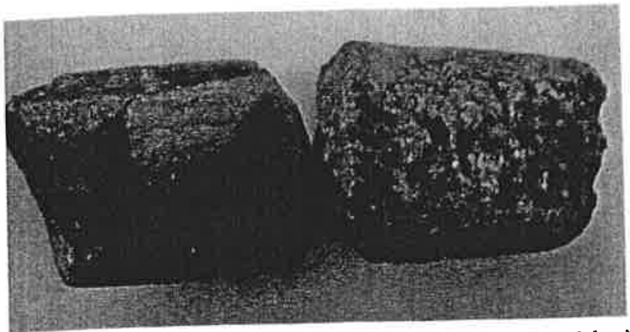
(a)View of Hypersthene Granite (Charnockite) Core Sample found in BH 2 at 10.5 m to 12.0 m Depth



(b)View of Quartzitic Sandstone Core Sample found in BH 2 at 12.0 m to 13.5 m Depth

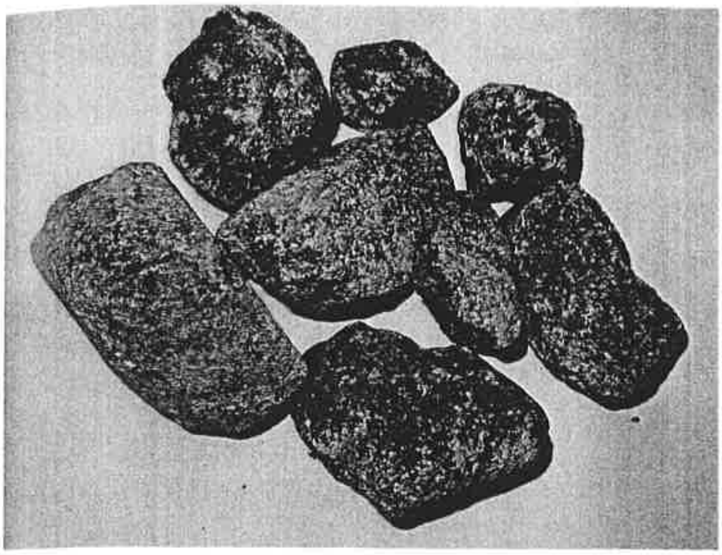


(c)View of Hypersthene Granite (Charnockite) Core Sample found in BH 2 at13.5 m to 15.0 m Depth

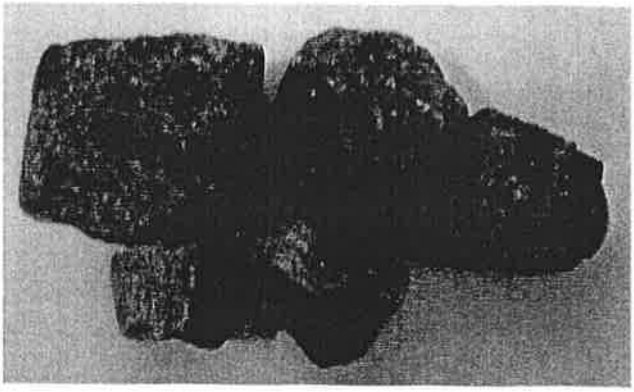


(d)View of Hypersthene Granite (Charnockite) Core Sample found in BH 2 at 15.0 m to 15.8 m Depth

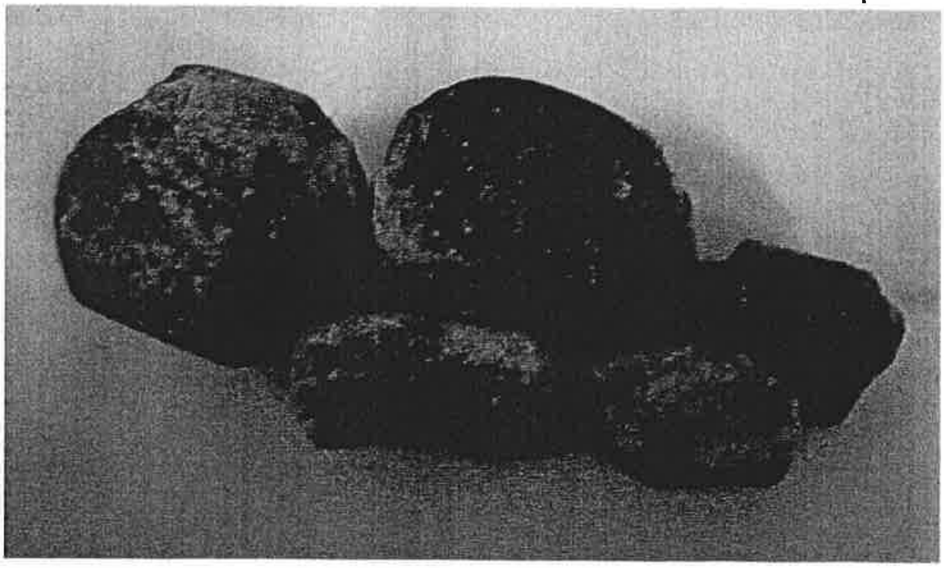
Plate 5 A top view of Hypersthene Granite (Charnockite) Core Sample found in BH 2 at 10.5 to 15.8 m Depth



(a)View of Hypersthene Granite (Charnockite) Core Sample found in BH 3 at 12.0 m to 13.5 m Depth



(b)View of Hypersthene Granite (Charnockite) Core Sample found in BH 3 at 13.5 m to 15.0 m Depth



(c)View of Hypersthene Granite (Charnockite) Core Sample found in BH 3 at 15.0 m to 16.5 m Depth

Plate 6 A top view of Hypersthene Granite (Charnockite) Core Sample found at BH 3 in 12.0 to 16.5 m Depth

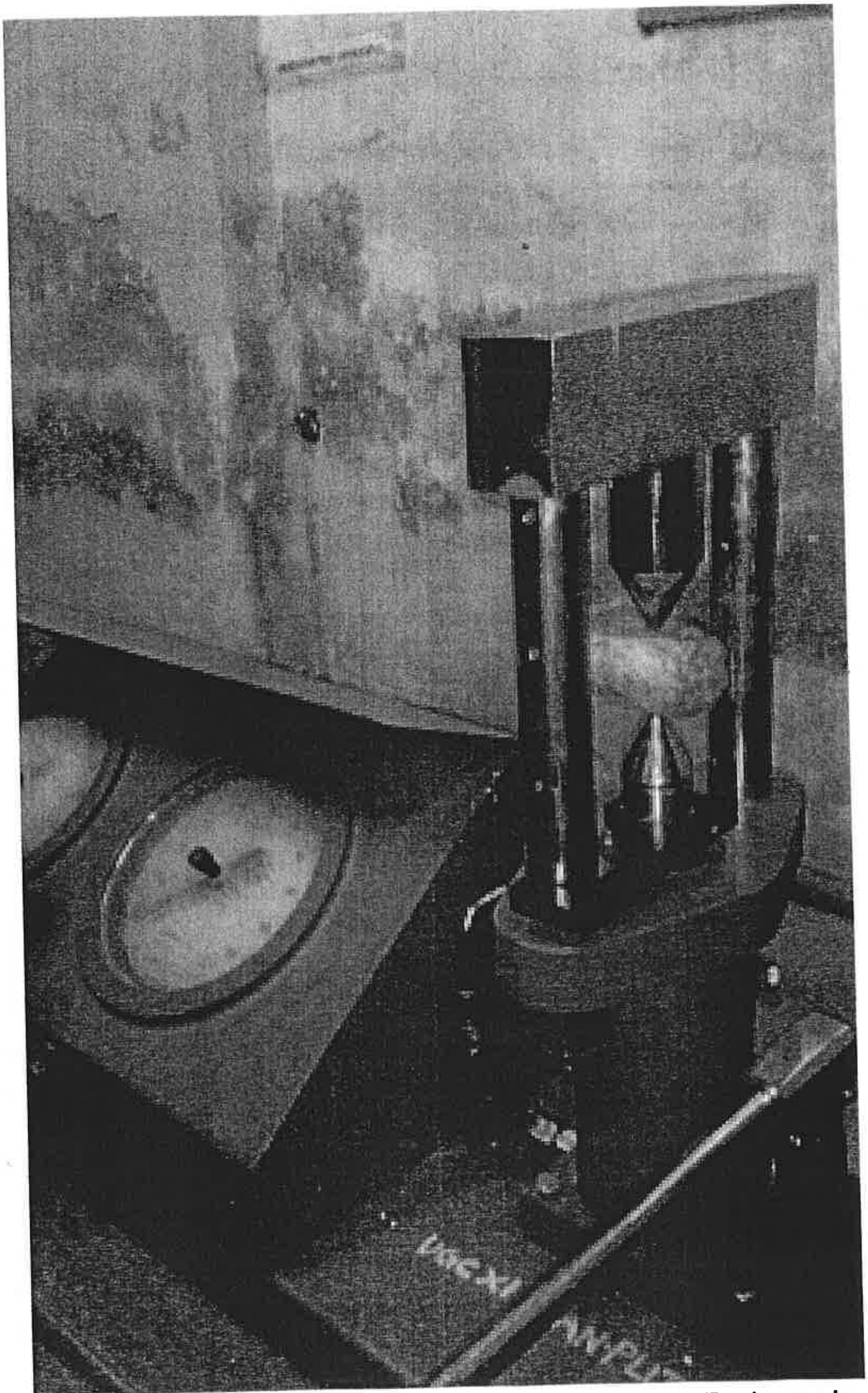


Plate 7 View of conducting Point Load Strength Index Test on Rock samples in Department of Civil Enginering, CEG, AU,Chennai-25

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Annexure - 2 (Calculation)

Pile Carrying Capacity using Static Formula (IS 2911: 2010)

$$Q_u = A_p\,(cN_c + \sigma_{vb}\,N_q + 0.5\gamma DN_\gamma) + \Sigma A_s(\alpha c + K\sigma_v \tan\Phi)$$

Where,

Q_u	-	Ultimate Pile Capacity, kN
A_p	-	Cross Sectional Area of Pile, m^2
c	-	Cohesion value of soil at pile base, kN/m^2
σ_{vb}	-	Effective overburden pressure at pile base , kN/m^2
γ	-	Unit weight of soil, kN/m^3
D	-	Diameter of pile, m
A_s	-	Surface Area of Pile at respective soil layers, m^2
α	-	Adhesion factor
c	-	Cohesion value of soil at respective soil layers, kN/m^2
K	-	Co-efficient of earth pressure
σ_v	-	Effective overburden pressure at respective soil layers, kN/m^2
Φ	-	Friction angle, degrees
N_c, N_q, N_γ	-	Bearing capacity factors based on IS 2911 Part I 2010 and IS 6403 -1981

Pile Carrying Capacity using Mererhof's Formula (1959) – Sand

$$Q_u = 4\,(L/D)\,N\,A_p + (N_{av} / 5)\,A_s$$

Where,

Qu	-	Ultimate Pile Capacity, Tons
L	-	Length of pile, m
D	-	Diameter of pile, m
A_p	-	Cross Sectional Area of Pile, m^2
N	-	SPT 'N' value at pile base
N_{av}	-	Average SPT 'N' value at respective soil layers
A_s	-	Surface Area of Pile at respective soil layers, m^2
(L/D)	-	If it exceeds 10 then it shall be restricted to 10 for such cases

End Bearing Capacity of Pile (BH 2 – Worst Soil condition)

Pile length (L)	=	12 m
c	=	0 kN/m^2 at 21 m depth
Φ	=	45^0
N_c	=	9

N_q	=	330
N_γ	=	271.3
γ	=	12 kN/m³ at 12 m depth (Submerged condition)
σ_{vb}	=	144 kN/m² at 12 m depth
End Bearing Resistance	=	4833.39 t/m² for 500 mm diameter pile
L / D	=	24 for 500 mm diameter pile. Take (L/D) as 10
SPT 'N'	=	> 100 at 12 m depth. Take N = 50
End Bearing Resistance	=	2000 t/m² for 500 mm diameter pile

The computed end bearing resistance vales are shown in table (a). The Indian Standard IS 2911 (2010) specifies that the base resistance should not exceed 1100 t/m² for bored cast-in-situ piles and 1500 t/m² for precast driven piles. ***The factor of safety is 3 for bored cast-in-situ pile*** to arrive safe pile carrying capacity.

Table (a) End Bearing Resistance of Piles

Computed End Bearing Resistance,(t/m²)		Limiting End Bearing Resistance, (t/m²) as per IS 2911	
IS 2911 Part-I 2010 Static Formula	Meyerhof 1959	Bored Piles	Driven Piles
4833.39	2000	1100	1500

Table (b) Ultimate End Bearing Capacity of piles for varying diameter

Diameter of Pile (mm)	End Bearing capacity, (t)	
	IS 2911 Part-I 2010 Static Formula	Meyerhof 1959
450	174.9	174.9
500	215.9	215.9
600	310.9	310.9
750	486.0	486.0

Table (c) Safe End Bearing Capacity of piles for varying diameter

Diameter of Pile (mm)	End Bearing capacity, (t)	
	IS 2911 Part-I 2010 Static Formula	Meyerhof 1959
450	58.3	58.3
500	72.0	72.0
600	103.6	103.6
750	162.0	162.0

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Frictional Capacity of Pile (BH 2 – Worst Soil condition)

In order to determine the frictional capacity of pile, cohesion, friction angle and 'N' values are taken from table II for different soil layers. The adhesion factor and coefficient of earth pressure values are assumed as 1. The computed frictional capacity of each soil layer is shown in table (d).

Table (d) Frictional Capacity of each soil layer for varying diameter of pile

Depth, (m)	Length of each soil layer, (m)	Frictional capacity of pile - IS 2911 Part-I 2010 Static Formula, (t)				Frictional capacity of pile - Meyerhof 1959, (t)			
		450 mm	500 mm	600 Mm	750 mm	450 mm	500 mm	600 mm	750 mm
0 – 3	3	2.1	2.4	2.8	3.5	2.5	2.8	3.4	4.2
3 – 4	1	1.4	1.6	1.9	2.4	2.5	2.8	3.3	4.2
4 – 5	1	0.0	0.0	0.0	0.0	1.5	1.6	2.0	2.5
5 – 6	1	0.6	0.6	0.8	0.9	2.3	2.5	3.0	3.8
6 – 7.5	1.5	2.5	2.8	3.4	4.2	5.3	5.9	7.0	8.8
7.5 – 9	1.5	12.7	14.1	17.0	21.2	5.4	6.0	7.2	9.0
9 – 10.5	1.5	12.7	14.1	17.0	21.2	12.7	14.1	17.0	21.2
10.5 – 12.0	1.5	12.7	14.1	17.0	21.2	12.7	14.1	17.0	21.2

The top 5 m uncompacted silty clay / clayey sand layer may induce negative skin friction to the pile during installation. Hence, the safe frictional capacity of pile is arrived by summing the each soil layer's frictional capacity beyond 5 m depth and divided by factor of safety (F.S = 3) and reducing the negative frictional capacity of top 5 m soil layer ((Frictional Capacity of 6 m to 12 m layer / 3) - Frictional Capacity of top 5 m depth)). The computed safe frictional load capacity of pile is shown in table (e)

Table (e) Safe Frictional capacity of pile for varying diameter

Diameter of Pile (mm)	Frictional capacity, (t)	
	IS 2911 Part-I 2010 Static Formula	Meyerhof 1959
450	6.3	10.2
500	7.0	11.4
600	8.4	13.6
750	10.4	17.0

The total load carrying capacity of pile is arrived by adding the end bearing capacity (table (c)) and frictional capacity (table (e)). The safe load carrying capacity of pile for varying diameter is shown in table (f) for 12 m length pile.

Table (f) Safe Load carrying capacity of Pile for varying diameter

Diameter of Pile (mm)	Pile Capacity, (t)	
	IS 2911 Part-I 2010 Static Formula	Meyerhof 1959
450	65	69
500	79	83
600	112	117
750	172	179

Pile Carrying Capacity of Socketed Piles by Cole and Stroud Approach(1977)

Total bearing resistance = (end bearing resistance)+(socket bond strength between rock and pile)

$$Q_{ap} = \left[c_u N_c \frac{\pi D^2}{4} \right] \frac{1}{FS} + \frac{\alpha \tau_a \pi D L}{FS}$$

Where

c_u = shear strength of rock below base of pile.
N_c = bearing capacity factor
D = diameter of pile
α = reduction factor
τ_a = average shear strength of socketed length
L = length of socketed length
FS = factor of safety (recommended = 3)
$\alpha\tau_a$ = adhesion for which the lesser value of 0.05 times cylinder strength of concrete and 0.05 times unconfined compression strength of rock has been recommended.

Socketing Length (L) = 1 D ('D' is the Diameter of Pile)

c_u = 2.05 Mpa (2050 kN/m²)

The computed Safe End Bearing Capacity and Safe Bond Strength values are shown in table (g) and (h). The total End Bearing Capacity of Piles for varying diameter is shown in table (i) as per Cole and Stroud approach. The safe pile carrying capacity is arrived by adding safe frictional capacity of piles listed in table (e) as per IS 2911:2010 Equation and total safe end bearing capacity of piles listed in table (i) as per Cole and Stroud approach. The table (j) shows the Safe Pile Carrying Capacity for varying diameter of piles as per Cole and Stroud method.

Table (g) Safe End Bearing Capacity of piles for varying diameter (Cole and Stroud Approach)

Diameter of Pile (mm)	End Bearing resistance, (t) Cole and Stroud Approach (1977)
450	97.76
500	120.69
600	173.79
750	271.56

Table (h) Safe Socket Bond Strength of piles for varying diameter (Cole and Stroud Approach)

Diameter of Pile (mm)	Socket bond strength, (t) Cole and Stroud Approach(1977)
450	13.03
500	16.09
600	23.17
750	36.21

Table (i) Total Safe End Bearing Capacity of Piles for varying diameter (Cole and Stroud Approach)

Diameter of Pile (mm)	Safe End Bearing Capacity, (t) Cole and Stroud Approach
450	110.8
500	136.8
600	197.0
750	307.8

Table (j) Safe Pile Carrying Capacity of Piles for varying diameter (Cole and Stroud Approach)

Diameter of Pile (mm)	Safe Frictional capacity, (t) Cole and Stroud Approach(1977)
450	117
500	144
600	205
750	318