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GEOTECHNICAL INVESTIGATIONS AND RECOMMENDATIONS FOR FOUNDATION DESIGN OF HUDCO'S LAND, SECTOR 4, VAISHALI, GHAZIABAD

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by

N. K. SAMADHIYA[1], PRITI MAHESHWARI[1], ADITYA SINGH[2]

ABSTRACT

Central Government Employees Welfare Housing Organisation (CGEWHO) is working as a Project Management Consultant for execution of a group housing project on HUDCO's land at Plot No. 28, Sector 4, Vaishali, Ghaziabad. This report presents details of various geotechnical investigations conducted at Ghaziabad site. The investigations included – i) 10 borings, each advanced to a depth of 25 m, along with standard penetration tests, ii) 9 dynamic cone penetration tests up to 15.0 m depth or refusal, iii) 3 plate load tests in test pits at a depth of 2.0 m. Both representative and undisturbed soil samples collected during borings and from test pits were transported to Geotechnical Engineering Laboratories of IIT Roorkee. This report therefore deals with systematic analysis of field and laboratory tests data, its interpretation and suggestions for appropriate foundation type along with estimation of allowable bearing pressure for design of the foundations of proposed structures. The report concludes with all recommendations which can be used for the structural design and construction of foundations.

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1.0 INTRODUCTION

Central Government Employees Welfare Housing Organisation (CGEWHO) is working as a Project Management Consultant for execution of a group housing project on HUDCO's land at Plot No. 28, Sector 4, Vaishali, Ghaziabad. The land is 155 m x 64 m in a plan area where 200 dwelling units have been planned to be constructed. Er. Gagan Gupta, Director (Technical), Central Government Employees Welfare Housing Organization New Delhi vide letter No. T 801/4/1 dated January 17, 2022 requested Dr. N.K. Samadhiya, Professor, Department of Civil Engineering, IIT Roorkee to carry out geotechnical investigations at the proposed site for ascertaining the type of foundations. A geotechnical investigation program was proposed by Dr. N.K. Samadhiya, Professor, Department of Civil Engineering, IIT Roorkee vide letter No. CED/GTE/NKS/3011 dated November 30, 2022. The acceptance of the proposal was communicated by the Executive Director (Projects), Construction & Consultancy, HUDCO, New Delhi vide work order No.: HUDCO/C&PM/03/Vaishali/2023 dated 20/02/2023. The field investigations were carried out from March 13 to March 23, 2023. This report presents the details of field and laboratory investigations, the interpretation and recommendation on allowable bearing pressure/load for the design of foundation.

The opinion in this report is the personal and professional opinion of the project investigators involved in this project and should not be considered as an opinion of IIT Roorkee.

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2.0 SCOPE OF WORK

At the proposed housing complex, the site is more or less a leveled one. The type of construction involves a multi storeyed building complex with basement. This report presents – i) all the field test data obtained at the site during field exploration and testing, ii) laboratory test data obtained from various laboratory tests conducted on all the soil samples collected from site, iii) analysis and interpretation the entire test data, and iv) recommendations on the suitability of the types of foundations to be provided for various structures and their respective values of allowable soil pressure / load carrying capacity for design.

3.0 FIELD INVESTIGATIONS

In view of what has been stated above, it was decided to conduct following field tests:

i) **Borings** to be advanced to a depth of 25.0 m or refusal, whichever is earlier – at 10 locations,

ii) **Standard Penetration Tests** (SPT), to be conducted in each borehole at an interval of 1.5 m up to a depth of 15.0 m and at an interval of 3.0 m beyond.

iii) **Dynamic cone penetration tests** (DCPT) – to be extended to a depth of 15.0 m or refusal, whichever is earlier. These tests, which give an idea of the variation of continuous penetration resistance with depth, also indicate the presence of loose pockets of soil and presence of fill material, if any - 09 locations.

iv) **Plate load tests** (PLT) – to be conducted in a test pit at a depth of 2.0 m on a 300 mm x 300 mm size square test plate– at 03 locations.

v) Collection of undisturbed and representative soil samples from different depths during borings and plate load test pits, for laboratory tests, and

vi) Observation of ground water table.

Figure 1 shows the overall layout plan of the faculty housing complex along with various test locations.

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4.0 LABORATORY INVESTIGATIONS

The laboratory testing program included:

i) **Soil Classification Tests** – to be conducted on all soil samples including: a) mechanical sieve analysis for studying the grain size distribution of soil and b) Atterberg Limits tests for studying the plasticity characteristics of soil,

ii) **Unconfined Compression Tests** - on all undisturbed soil samples collected during borings from various depths and,

iii) **Consolidation Tests** - on all undisturbed soil samples collected during borings from various tests.

5.0 FIELD TEST DATA

5.1 BORINGS

Locations of all the exploratory bore holes have been shown in Fig. 1. Standard penetration tests were conducted in each borehole. Representative samples collected during these tests were used in the laboratory for various classification and identification tests on soils. Undisturbed soil samples were also collected during borings, wherever possible, in clay strata for performing unconfined compression and consolidation tests in the laboratory.

5.2 PENETRATION RESISTANCE TESTS

5.2.1 Standard Penetration Tests

Standard penetration tests were conducted as per IS: 2131-1997. These tests were conducted in each borehole at every 1.5 m depth interval up to a depth of 15.0 m and at an interval of 3.0 m beyond i.e. up to the termination depth of borings. Variation of the observed penetration resistance with depth has therefore been presented in Appendix-A (Figs. A-1 – A-10) for the ten borehole locations. The observed values have been corrected for overburden and dilatancy, wherever required. Figures A-1 to A-10 also show the variation of the corrected values of standard penetration resistance with depth which are subsequently used in design.

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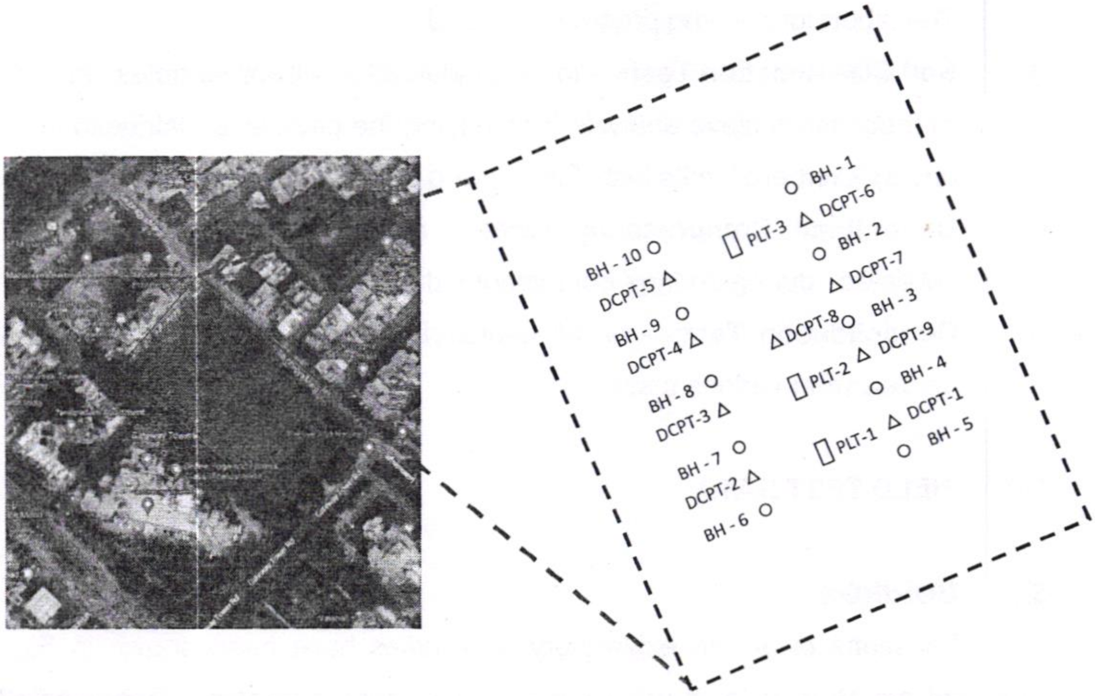

Fig. 1. Layout map

5.2.2 Dynamic Cone Penetration Tests

Dynamic cone penetration tests were conducted at 9 locations as shown in Fig. 1 according to the guidelines of IS: 4968-1997. The data obtained from any dynamic cone penetration test, which gives a continuous penetration resistance with depth, helps in identifying the presence of weak / loose strata or soft pockets, if any, in the soil mass beneath the ground surface. Figures B-1 to B-5 in Appendix-B show the plots of variation of dynamic cone penetration resistance observed with depth. The penetration resistance, N_{cd} has been plotted for every successive 30 cm penetration.

5.3 PLATE LOAD TESTS

Plate Load Tests (PLT) were conducted according to IS: 1888-1997 with monotonic loading at three locations as shown in Fig. 1. These were conducted on a 300 mm x 300 mm size rigid square steel test plate. All the tests were conducted at a depth of 2.0 m below the ground surface up to failure. Figures C-1 to C-3 in

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Appendix–C show respectively the plots of load intensity versus plate settlement obtained at two test locations. These plots show a trend of general shear failure of the test plate.

5.4 COLLECTION OF UNDISTURBED SAMPLES

Representative soil samples were collected from different depths during borings whenever a standard penetration test was conducted. Undisturbed soil samples were also collected in plastic soil strata, i.e. wherever clay strata were met with during boring for the conduct of consolidation and unconfined compression tests in the laboratory.

6.0 LABORATORY TEST DATA

6.1 CLASSIFICATION TESTS

These tests were conducted according to IS: 2720 (Part IV and V). All the basic classification tests including mechanical sieve analysis and the Atterberg's limits (liquid limit and plastic limit) tests were conducted on all the SPT soil samples collected from various depths from boreholes. The soils were classified according to IS: 1498-2021 and the bore logs for ten boreholes have been presented in Tables D-1 – D-10 of Appendix–D.

6.2 UNCONFINED COMPRESSION AND CONSOLIDATION TEST DATA

Undisturbed clay samples were collected in sampling tubes from each bore hole. Unconfined compression strength (UCS) tests were conducted on all such samples. Consolidation tests were also conducted on all these samples. Each consolidation test takes minimum eight days for its completion.

7.0 INTERPRETATION OF TEST DATA

The field and laboratory test data obtained for the site have been interpreted and based on this data; further computations have been made for deciding the suitability of the type of foundation and the allowable soil pressure / load carrying capacity.

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7.1 SOIL CLASSIFICATION

The bore logs obtained at locations BH-01 – BH-10 have been presented in Appendix – D (Tables D-1 – D-10). A close study of these borelogs (Appendix - D) suggests that the entire soil mass up to the depth of exploration (25-30 m) can be sub-divided into four layers as follows:

i) Soil strata upto a depth of about 1.0 m below the ground surface is non-plastic silt (ML-NP) (Layer-1),

ii) Soil strata from 1.0 m to 6.5 m below the ground surface consists of poorly graded silty sand (SP-SM) (Layer-2: cohesionless layer),

iii) Soil strata from 6.5 m to 13.0 m below the ground surface is comprised of clay of low compressibility (CL) or silt of low compressibility (ML), and

iv) A soil layer of poorly graded silty sand (SP-SM) / non-plastic silt beyond the depth of 13.0 m and up to the depth of exploration (25 m) (Layer-4: cohesionless layer).

Two boreholes (BH-01 and BH-06) were extended up to 30 m to ensure the extent of bottom cohesionless soil layer and it was obtained that CL / ML(NP) layer was found at 30 m depth below the ground surface. Looking at the soil strata at the site and presence of ML (NP) at 30 m below the ground surface, the soil layer beyond 28 m below the ground surface is assumed to be a cohesionless soil layer.

On basis of this, representative soil profile has been presented in Fig. 2. The water table was not encountered below the ground surface at various test locations during March 2023.

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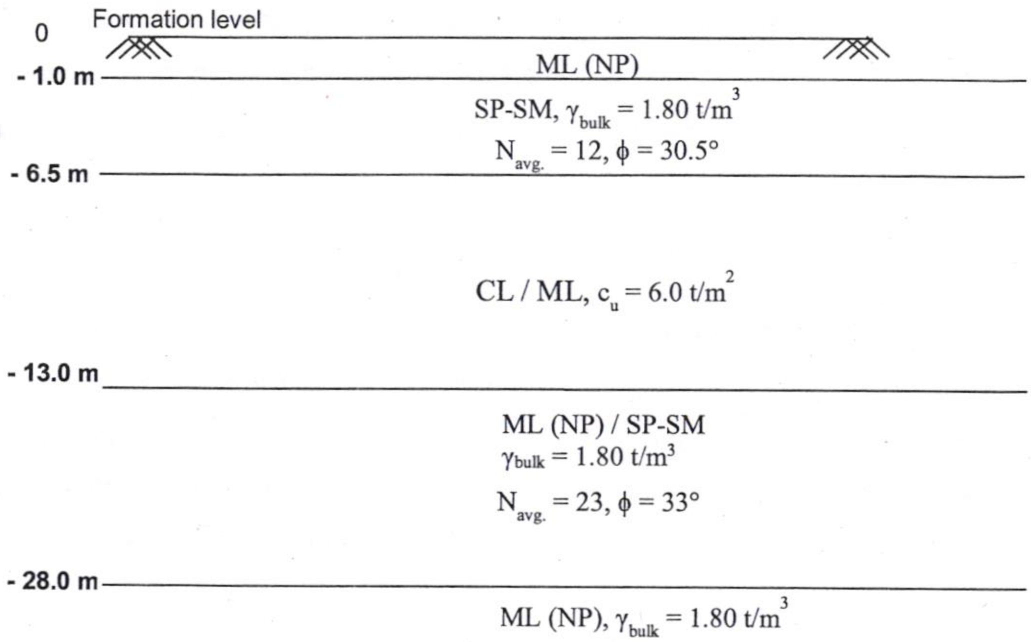

Fig. 2. Representative soil profile

7.2 PENETRATION RESISTANCE

Variation of the observed standard penetration resistance with depth has been presented in Figs. A-1 to A-10 for the various boreholes BH-01 – BH-10. The variation of the corrected values of standard penetration resistance which are used in design calculations is also shown in respective plots. Based on these plots, representative values of corrected penetration resistance are presented for different soil layers in Fig. 2.

Figures B-1 to B-5 show the variation of dynamic cone penetration resistance with depth at different locations in the proposed residential complex. These plots show in general, loose strata up to about 1-2 m and subsequently increasing resistance with depth and occurrence of no soft / loose pocket at any depth.

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7.3 PLATE LOAD TEST DATA

The load intensity versus settlement curves from plate load tests data conducted at different locations, have been presented in Appendix–C (Figs. C-1–C-3). At all the three locations, values of ultimate bearing capacity of the test plate have been obtained by double tangent method, which are presented in respective plots (28.5 t/m^2 for PLT1, 23.8 t/m^2 for PLT2 and 23.9 t/m^2 for PLT3).

7.4 UNCONFINED COMPRESSION AND CONSOLIDATION TEST DATA

Values of unconfined compressive strength, q_c obtained from the tests have been tabulated in Table - 1 and the effective pressure versus void ratio plots are presented in Appendix – E (Figs. E-1 – E-9). Table 1 shows that average unconfined compressive strength of plastic soil has been found to about 8-9 t/m^2.

Table 1 Summary of data of unconfined compression tests

Borehole location	Depth (m)	Bulk Unit weight (t/m^3)	Dry unit weight (t/m^3)	UCS (t/m^2)	Undrained Cohesion, c_u (t/m^2)
BH – 1	9.0	2.13	1.81	26.8	13.4
BH – 1	9.0	2.09	1.81	22.8	11.4
BH – 1	9.0	2.11	1.83	25.5	12.7
BH – 1	12.0	2.07	1.67	19.8	9.9
BH – 1	12.0	2.07	1.72	21.1	10.6
BH – 1	12.0	2.02	1.65	30.9	15.4
BH – 2	10.0	2.04	1.79	13.5	6.7
BH – 2	10.0	2.04	1.79	12.7	6.4
BH – 3	10.0	2.11	1.79	12.8	6.4
BH – 3	10.0	2.13	1.86	12.9	6.4
BH – 3	10.0	2.16	1.86	10.4	5.2
BH – 6	7.0	2.11	1.83	14.0	7.0
BH – 6	7.0	2.09	1.81	14.5	7.3
BH – 6	7.0	2.07	1.81	12.0	6.0
BH – 6	10.0	2.11	1.86	14.2	7.1
BH – 6	10.0	2.13	1.86	18.4	9.2
BH – 7	7.0	2.20	1.90	21.3	10.7
BH – 7	7.0	2.16	1.88	16.9	8.5
BH – 9	11.0	2.11	1.81	19.9	10.0
BH – 9	11.0	2.09	1.81	15.3	7.7

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Borehole location	Depth (m)	Bulk Unit weight (t/m^3)	Dry unit weight (t/m^3)	UCS (t/m^2)	Undrained Cohesion, c_u (t/m^2)
BH – 10	10.0	2.11	1.88	19.3	9.6
BH – 10	10.0	2.09	1.81	14.3	7.1

8.0 DESIGN CRITERIA

Foundations, in general, are designed for safety against two criteria:

i) Foundations must be safe against shear failure and

ii) Foundations should not settle excessively.

Attempt has, therefore been made to design the foundations of various proposed structures considering both these criteria. IS 1904: 1995 gives limits of total settlement, differential settlements and angular distortions for shallow foundations of the framed structures. These limiting values have been specified for - a) sand and hard clay and b) plastic clays. In view of the fact that there is no water table present at the site, possibility of liquefaction does not exist. Hence, a raft foundation can be considered as one of the options for suitable foundation. Further, this is also a fact that various buildings in the complex will be quite tall, it is anticipated that the load intensity transferred to the soil will be quite high and hence a combined pile-raft system can also be explored as a foundation to support these heavily loaded tall structures. Hence subsequent Articles of this report show calculations both for pile foundations and raft foundation.

9.0 ALLOWABLE LOADS ON PILES

A close study of various soil bore logs (Appendix - D) suggests the soil strata from 1 m to 6.5 m below the ground surface consists of cohesionless soil followed by a maximum of 6.5 m thick cohesive soil layer. Below this cohesive layer, i.e., beyond 13 m below the ground surface, again a cohesionless soil layer has been encountered up to the depth of exploration (25 m). Further, two boreholes (BH-01 and BH-06) were extended up to 30 m to ensure the extent of bottom cohesionless soil layer and it was obtained that CL / ML(NP) layer was found at 30 m depth

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below the ground surface. Looking at the soil strata at the site and presence of ML (NP) at 30 m below the ground surface, the soil layer beyond 28 m below the ground surface is assumed to be a cohesionless soil layer.

The geotechnical state of art suggests that cohesive soil stratum (between 6.5 m to 13 m below the ground surface) should be completely penetrated and the load of the structure should be transferred to a stronger sandy soil layer below. In view of one basement, the cut-off level of the base of the pile foundation has been decided as 5.0 m below the existing ground level. The minimum suitable length of the pile foundation, therefore, works out to be 10 m with a penetration length of 2 m into the sandy soil layer below.

9.1 AXIAL LOAD CARRYING CAPACITY OF BORED CAST-IN-SITU PILES

Figure 3 shows the position of bored cast-in-situ pile in relation to various soil layers below the ground surface along with representative soil properties.

The point bearing resistance of bored cast-in-situ pile in granular soils is given by:

$$Q_p = A_b\, \sigma_v'\, N_q + 0.5\, A_b\, \gamma\, D\, N_\gamma \quad (1)$$

where, A_b is the area of pile tip, σ_v', the effective overburden pressure at pile tip and N_q, the bearing capacity factor (= 32 in present case; IS: 2911 (Part I / Sec 2) – 2010) and N_γ, the bearing capacity factor (= 37.8 in present case; IS: 6403 – 1997).

The frictional resistance mobilized along the pile shaft in c - ϕ soil strata is given by –

$$Q_s = f.\, A_s = A_s\, [K_0.\, \sigma.\, \tan\delta + \alpha.\, c_u] \quad (2)$$

where, f is the unit skin friction (t/m^2); $K_0 = (1 - \sin\phi)$, the earth pressure coefficient at rest; σ, the average effective overburden pressure (t/m^2) over embedded length of pile; $\delta\ (=2\phi / 3)$, angle of wall friction, c_u, the undrained cohesion (= 6.0 t/m^2 in present case) and A_s is the embedded surface area (m^2) of pile in soil strata.

The ultimate axial load carrying capacity of pile is therefore given by –

$$Q_{ult} = Q_p + Q_s \quad (3)$$

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The values of ultimate and safe axial load carrying capacity of bored cast-in-situ piles were evaluated using equations (1), (2) and (3) for different values of pile diameter, i.e., 450 mm, 600 mm and 800 mm and for the pile length of 10 m. These values are presented in Table 2.

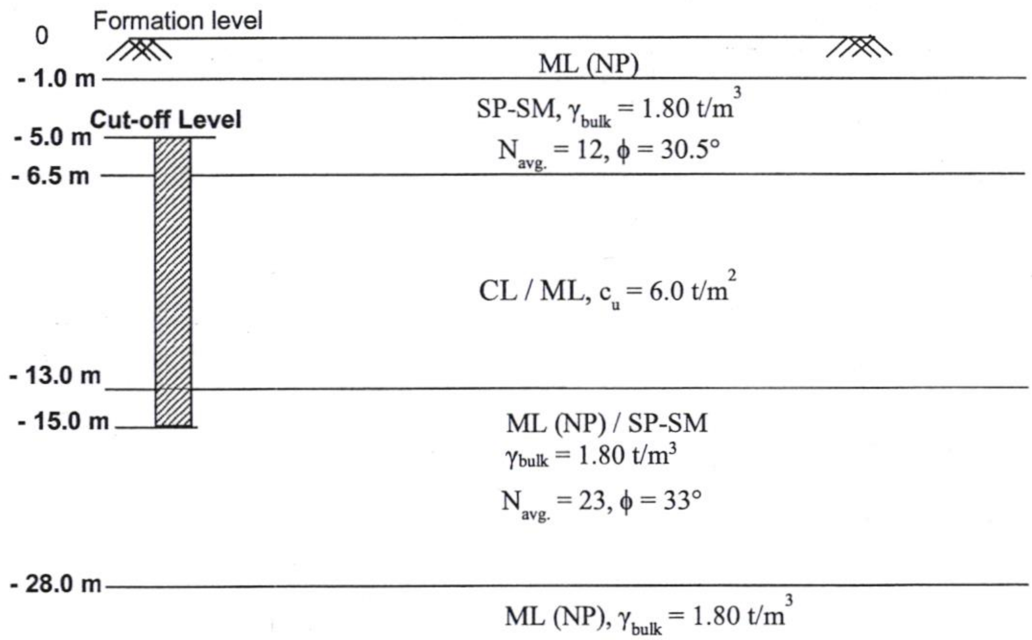

Fig. 3 Representative soil profile for calculating load carrying capacity of a pile

Table 2 Ultimate and safe axial load carrying capacity of bored cast-in-situ piles

Pile Length (m)	Pile Diameter (mm)	Point Bearing Resistance Q_p (t)	Skin Friction Resistance Q_s (t)	Total Ultimate Capacity, Q_{ult} (t)	Safe Pile Capacity, Q_{safe} (t)*
10	450	94.04	50.30	144.34	72.17
	600	168.63	67.07	235.70	117.85
	800	303.21	89.42	392.63	196.32

* Factor of safety = 2 for estimating Q_{safe}.

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9.2 PILE GROUPS

The bored cast-in-situ piles, when provided in the form of pile group as a foundation supporting the various multi-storeyed buildings of the proposed housing complex, may be installed at a centre to centre spacing of 2.5 times the pile diameter. The piles may be provided in a staggered fashion. The load carrying capacity of the entire pile group is expected to be more than the load carrying capacity of the individual pile times the number of piles in the group. A typical configuration of the pile - raft system is shown in Fig. 4. In view of one basement, the cut-off level of base of pile cap has been decided as 5.0 m below the formation level.

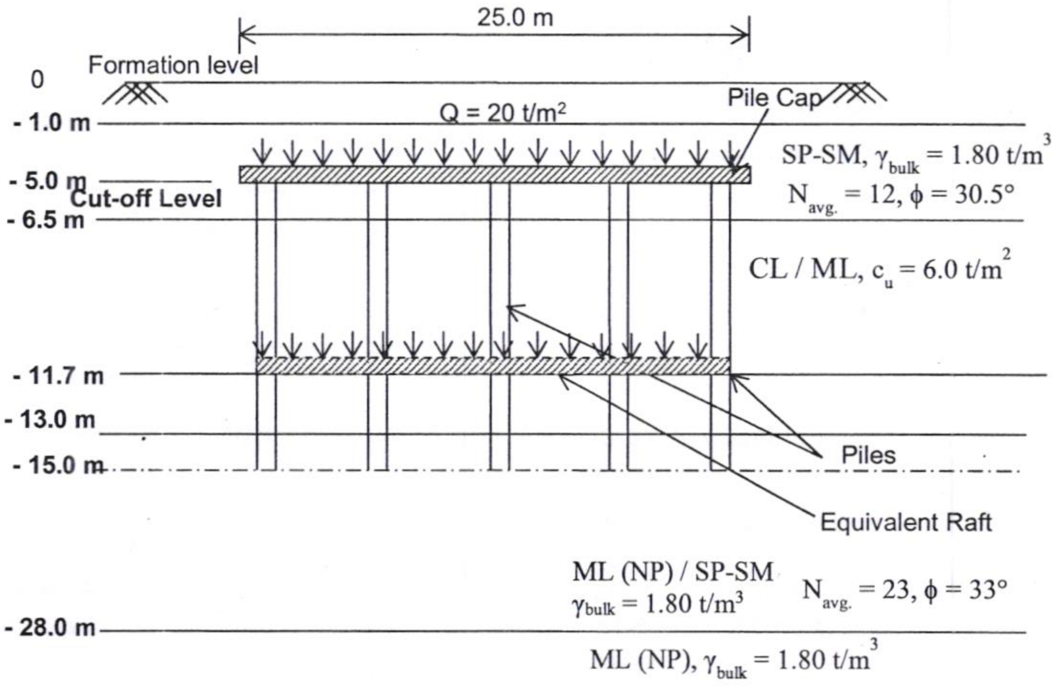

Fig. 4 Typical raft - pile group system

10.0 SETTLEMENT OF PILE GROUPS

The settlement of a pile-raft system at faculty housing complex has been computed using the equivalent raft method. In this method, pile-raft system is replaced by an equivalent raft placed at a depth equal to two thirds of the length of

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the pile. The pressure distribution below the equivalent raft has been computed following 2:1 pressure distribution profile. An allowable settlement of 125 mm (IS: 1904-1995) has been considered as the permissible total settlement for equivalent raft resting on clay.

The total settlement of the equivalent raft will be comprised of the immediate settlement of granular sand layer below equivalent raft and consolidation settlement of the clay layer below.

The settlement of the clay layer, S_c is obtained as, $S_c = \frac{\Delta e}{1+e_o} H$

Δe	=	change in void ratio due to change in stress from p_o to $p_o+\Delta p$ in the compressible clay soil
e_o	=	initial void ratio corresponding to p_o, the effective overburden stress at mid depth of the clay layer
H	=	thickness of the compressible clay layer

The increase in stress, Δp, at the middle of clay layer due to foundation has been taken as 20 t/m^2 and consolidation test data of borehole location BH-06 at 10 m is used. This settlement works out to be 25 mm.

For 10 m long pile, the allowable soil pressure for a raft on sand is given by: $q_a = 0.044\ N\ S\ C_w$ t/m^2. For N = 12, and q_a = 20 t/m^2, the settlement, S works out to be 75.8 mm.

Considering the effect of placement depth and the rigidity of foundation, the appropriate correction factors (0.9 for depth and 0.8 for rigidity effect) have been applied [IS: 8009(Part-I)-197] and therefore, the settlement of equivalent raft works out to be 72.58 mm < 125 mm (permissible).

11.0 LATERAL CAPACITY OF PILES

Lateral capacity of bored cast-in-situ piles has been obtained as per IS 2911 (Part 1/ Section 2): 2010 for different values of pile diameter adopting a lateral deflection of 5 mm as per the procedure given below:

Stiffness factor, T (m) = $\sqrt[5]{\frac{EI}{\eta_h}}$

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Where, E: Elastic modulus of pile material; I: moment of inertia or pile cross-section and η_h: modulus of subgrade reaction (= 2.9 MN/m^3 in present case).

The cantilever length above the ground to the point of load application shall be zero in the present case. Accordingly, the ratio, z_f/T is obtained as 2.2 from IS 2911 (Part 1/ Section 2): 2010. From this ratio, knowing the value of T, depth to point of fixity (z_f) can be obtained.

For fixed headed piles, the deflection is given by: $y = \frac{H\,(e+z_f)^3}{12EI} \times 10^3$

where, H: Lateral load (kN); y: deflection of pile head (mm)

Following this procedure, the depth to point of fixity from pile cutoff level along with the lateral capacity of pile has been computed for a permissible lateral deflection of 5 mm and have been presented in Table 3.

Table 3 Lateral capacity of bored cast-in-situ piles

Pile Length (m)	Pile Diameter (mm)	Depth to point of fixity from ground level (m)	Lateral Capacity, (t)
10	450	3.96	5.31
	600	4.99	8.41
	800	6.28	13.32

12.0 UPLIFT CAPACITY OF PILES

Uplift capacity of the bored cast-in-situ piles has been obtained as per IS 2911 (Part 1/ Section 2): 2010 for different values of pile diameter adopting a factor of safety of 3.0 and has been presented in Table 4.

Table 4 Uplift capacity of bored cast-in-situ piles

Pile Length (m)	Pile Diameter (mm)	Uplift Capacity* (t)
10	450	27.06
	600	36.93
	800	50.74

* Factor of safety = 2 for estimating safe uplift capacity

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13.0 BEARING CAPACITY / ALLOWABLE PRESSURE OF RAFT FOUNDATIONS

The bearing capacity / allowable soil pressure for shallow foundations can be estimated based on the projected average value of standard penetration resistance (**IS: 6403 – 1997**).

13.1 SHEAR FAILURE CRITERION

13.1.1 Based on penetration resistance

The angle of shearing resistance of the soil, ϕ can be obtained based on the SPT, N by using chart given by Peck et al. (1974). Corresponding to an average value of SPT, N equal to 12 (average standard penetration resistance in the soil layer where the raft is placed), the value of ϕ works out to be equal to 30.5°. Accordingly, bearing capacity factors are obtained as:

$N_c = 31.74$, $N_q = 19.89$, and $N_\gamma = 24.96$.

The net ultimate bearing capacity of a raft foundation is given by IS: 6403-1997 –

$$q_{n,ult} = \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma W' \qquad (4)$$

where,

γ = Unit weight of soil = 1.80 t/m^3

D_f = Depth of foundation = 5.0 m (assumed)

B = Width of raft foundation = 25.0 m (assumed)

L = Length of raft foundation = 50 m (assumed)

s_q, s_γ = Shape factors (=1.1 & 0.8 respectively in present case)

d_q, d_γ = Depth factors (due to presence of basement, these are not applied)

W' = Water table correction factor = 0.5

The net ultimate bearing capacity, therefore, works out to be 411.68 t/m^2. Assuming a factor of safety of three, the safe bearing capacity works out to be 137.2 t/m^2.

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13.1.2 Based on plate load test data

The values of ultimate bearing capacity of the test plate (q_{up}) as obtained from Figs. C.1 – C.3 (Appendix – C) vary between 23.8 t/m^2 and 28.5 t/m^2. A close look at the load intensity vs. settlement curves for all three locations of PLT shows that the ultimate settlement of the plate at location PLT – 3 is about 44 mm (maximum of all PLT locations). Hence, further calculations are done using the data of PLT – 3. The net ultimate bearing capacity of test plate is given by

$$q_{net,\,ult} = 0.4\,\gamma\,B_p\,N_\gamma \qquad (5)$$

$$23.9 = 0.4 \times 1.8 \times 0.3 \times N_\gamma$$

Therefore, N_γ = 110.65, correspondingly, the value of friction angle ϕ works out to be 40°. Corresponding value of bearing capacity factor, N_q works out to be equal to 64.2

Using Eq. (4), therefore, the value of net ultimate bearing capacity of raft has been worked out to be 1621.51 t/m^2 and the net safe bearing capacity of raft as 540.5 t/m^2.

13.2 SETTLEMENT CRITERION

13.2.1 Based on penetration resistance

Peck et al. (1974) proposed design charts to estimate net safe bearing pressure on the basis of SPT, N value such that the maximum settlement of a raft resting on sand does not exceed 75 mm. These design charts can be represented by the following relation:

$$q_{ns} = 0.044\,N\,r_w'\,S_a \qquad (6)$$

Where

q_{ns} = Net safe bearing pressure in t/m^2.

N = Average standard penetration resistance below the footing = (12 in present case)

r_w' = Water table correction factor = 0.5 in present case, and

S_a = Allowable settlement in mm.

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= 75 mm (IS: 1904-1995)

The value of net safe bearing pressure for a 25 m size raft, therefore works out to be equal to 19.8 t/m^2.

13.2.2 Based on plate load test data

Terzaghi and Peck (1948) have recommended that the settlement of a footing resting on cohesionless soil can be extrapolated from the settlement experienced by the test plate at the same loading intensity by the following equation:

$$\frac{S_f}{S_p} = \left[\frac{B_f(B_p + 0.30)}{B_p(B_f + 0.3)}\right]^2 \qquad (7)$$

Where

S_f = Settlement of a foundation of width, B_f in m.

S_p = Settlement of test plate of width, B_p in m at the same load intensity as on the foundation.

As explained above, the extrapolated results from PLT – 3 are employed here for calculating the allowable pressure. The permissible settlement of raft on sand is 75 mm as per IS: 1904-1995. As per IS: 8009 (Part –I), 1998, this permissible settlement has been corrected for depth, rigidity and water table. Therefore, the corrected value of settlement of raft works out to be 48.83 mm. Corresponding to this value, the settlement of the test plate from the above equation (7) works out to be equal to 12.5 mm. From load intensity vs. settlement curve for PLT – 3, the allowable pressure of raft has been obtained as 23.0 t/m^2.

Comparing the net safe bearing capacity / allowable pressure of raft from the above two criteria, it can be observed that the settlement criterion governs the design. Therefore, considering a net safe bearing capacity of 19.0 t/m^2, the settlement of the raft has been worked out. The total settlement of the raft will comprise of settlement of top sand layer and the clay layer below.

First the settlement of upper sand layer has been worked out as follows:

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As per IS: 8009 (Part-I)-1998, for average penetration resistance of 12 below the raft having a width of 25 m, the settlement (m) per unit pressure (1 kg/cm^2) comes out to be 3×10^{-2}, i.e., 30 mm. Therefore, for an allowable pressure of 19 t/m^2, the settlement works out to be 57 mm. Further, considering the water table at the base of raft, the water table correction factor, W' is 0.5 and hence the settlement of sand layer works out to be equal to 114 mm. It is expected that about 75% of this settlement will occur by the end of construction period and hence the remaining 28.5 mm will contribute towards the settlement of this layer.

The settlement of the clay layer, S_c is obtained as, $S_c = \frac{\Delta e}{1+e_o} H$

Δe	=	change in void ratio due to change in stress from p_o to $p_o+\Delta p$ in the compressible clay soil
e_o	=	initial void ratio corresponding to p_o, the effective overburden stress at mid depth of the clay layer
H	=	thickness of the compressible clay layer

Accordingly, the settlement of 6.5 m thick clay layer has been worked out to be 114.12 mm.

The total settlement of the raft therefore comes out to be equal to 142.62 mm. Considering the effect of placement depth and the rigidity of foundation, the appropriate correction factors (0.95 for depth and 0.8 for rigidity effect) have been applied (IS: 8009(Part-I)-1998) and therefore, the settlement of raft works out to be 108.39 mm **> 75 mm (permissible).**

Since the settlement works out to be more than the permissible one, another set of calculations have been carried out for an allowable pressure of 15 t/m^2. Accordingly, the settlement works out to be (28.5 + 69.5 =) 98 mm and after application of all the corrections the settlement of the raft becomes 74.5 mm < **75 mm (permissible).**

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13.3 RECOMMENDED ALLOWABLE BEARING PRESSURE

Following the computations made in Arts. 13.1 and 13.2 above, **it is recommended that the allowable pressure on the raft foundation be taken as 15 t/m^2.**

14.0 SUMMARY AND RECOMMENDATIONS

On the basis of limited field and laboratory geotechnical investigations carried out and analysis of tests data, following recommendations have been made:

1) Analysis of borelogs at various locations suggests that there are intermittent layers of clay/silt and sandy soils. Figure 2 showed the representative soil profiles for the site.
2) The water table was not found to occur up to the depth of exploration during March 2023.
3) Average penetration resistance (N) in various soil layers and the corresponding value of friction angle, ϕ have been shown in Fig. 2 (varying between 30 to 33°).

   The average undrained cohesion of plastic layer has been found between 8.0 to 9.0 t/m^2.
4) The site is not prone to liquefaction in the absence of water table below the ground surface.
5) The recommended value of allowable bearing pressure of raft foundation of width 25 m can be taken as 15 t/m^2.
6) In view of the fact that various buildings in the complex are quite tall (14 storeys), it is anticipated that the load intensity transferred to the soil will be quite high and hence combined pile-raft system can also serve as a foundation to support these heavily loaded tall structures.

   The capacity of the individual piles has been worked out assuming the cut-off level of piles (base of the pile cap) at -5.0 m below the ground surface in view of presence of one basement level. The individual pile capacity has been worked out for bored cast-in-situ piles with diameter as 450 mm, 600

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mm and 800 mm. In view of the soil strata stated above, the minimum length of the piles which are supposed to rest in non-plastic soil layer, has been decided as 10 m below the cut-off level. The recommended values of ultimate and the safe pile capacities for different length and diameter have been tabulated below:

Pile Length (m)	Pile Diameter (mm)	Total Ultimate Capacity, Q_{ult} (t)	Safe Pile Capacity, Q_{safe} (t)*
10	450	144.34	72.17
	600	235.70	117.85
	800	392.63	196.32

* assuming factor of safety equal to 2.

6) The permissible settlement of the pile groups has been taken as 75 and 125 mm for sands and clays respectively. Accordingly, settlement of the buildings has been worked out which are within the permissible limit.

7) Recommended values of uplift capacity of the bored cast-in-situ piles have been tabulated below:

Pile Length (m)	Pile Diameter (mm)	Uplift Capacity (t)
10	450	27.06
	600	36.93
	800	50.74

8) Recommended values of lateral capacity of the bored cast-in-situ piles have been tabulated below:

Pile Length (m)	Pile Diameter (mm)	Lateral Capacity, (t)
10	450	5.31
	600	8.41
	800	13.32

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9) The actual load carrying capacity of the piles depends upon many factors such as the quality of construction of piles, disturbance to sub-soil during construction etc. The influence of these factors is not amenable to theoretical computations. It is therefore a usual practice to confirm the theoretically predicted pile capacity by conducting pile load tests on prototype piles in-situ.

a) It is therefore, recommended that **initial load tests** be conducted on piles at the site as per Indian Standard (IS: 2911 (Part 4) - 2013) by subjecting the pile to a load level of 2.5 times the safe load carrying capacity of the piles.

b) Further, it is also advisable to conduct few **routine load tests** on arbitrarily chosen piles by subjecting the piles to a load level of 1.5 times the design load so as to check the quality of construction (IS: 2911 (Part 4) - 2013).

10) The above recommendations have been made on basis of the assumption that the sandy strata continues beyond the depth of exploration.

11) The above recommendations have been made on the basis of limited investigations conducted at the site of housing complex, Ghaziabad. However, if during construction, any deviation is observed regarding the soil type and the nature of the strata, the matter may be referred back to the authors for advice or any competent geotechnical expert.

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APPENDIX – A

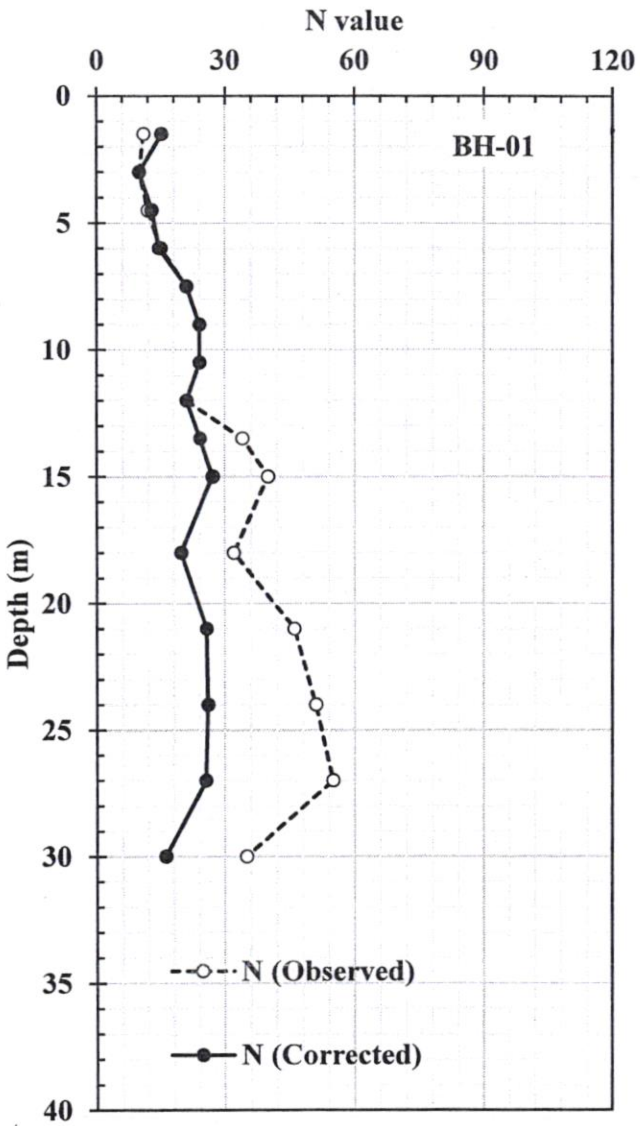

Fig. A-1 Variation of standard penetration resistance with depth (Borehole: BH–01)

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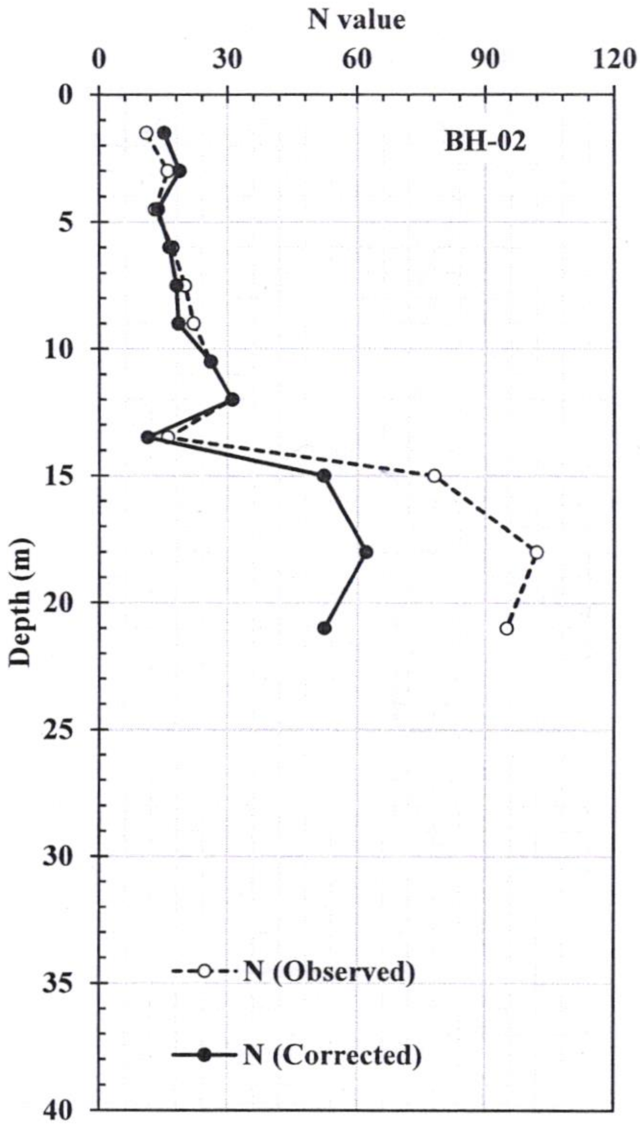

Fig. A-2 Variation of standard penetration resistance with depth (Borehole: BH–02)

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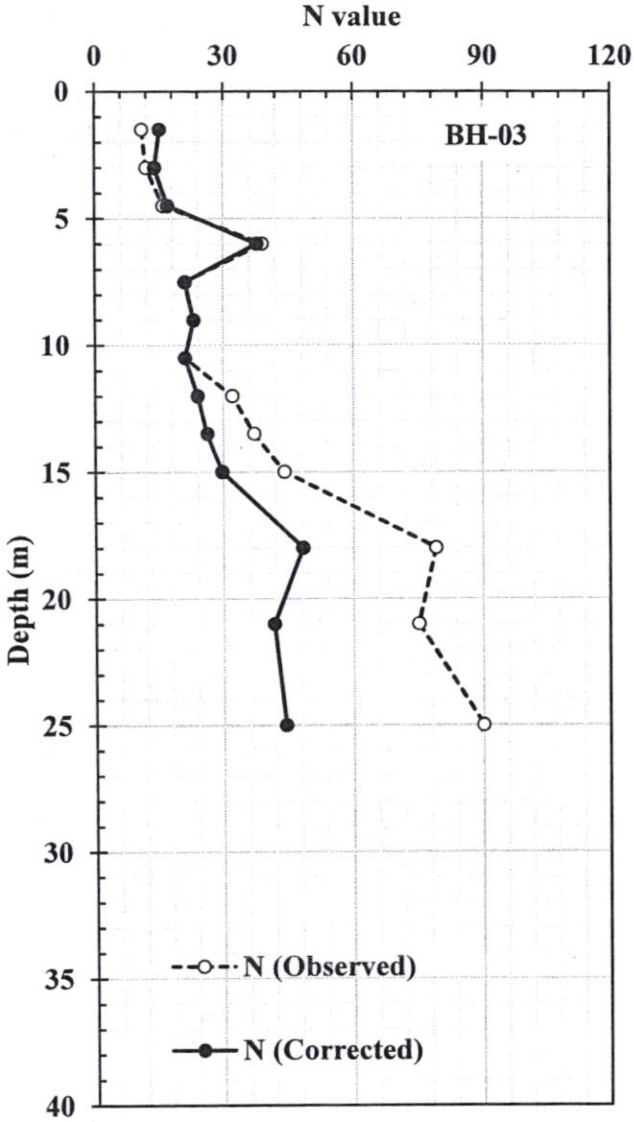

Fig. A-3 Variation of standard penetration resistance with depth (Borehole: BH–03)

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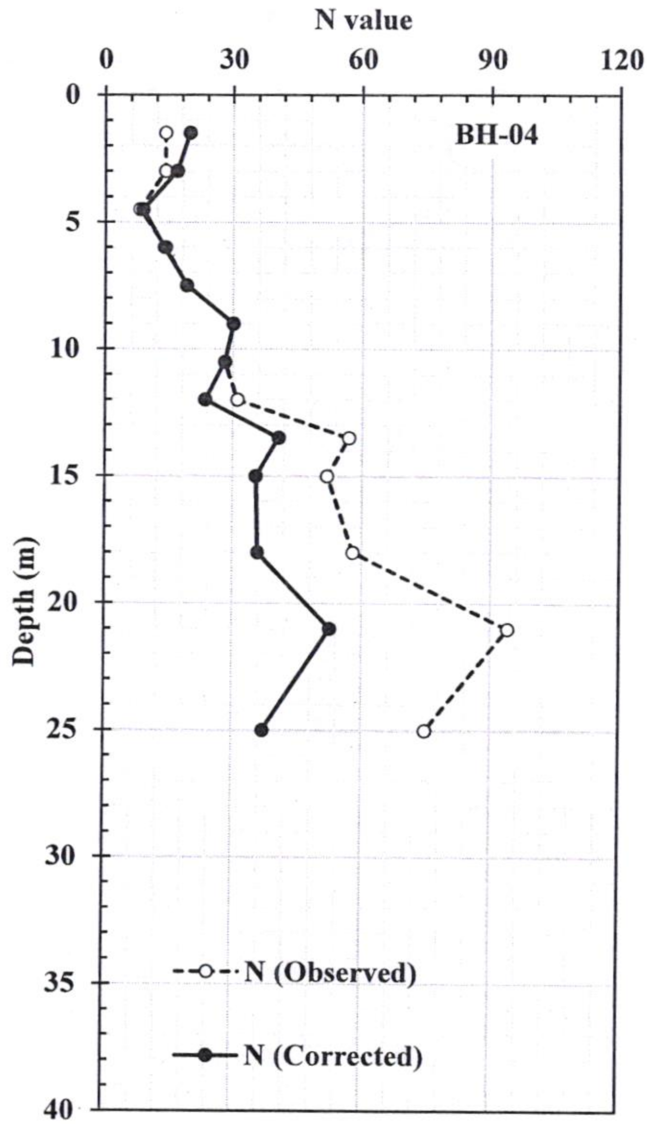

Fig. A-4 Variation of standard penetration resistance with depth (Borehole: BH–04)

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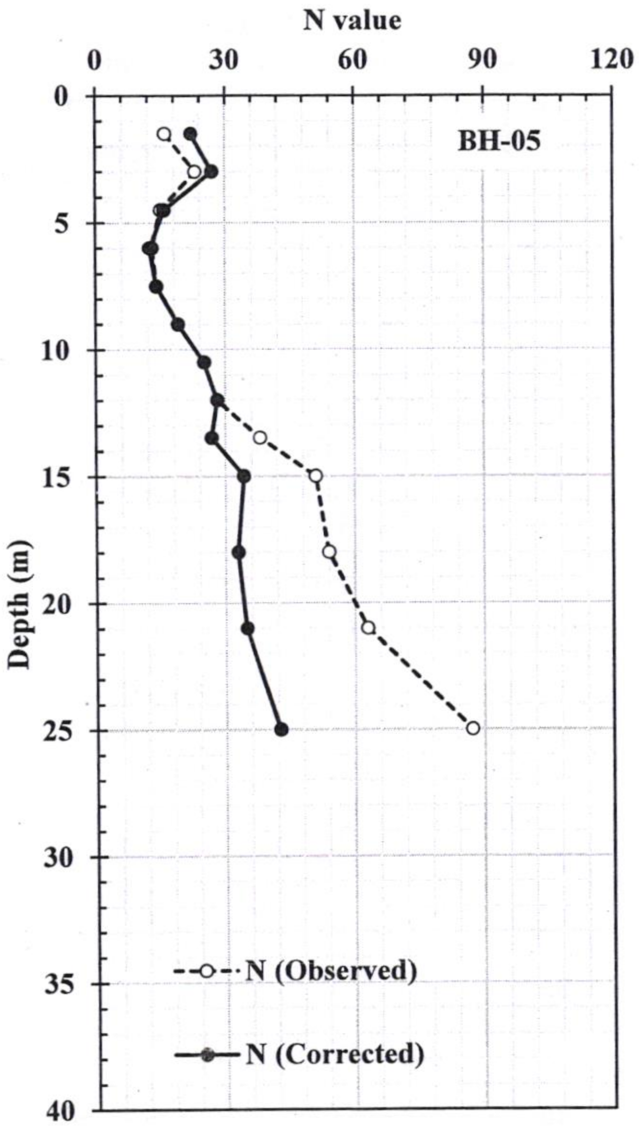

Fig. A-5 Variation of standard penetration resistance with depth (Borehole: BH–05)

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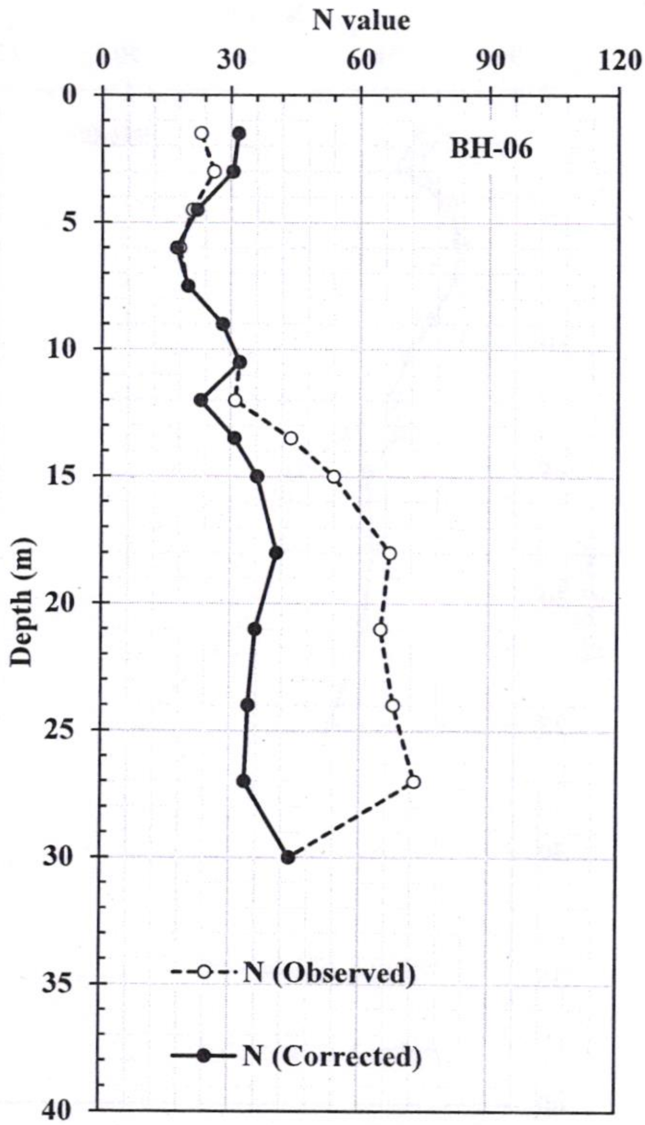

Fig. A-6 Variation of standard penetration resistance with depth (Borehole: BH–06)

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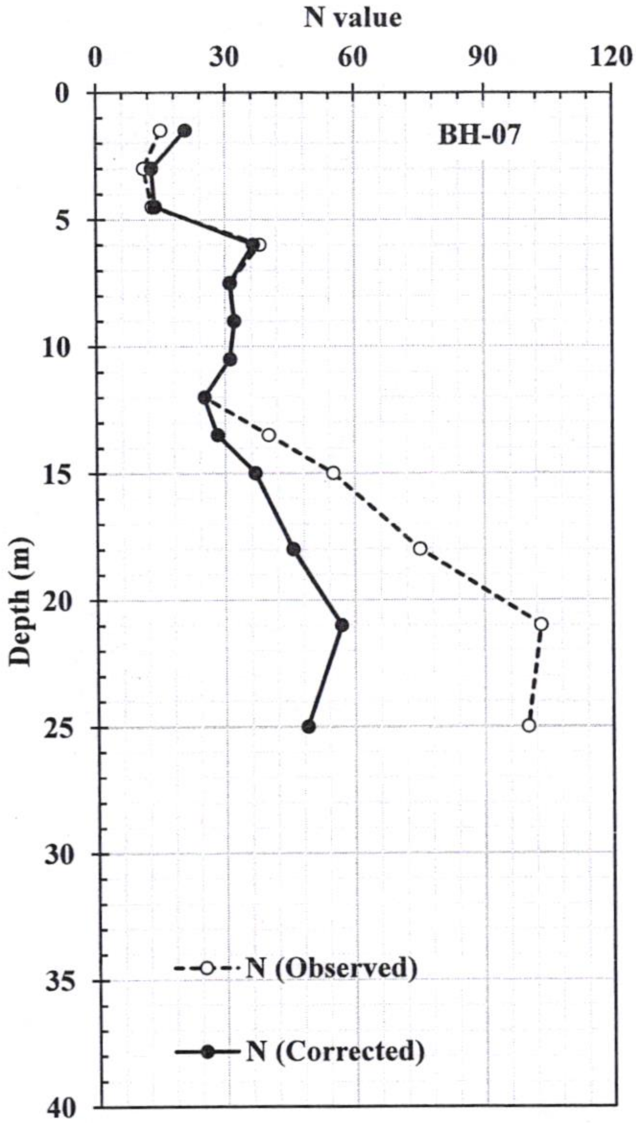

Fig. A-7 Variation of standard penetration resistance with depth (Borehole: BH–07)

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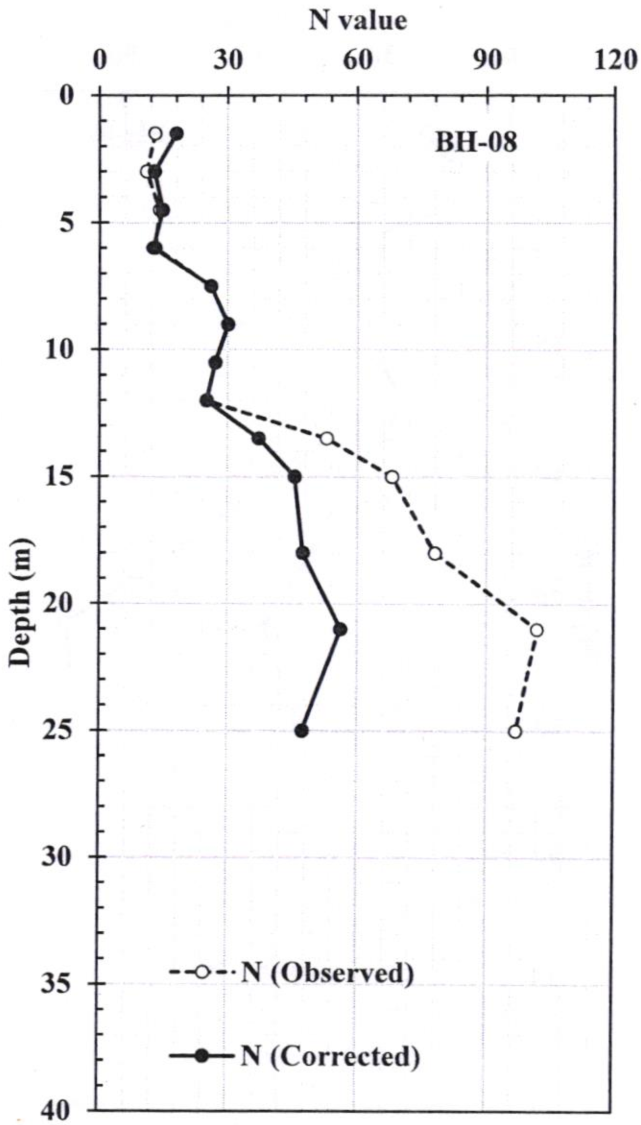

Fig. A-8 Variation of standard penetration resistance with depth (Borehole: BH–08)

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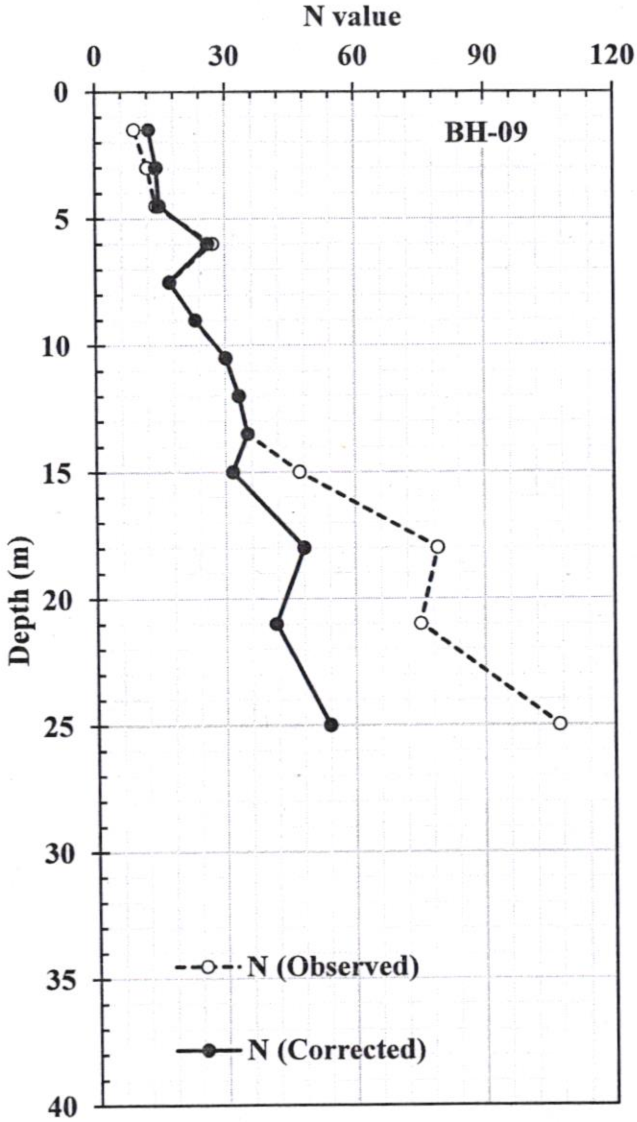

Fig. A-9 Variation of standard penetration resistance with depth (Borehole: BH–09)

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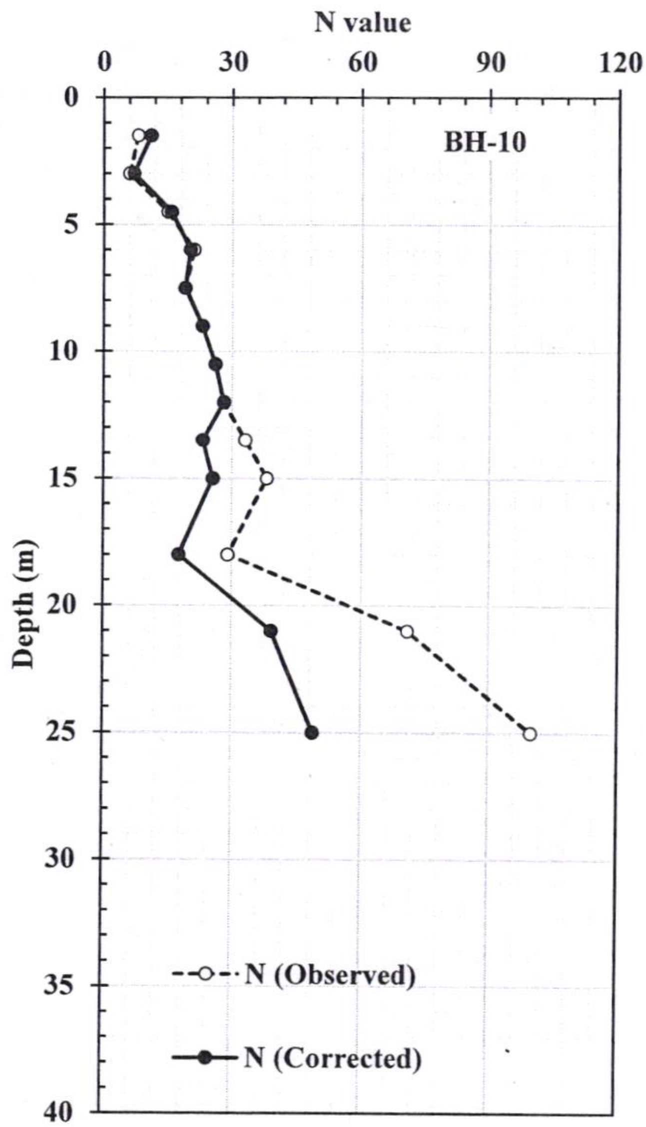

Fig. A-10 Variation of standard penetration resistance with depth (Borehole: BH–10)

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APPENDIX – B

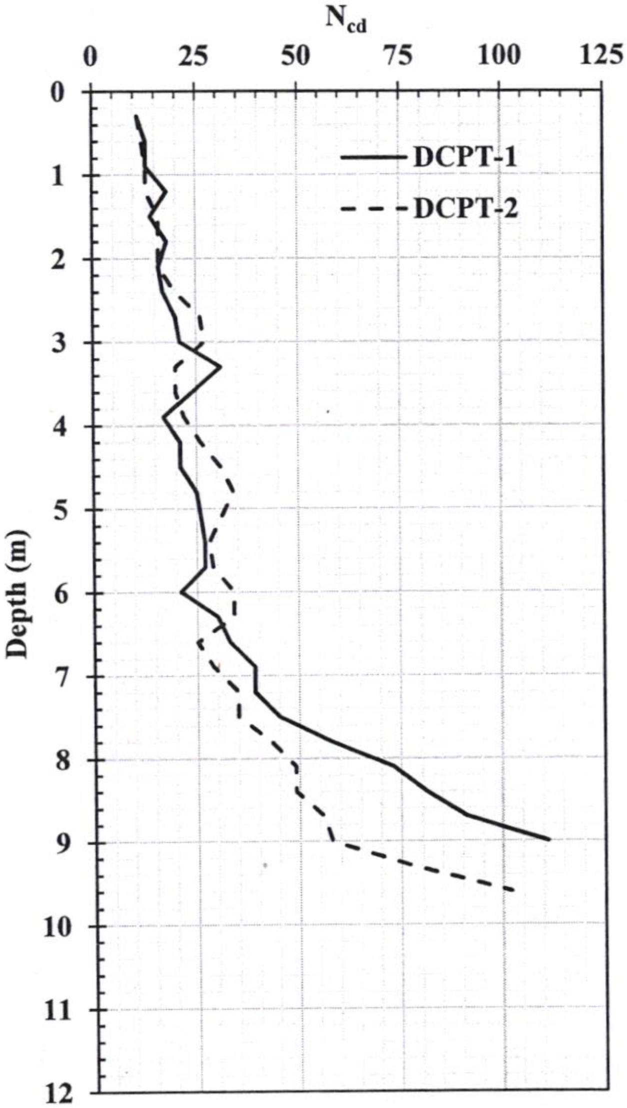

Fig. B-1 Variation of dynamic cone penetration resistance with depth (Locations: DCPT 1 and DCPT 2)

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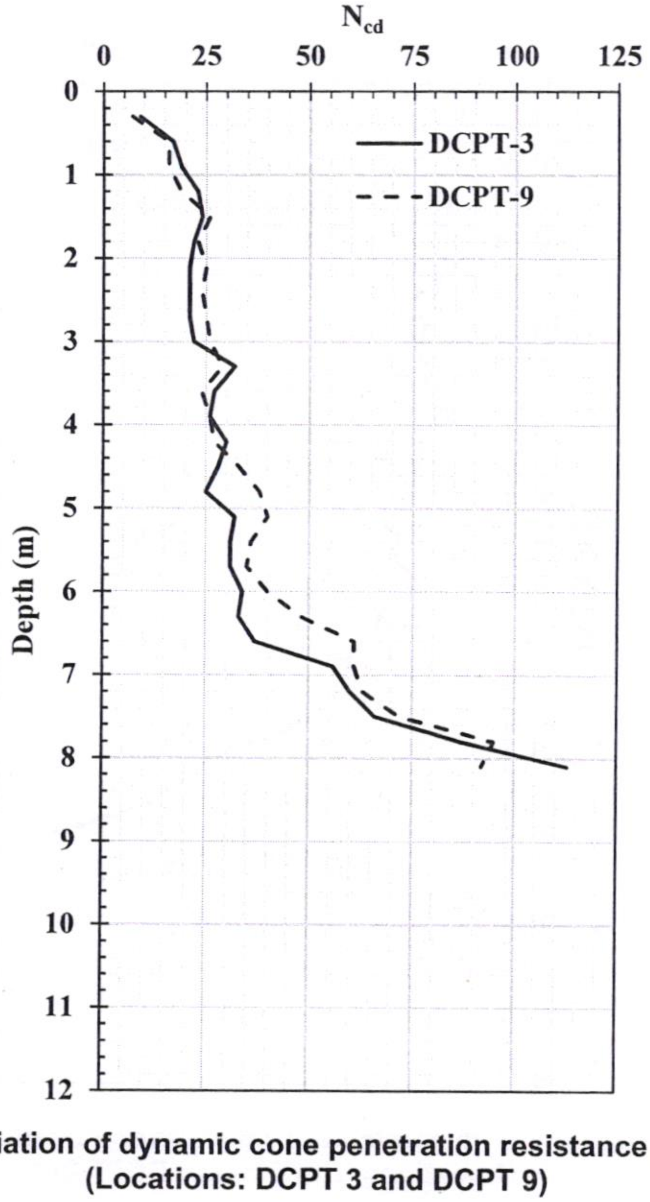

Fig. B-2 Variation of dynamic cone penetration resistance with depth (Locations: DCPT 3 and DCPT 9)

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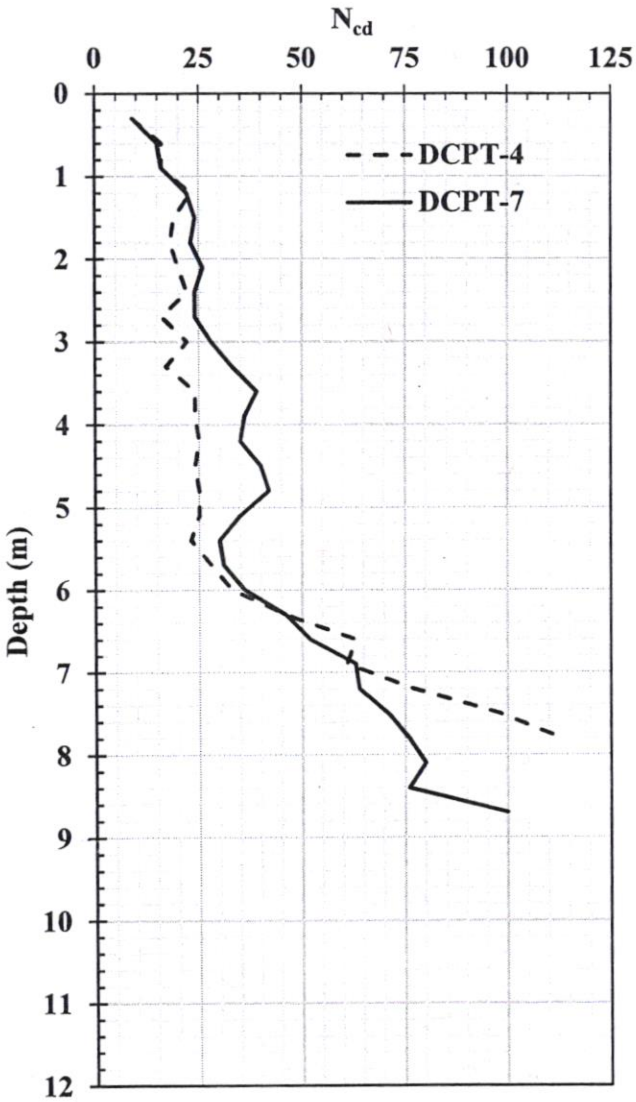

Fig. B-3 Variation of dynamic cone penetration resistance with depth (Locations: DCPT 4 and DCPT 7)

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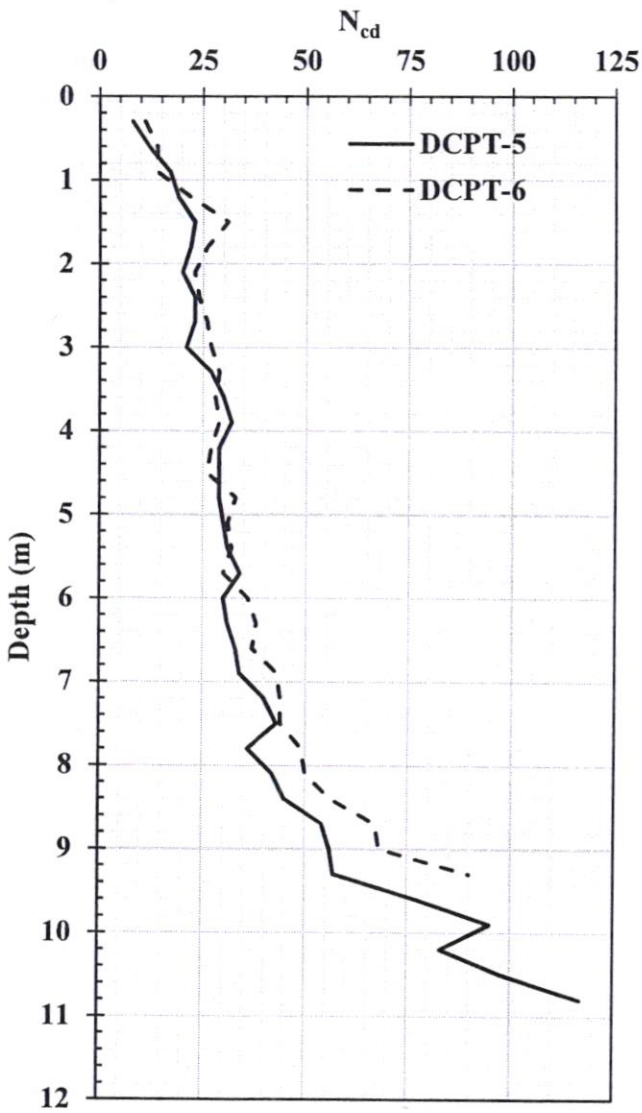

Fig. B-4 Variation of dynamic cone penetration resistance with depth (Locations: DCPT 5 and DCPT 6)

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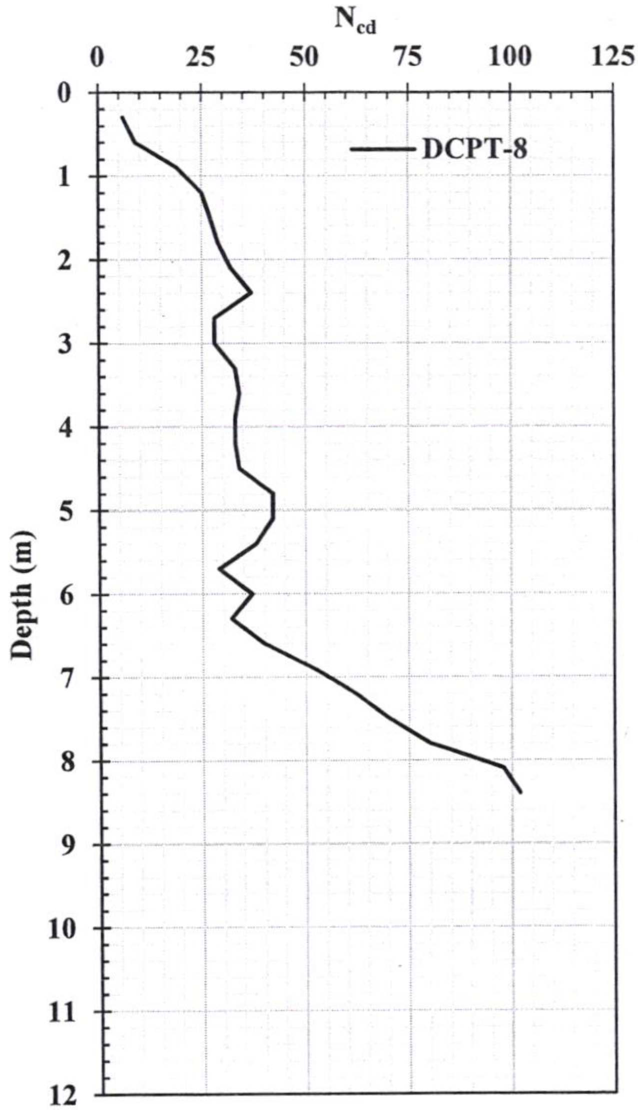

Fig. B-5 Variation of dynamic cone penetration resistance with depth (Locations: DCPT 8)

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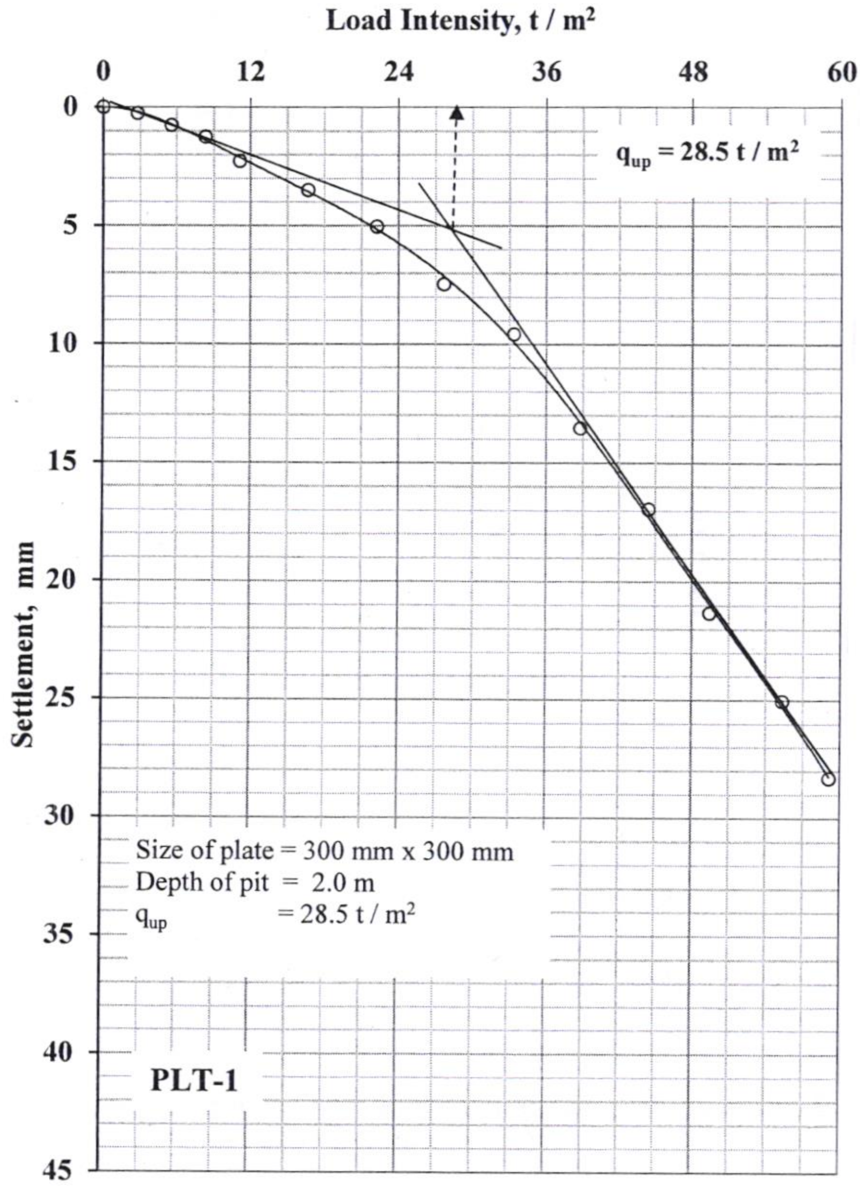

Fig. C-1 Plate load test data at location: PLT 1

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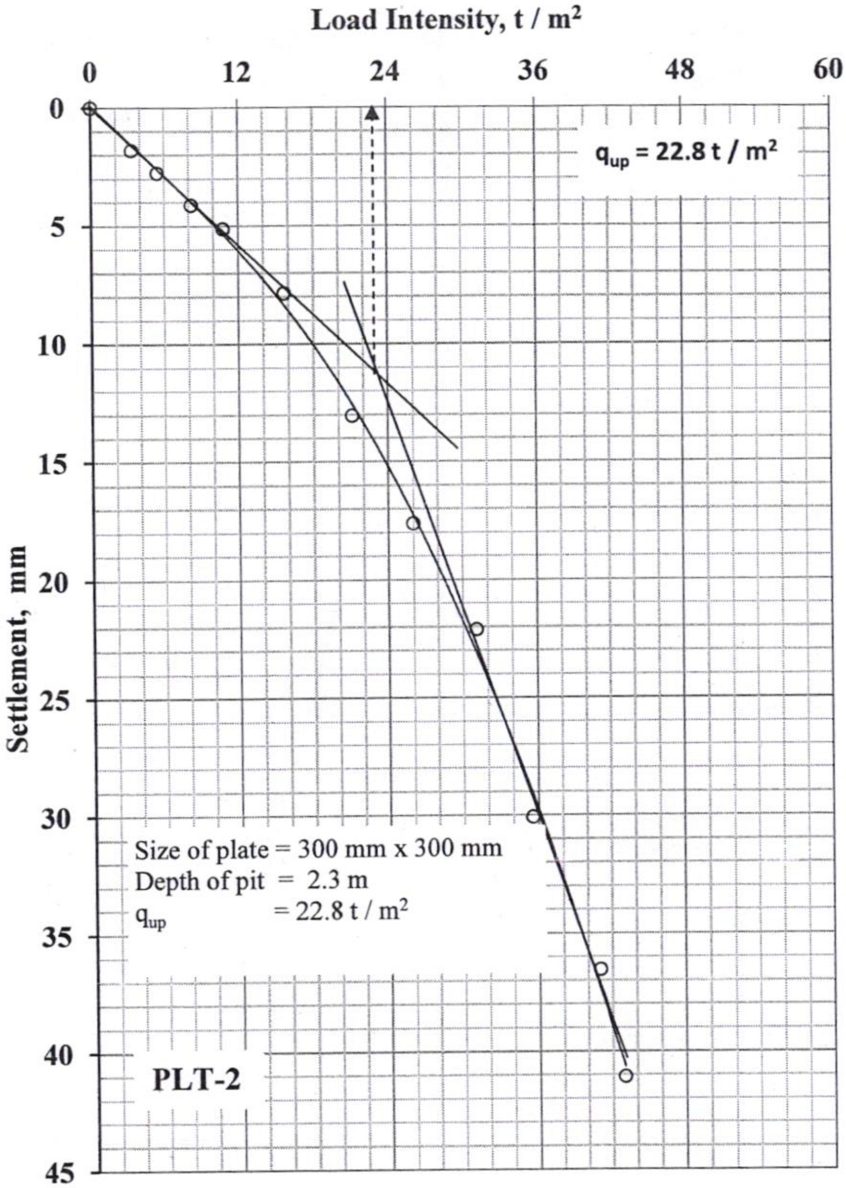

Fig. C-2 Plate load test data at location: PLT 2

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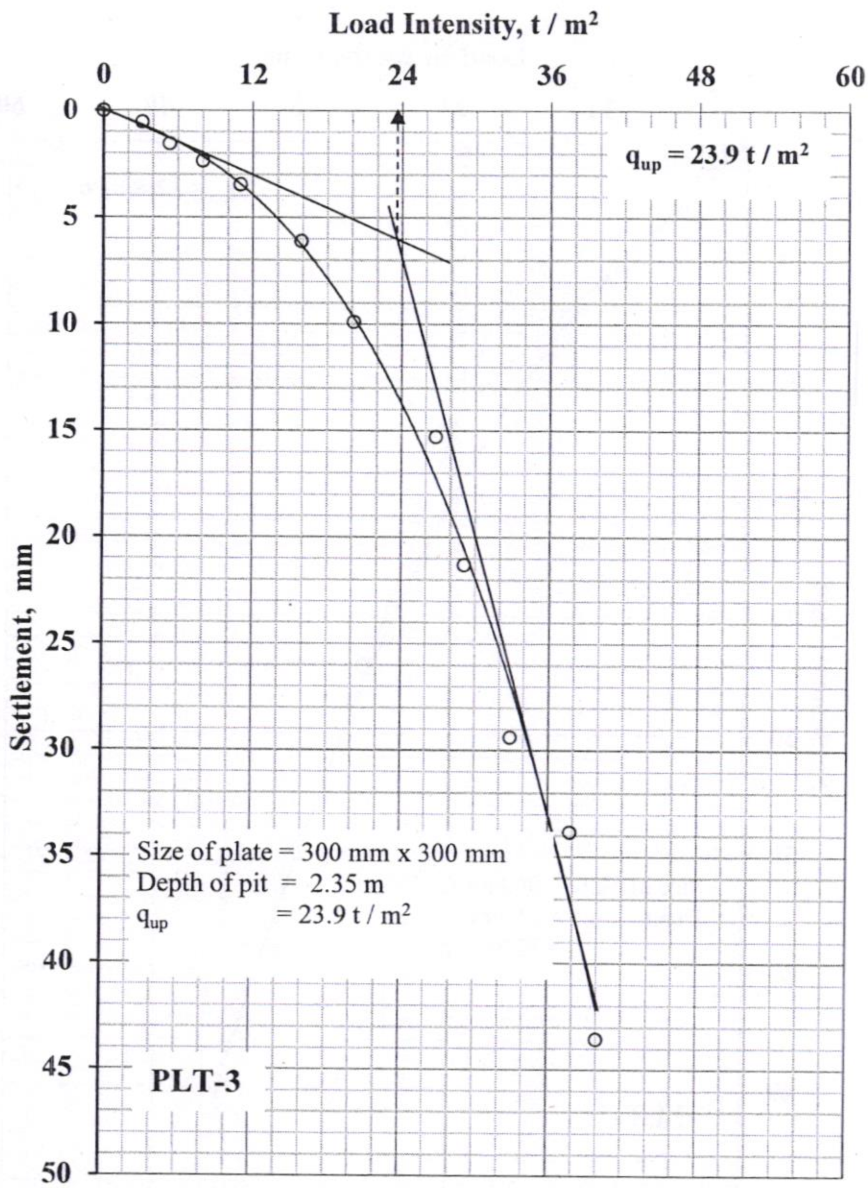

Fig. C-3 Plate load test data at location: PLT 3

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APPENDIX - D

Table D-1 Sub-soil borelog at borehole location – BH- 01

Depth (m)	I.S. Classification	Grain Size Analysis Gravels %	Sand %	Fines %	Liquid Limit %	Plastic Limit %
0.5	ML(NP)	1.2	7.2	91.6	NP	NP
1.5	CL-ML	0.0	24.5	75.5	26.6	20.9
3	SP-SM	0.0	89.1	10.9	NP	NP
4.5	SP-SM	0.0	92.1	7.9	NP	NP
6	SP-SM	2.5	90.5	7.0	NP	NP
7.5	CL	0.0	8.0	92.0	29.6	20.8
9	CL	0.7	9.9	89.4	29.6	20.7
10.5	CL	2.0	9.5	88.5	30.4	21.3
12	CL	0.0	13.8	86.2	27.9	19.9
13.5	ML(NP)	0.0	19.4	80.6	NP	NP
15	SP-SM	0.0	71.8	28.2	NP	NP
18	SP-SM	0.0	68.0	32.0	NP	NP
21	SP-SM	1.7	71.2	27.1	NP	NP
24	SP-SM	7.8	69.4	22.8	NP	NP
27	SP-SM	0.0	78.8	21.2	NP	NP
30	ML(NP)	0.0	12.8	87.2	NP	NP

Table D-2 Sub-soil borelog at borehole location – BH-02

Depth (m)	I.S. Classification	Grain Size Analysis Gravels %	Sand %	Fines %	Liquid Limit %	Plastic Limit %
0.5	ML(NP)	0.0	21.4	78.6	NP	NP
1.5	SP-SM	0.0	66.3	33.7	NP	NP
3	SP-SM	0.0	80.7	19.3	NP	NP
4.5	SP-SM	0.0	89.3	10.7	NP	NP
6	SP-SM	3.8	86.5	9.7	NP	NP
7.5	ML(NP)	6.9	12.4	80.7	NP	NP
9	ML(NP)	4.6	13.9	81.6	NP	NP
10.5	CL	33.7	10.4	55.9	31.4	20.7
12	CL	0.0	27.9	72.1	27.9	19.9
13.5	SP-SM	0.0	92.3	7.7	NP	NP
15	SP-SM	0.0	76.6	23.4	NP	NP
18	SP-SM	0.3	73.2	26.5	NP	NP
21	SP-SM	2.0	63.3	34.7	NP	NP
25	SP-SM	0.0	77.1	22.9	NP	NP

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Table D-3 Sub-soil borelog at borehole location – BH-03

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	2.1	24.0	73.9	NP	NP
1.5	SP-SM	0.0	70.5	29.5	NP	NP
3	SP-SM	0.0	80.5	19.5	NP	NP
4.5	SP-SM	2.7	87.4	9.9	NP	NP
6	ML(NP)	4.0	11.2	84.8	NP	NP
7.5	CL	0.0	14.2	85.8	28.6	20.2
9	CL	0.0	10.6	89.4	29.5	20.0
10.5	CL	1.1	16.3	82.7	28.1	18.4
12	ML(NP)	0.0	29.5	70.5	NP	NP
13.5	SP-SM	0.0	65.3	34.7	NP	NP
15	SP-SM	6.3	70.8	22.9	NP	NP
18	SP-SM	1.9	69.1	29.0	NP	NP
21	SP-SM	1.6	66.4	32.0	NP	NP
25	SP-SM	0.0	77.8	22.2	NP	NP

Table D-4 Sub-soil borelog at borehole location – BH-04

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	1.6	22.3	76.1	NP	NP
1.5	SP-SM	0.0	71.0	29.0	NP	NP
3	SP-SM	0.0	82.5	17.5	NP	NP
4.5	SP-SM	0.8	90.8	8.4	NP	NP
6	SP-SM	1.2	89.8	9.0	NP	NP
7.5	CL-ML	0.0	9.6	90.5	29.1	22.2
9	CL	3.5	10.6	85.9	29.7	20.2
10.5	CL	5.1	19.1	75.8	27.3	18.7
12	ML(NP)	0.0	27.5	72.5	NP	NP
13.5	SP-SM	0.0	63.9	36.1	NP	NP
15	SP-SM	0.0	73.3	26.7	NP	NP
18	SP-SM	0.0	66.3	33.7	NP	NP
21	SP-SM	0.0	67.0	33.0	NP	NP
25	SP-SM	0.0	75.0	25.0	NP	NP

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Table D-5 Sub-soil borelog at borehole location – BH-05

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	2.4	35.0	62.6	NP	NP
1.5	SP-SM	0.0	82.6	17.4	NP	NP
3	SP-SM	0.0	80.1	19.9	NP	NP
4.5	SP-SM	4.8	87.7	7.5	NP	NP
6	SP-SM	0.0	91.7	8.3	NP	NP
7.5	CL	0.0	9.4	90.6	29.5	20.1
9	CL	20.0	8.8	71.1	30.1	20.2
10.5	CL	2.3	10.2	87.5	28.9	18.7
12	CL	3.7	11.8	84.6	27.6	19.7
13.5	SP-SM	2.9	70.6	26.6	NP	NP
15	SP-SM	2.0	67.1	30.9	NP	NP
18	SP-SM	0.0	81.0	19.0	NP	NP
21	SP-SM	0.0	66.5	33.5	NP	NP
25	SP-SM	0.0	76.8	23.2	NP	NP

Table D-6 Sub-soil borelog at borehole location – BH-06

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	0.0	27.5	72.5	NP	NP
1.5	SP-SM	0.0	86.7	13.3	NP	NP
3	SP-SM	0.0	80.1	19.9	NP	NP
4.5	SP-SM	0.4	89.8	9.8	NP	NP
6	SP-SM	9.3	82.6	8.1	NP	NP
7.5	CL	1.9	9.9	88.2	30.0	19.4
9	CL	5.9	13.4	80.7	28.9	18.7
10.5	CL	1.4	8.7	90.0	29.5	18.5
12	ML(NP)	0.0	38.9	61.2	NP	NP
13.5	SP-SM	0.0	64.4	35.6	NP	NP
15	SP-SM	0.0	74.4	25.6	NP	NP
18	SP-SM	0.9	77.6	21.5	NP	NP
21	SP-SM	1.7	69.2	29.2	NP	NP
25	SP-SM	0.0	79.2	20.8	NP	NP
27	SP-SM	0.0	78.0	22.0	NP	NP
30	CL	0.0	10.4	89.6	29.6	22.3

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Table D-7 Sub-soil borelog at borehole location – BH-07

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	0.0	24.9	75.1	NP	NP
1.5	SP-SM	0.0	83.2	16.9	NP	NP
3	SP-SM	0.2	87.2	12.7	NP	NP
4.5	SP-SM	0.8	91.1	8.1	NP	NP
6	SP-SM	1.9	83.6	14.5	NP	NP
7.5	CL	8.6	12.9	78.5	27.3	18.2
9	CL	22.3	9.1	68.6	28.4	19.3
10.5	CL	5.6	18.1	76.3	26.6	18.1
12	CL	2.5	27.6	69.9	26.3	19.1
13.5	ML(NP)	0.0	18.5	81.5	NP	NP
15	SP-SM	0.0	69.8	30.2	NP	NP
18	SP-SM	0.0	72.7	27.3	NP	NP
21	SP-SM	0.0	66.5	33.5	NP	NP
25	SP-SM	5.7	67.3	27.0	NP	NP

Table D-8 Sub-soil borelog at borehole location – BH-08

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	0.0	17.7	82.3	NP	NP
1.5	SP-SM	0.0	79.4	20.6	NP	NP
3	SP-SM	0.5	86.9	12.6	NP	NP
4.5	SP-SM	5.0	86.8	8.2	NP	NP
6	SP-SM	3.4	90.0	6.6	NP	NP
7.5	CL	1.2	12.6	86.2	27.3	16.6
9	CL	0.9	11.7	87.4	27.7	19.0
10.5	CL	8.5	18.4	73.1	27.0	17.2
12	CL	0.0	25.5	74.5	26.4	19.2
13.5	SP-SM	0.0	81.0	19.0	NP	NP
15	SP-SM	1.6	71.2	27.2	NP	NP
18	SP-SM	1.3	85.9	12.9	NP	NP
21	SP-SM	23.6	57.0	19.4	NP	NP
25	SP-SM	0.4	77.0	22.6	NP	NP

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Table D-9 Sub-soil borelog at borehole location – BH-09

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	0.0	11.2	88.8	NP	NP
1.5	SP-SM	0.0	87.9	12.1	NP	NP
3	SP-SM	0.0	87.7	12.3	NP	NP
4.5	SP-SM	13.3	80.2	6.5	NP	NP
6	SP	6.3	89.3	4.5	NP	NP
7.5	CL	0.0	10.2	89.8	29.2	17.8
9	CL	5.8	9.6	84.7	29.7	19.2
10.5	CL	0.0	20.4	79.6	27.9	18.8
12	CL	0.0	14.9	85.1	34.1	22.4
13.5	ML	0.0	10.7	89.3	33.2	24.0
15	SP-SM	0.0	73.1	26.9	NP	NP
18	SP-SM	1.4	68.7	29.9	NP	NP
21	SP-SM	1.1	75.5	23.4	NP	NP
25	SP-SM	0.0	79.7	20.4	NP	NP

Table D-10 Sub-soil borelog at borehole location – BH-10

Depth (m)	I.S. Classification	Grain Size Analysis			Liquid Limit %	Plastic Limit %
		Gravels %	Sand %	Fines %		
0.5	ML(NP)	0.0	19.2	80.8	NP	NP
1.5	ML(NP)	0.4	17.9	81.7	NP	NP
3	SP-SM	0.2	87.7	12.1	NP	NP
4.5	SP-SM	0.1	72.8	27.1	NP	NP
6	SP-SM	3.9	89.6	6.5	NP	NP
7.5	CL	9.8	20.3	69.8	29.0	17.5
9	CL	0.8	15.8	83.4	27.5	18.1
10.5	CL	4.5	9.3	86.2	28.3	18.7
12	CL	8.6	26.2	65.2	29.6	20.5
13.5	ML(NP)	0.0	13.1	86.9	NP	NP
15	ML(NP)	0.0	43.2	56.8	NP	NP
18	SP-SM	0.0	79.1	20.9	NP	NP
21	SP-SM	4.1	78.7	17.2	NP	NP
25	SP-SM	0.1	91.8	8.0	NP	NP

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APPENDIX - E

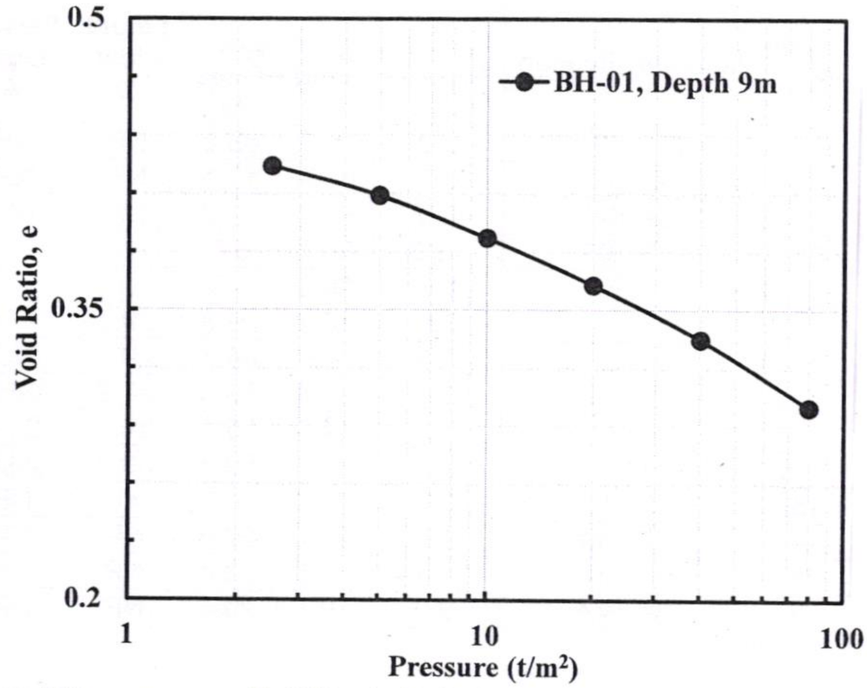

Fig. E-1 Pressure vs. Void Ratio Relationship from Consolidation Test (Location: BH-01, Depth = 9.0 m)

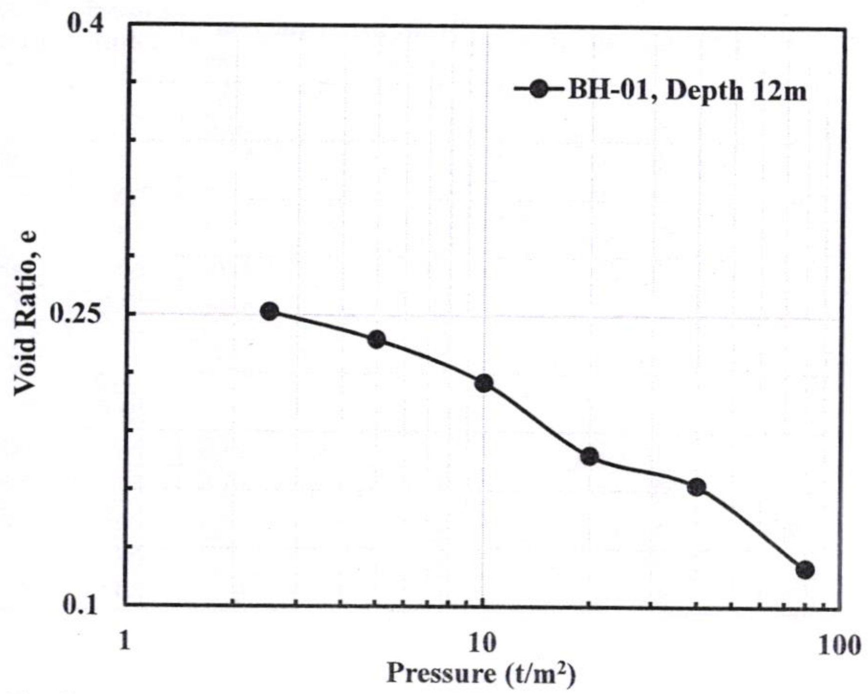

Fig. E-2 Pressure vs. Void Ratio Relationship from Consolidation Test (Location: BH-01, Depth = 12.0 m)

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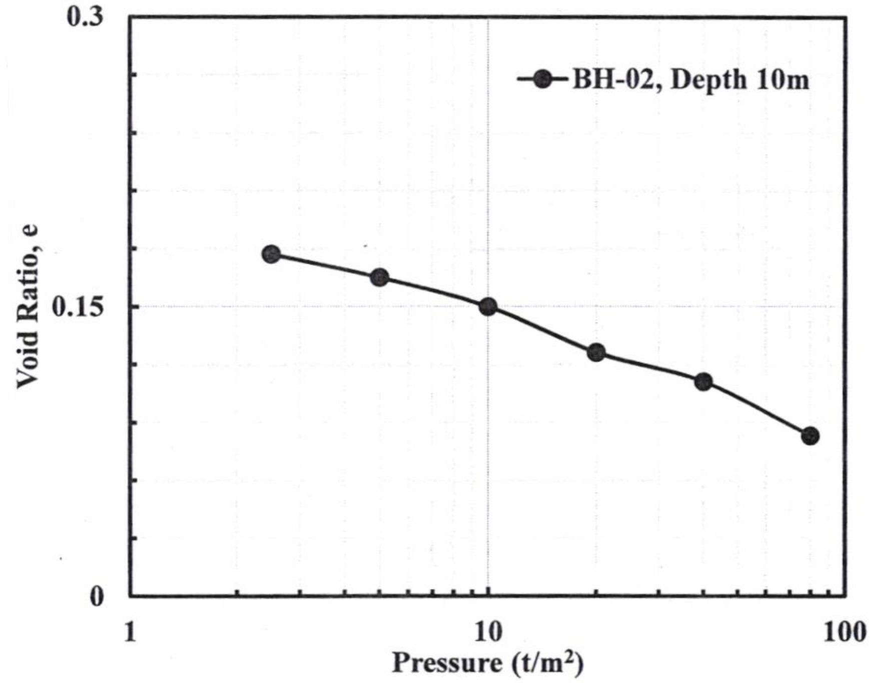

Fig. E-3 Pressure vs. Void Ratio Relationship from Consolidation Test
(Location: BH-02, Depth = 10.0 m)

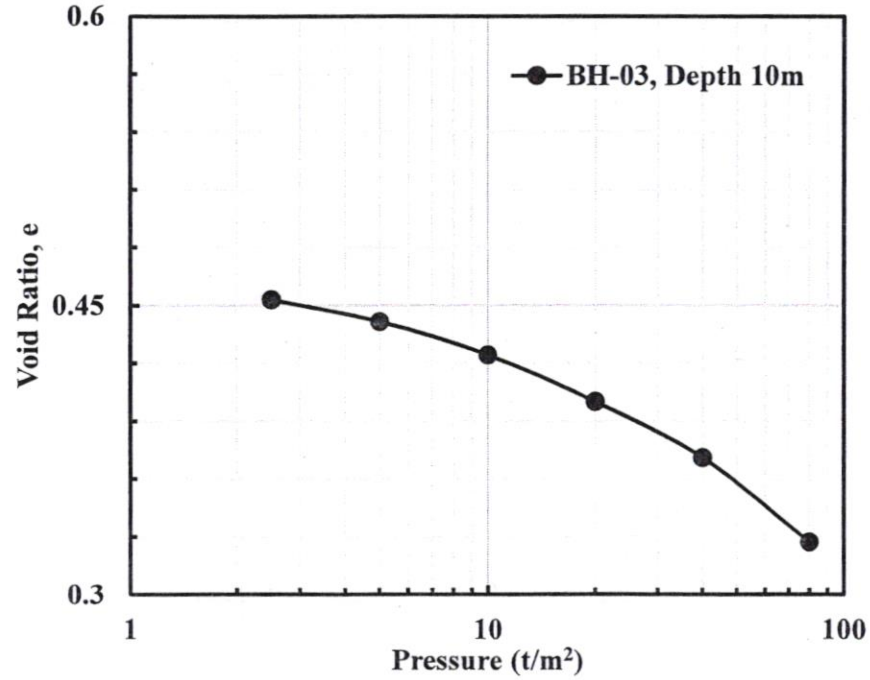

Fig. E-4 Pressure vs. Void Ratio Relationship from Consolidation Test
(Location: BH-03, Depth = 10.0 m)

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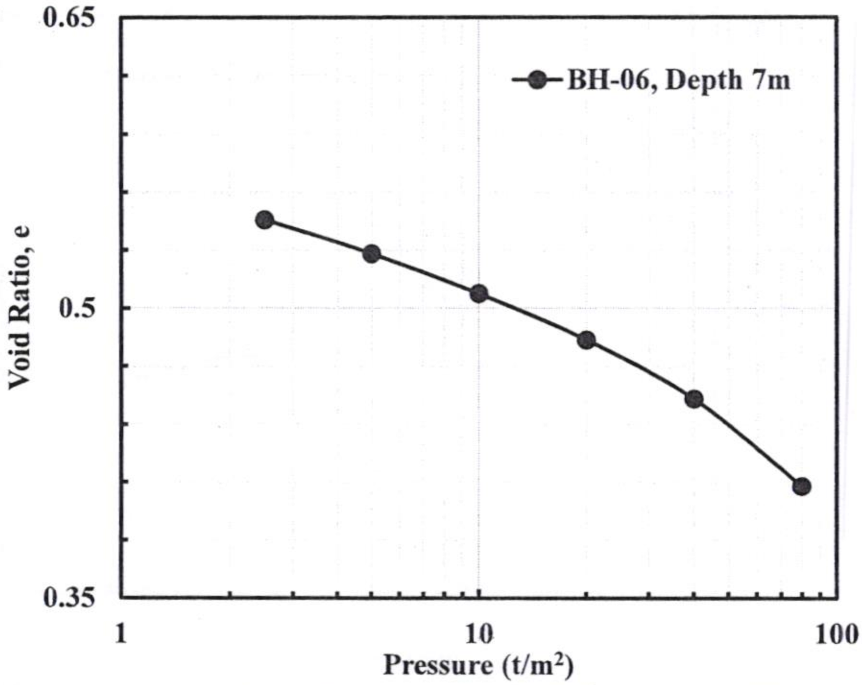

Fig. E-5 Pressure vs. Void Ratio Relationship from Consolidation Test (Location: BH-06, Depth = 7.0 m)

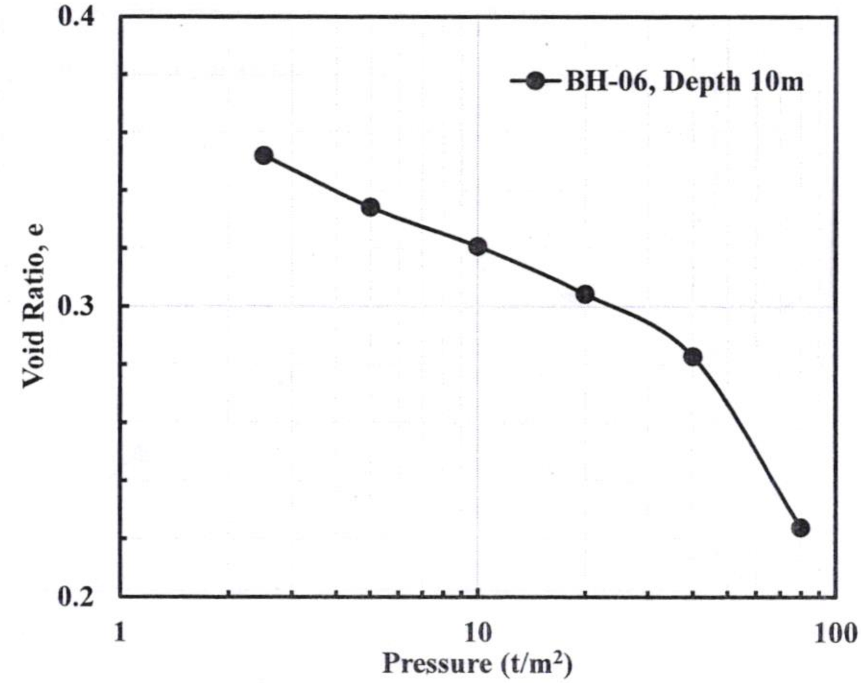

Fig. E-6 Pressure vs. Void Ratio Relationship from Consolidation Test (Location: BH-06, Depth = 10.0 m)

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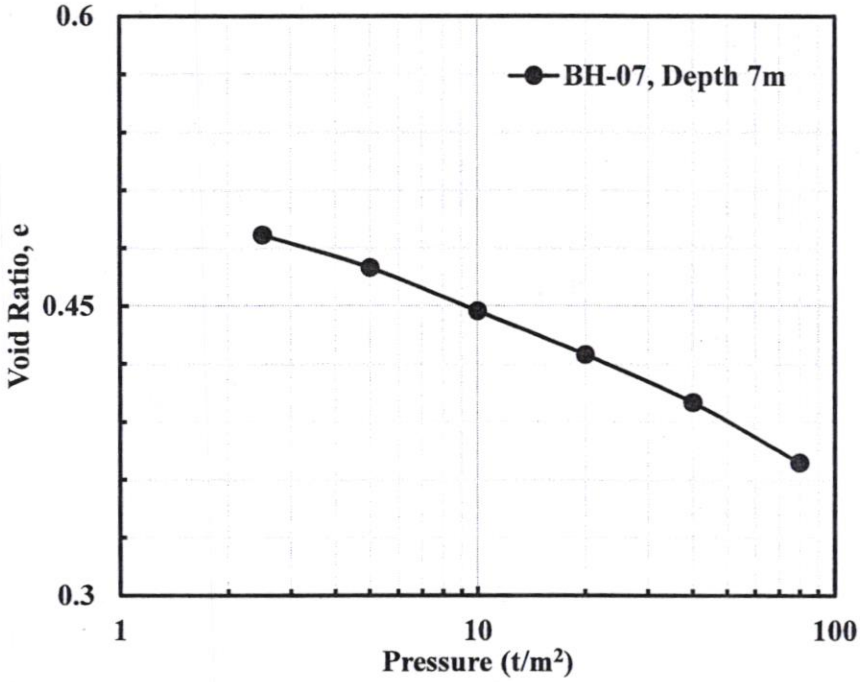

Fig. E-7 Pressure vs. Void Ratio Relationship from Consolidation Test (Location: BH-07, Depth = 7.0 m)

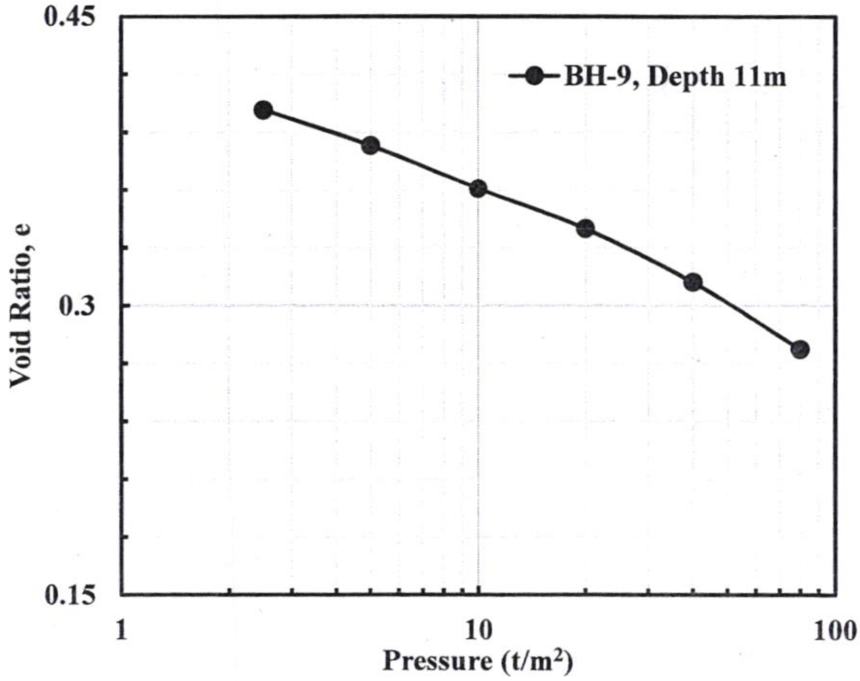

Fig. E-8 Pressure vs. Void Ratio Relationship from Consolidation Test (Location: BH-09, Depth = 11.0 m)

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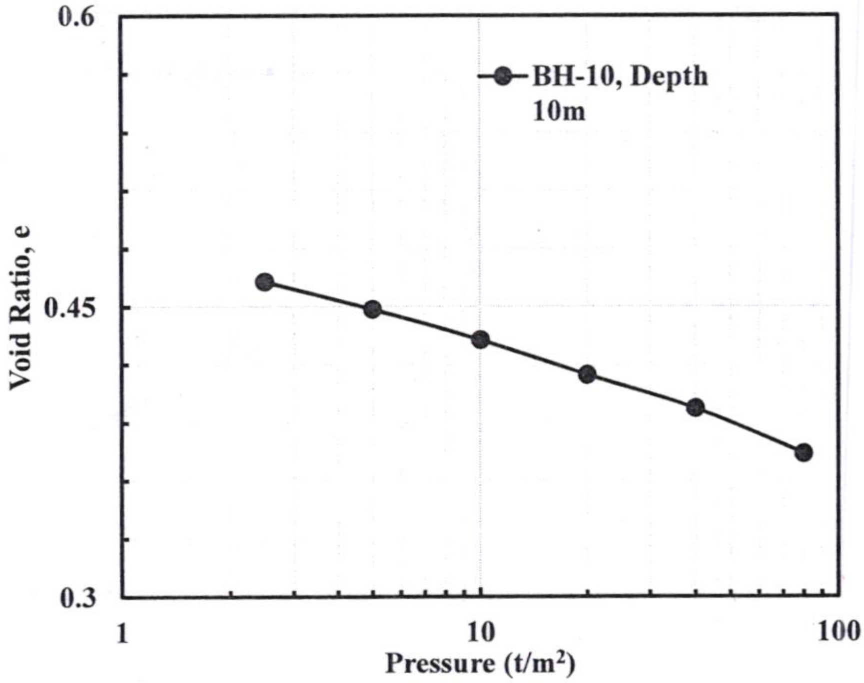

Fig. E-9 Pressure vs. Void Ratio Relationship from Consolidation Test (Location: BH-10, Depth = 10.0 m)

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