

**Report
On
Sub - Soil Investigation
For
Proposed Housing Complex
At**

**Mouza - Amtala, P.S. - Bishnupur, Block -
Bishnupur -2, J.L. -73, Khatian No. - 923,
R.S. Dag No. - Part of 838, 839, 840, 841, 842,
843, part of 846, under L.R. Dag No. - 1847,
1848, 1849, 1850, 1852, Part of 1846, P.O. -
Kanyanagar, P.S. - Bishnupur, under Chandi
Gram Panchayet, Dist. - 24 Pgs.(S).
(PH - 2B) (B.H.5 to B.H.12)**

Name of Client

SRIJAN STAR REALTY LLP.

Name of Architect

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Name of Executing Agency

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REPORT NO. : GT / SS / 143 / 2017 - 2018



**REPORT
ON
SUB - SOIL INVESTIGATION
FOR
PROPOSED HOUSING COMPLEX
AT**

**MOUZA - AMTALA, P.S. - BISHNUPUR, BLOCK - BISHNUPUR -2,
J.L. -73, KHATIAN NO. - 923, R.S. DAG NO. - PART OF 838, 839, 840, 841,
842, 843, PART OF 846, UNDER L.R. DAG NO. - 1847, 1848, 1849, 1850,
1852, PART OF 1846, P.O. - KANYANAGAR, P.S. - BISHNUPUR,
UNDER CHANDI GRAM PANCHYET, DIST. - 24 PGS.(S).
(PH - 2B) (B.H.5 TO B.H.12)**

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1. INTRODUCTION

The work of sub-soil exploration for the Proposed Housing Complex at Mouza - Amtala, P.S. - Bishnupur, Block - Bishnupur -2, J.L. -73, Khatlan No. - 923, R.S. Dag No. - Part of 838, 839, 840, 841, 842, 843, part of 846, under L.R. Dag No. - 1847, 1848, 1849, 1850, 1852, Part of 1846, P.O. - Kanyanagar, P.S. - Bishnupur, under Chandi Gram Panchayet, Dist. - 24 Pgs.(S). (PH - 2B) (B.H.5 to B.H.12) was awarded to M/S. GEOTEST ENGINEERS PVT. LTD. of 6A, Milan Park, Kolkata - 700 084 by the client SRIJAN STAR REALTY LLP. The sub-soil investigation purpose for the proposed buildings at the aforesaid site was to determine the sub-soil condition and to ascertain the foundation types that would be suitable for the proposed buildings. The Fieldwork was done in the months of January & February, 2018. Laboratory tests were conducted on soil samples at our own laboratory, for the analysis of sub-soil condition at the site.

2. SCOPE OF INVESTIGATION

In an attempt for optimization in the design of foundations for the proposed buildings at this site, geotechnical investigation programme had been divided mainly into two parts, like, field works part unfurling the sub-surface deposit types and their states of occurrences in-situ and laboratory tests part which would help to determine the relevant physical and the geotechnical properties of the sub-surface deposits leading to finalization of foundation type and foundation design bearing capacity with particular reference to the sub-surface deposit types and their strength parameters and settlement potentials in-situ. The scope is summarized as follows: -

- (a) Sinking 8 (Eight) numbers, 150 mm dia. exploratory boreholes, 5 (Five) with termination depths of about 20.00 m. below E.G.L. each and remaining 3 (three) with that of about 10.00 m. below E.G.L. each at prefixed location at the site. The borehole numbers, depths & locations were finalized & fixed by the **Client**. For location of the exploratory boreholes please refer 'Borehole Location Plan' at the Annexure of this report.
- (b) Collection of representative 100 mm dia. undisturbed soil samples as per the provisions as laid down in IS: 2132 (1986) as well as representative disturbed soil samples from the exploratory boreholes for carrying out detailed laboratory analysis which would help adoption of strength, settlement and other relevant parameters of the sub-surface deposits



- (c) *Carrying out standard penetration tests as per the provisions laid down in IS: 2131 (1981) in all the boreholes and subsequently maintaining penetration chart, depth-wise, upto the test depth in all the exploratory boreholes at this site.*

After completion of the above-mentioned field works, the appropriate laboratory tests, as were applicable to the sub-surface deposit types which were encountered at the explored location, were conducted to determine the physical and the relevant geotechnical properties of the sub-surface deposits and subsequently to finalize type and thereafter for design of foundation for the proposed buildings to be constructed at this site under investigation.

3. FIELD WORK

A brief description of boring method, field tests, sample collection etc. and type of equipment, are furnished below.

3.1 Rig

The entire fieldwork was done by deploying single number of rig.

3.2 Boring

Boring through the soil was carried out by Shell & Auger boring technique upto their termination depths below E.G.L. in all the boreholes by Mechanically Power-Driven Winch, by providing casing throughout the explored depths.

3.3 Representative Sample

Representative samples were collected from auger, S.P.T. sampler and cutting shoe of undisturbed sampling assembly. This was done to maintain a continuous record of strata encountered. The samples were labeled and placed in airtight polythene bags and shifted to the laboratory for testing and classification.

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3.4 Standard Penetration Test (S.P.T.)

This test was conducted at the boring points at suitable intervals. The number of blows required for last 30.00 cm penetration of split spoon sampler out of a total penetration of 45.00 cm driven by a 63.50 kg hammer falling freely through a height of 75.00 cm was recorded as 'N' values. The sample from split spoon were collected after each test and were labeled and placed in air-tight polythene bags before sending to the laboratory for identification and testing. The test procedure was performed to IS: 2131 (1981) (Reaffirmed 1987). The Split Spoon Sampler was as per I.S. 9640:1980 (Reaffirmed 1987).

3.5 Undisturbed Samples

Undisturbed samples were collected as per I.S. 2132 (1986) by means of a two-tier 100.00 mm I.D. open driven sampling assembly having area ratio of 15%. The sampling assembly (as per I.S. 11594: 1985) was driven to the required depth manually with the help of jarring link. Samples collected in the lower tube were retained, labeled and waxed at both ends before sending it to the laboratory.

3.6 Standing Water Level

Standing water level observation was made during boring in all the boreholes after 24 hours of completion of boreholes.

During the period of field work from 29.01.2018 to 06.02.2018, the standing water level were recorded and have been presented in Tabular form in the next page:

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Ground Water Table
Table - I

B.H.Nos. Dates	DEPTH OF GROUND WATER TABLE BELOW E.G.L (M.)										
	B.H.5	B.H.6	B.H.7	B.H.8	B.H.9	B.H.10	B.H.11	B.H.12			
26.01.2018	-	-	-	-	-	-	-	-			
27.01.2018	-	-	-	-	-	-	-	-			
28.01.2018	-	-	-	-	-	-	-	-			
29.01.2018	-	-	-	-	-	-	-	-			
30.01.2018	-	-	-	-	-	-	-	-			
31.01.2018	1.60 m.	-	-	-	-	-	-	-			
01.02.2018	1.50 m.	0.90 m.	-	-	-	-	-	-			
02.02.2018	1.45 m.	0.80 m.	0.95 m.	-	-	-	-	-			
03.02.2018	1.40 m.	0.70 m.	0.90 m.	0.80 m.	-	-	-	-			
04.02.2018	1.30 m.	0.65 m.	0.80 m.	0.75 m.	0.75 m.	-	-	-			
05.02.2018	1.25 m.	0.60 m.	0.78 m.	0.70 m.	0.70 m.	0.00 m.	0.00 m.	0.00 m.			
06.02.2018	1.17 m.	0.50 m.	0.70 m.	0.60 m.	0.60 m.	0.00 m.	0.00 m.	0.00 m.			

NOTE : For B.H.1 to B.H.4, refer PH-2A report.



4. LABORATORY INVESTIGATION

For proper identification and classification of the sub-surface deposits and for deriving adequate informations regarding its relevant physical and the geotechnical properties at the site under investigation, most or all of the following laboratory tests pertaining to the actual soil types, occurring at this site, were conducted on the representative soil samples, collected from all the exploratory boreholes.

- (a) Grain size analysis.
 - (i) Hydrometer analysis for cohesive soil samples.
- (b) Liquid limit and plastic limit for cohesive soil samples.
- (c) Specific Gravity.
- (d) Natural moisture content.
- (e) Natural density and dry density.
- (f) Unconfined compression tests on undisturbed cohesive soil samples.
- (g) Triaxial shear tests in unconsolidated Undrained condition on cohesive soil samples for determination of strength parameter values like, cohesion, C_{un} and angle of internal friction, ϕ_{un} .
- (h) Triaxial shear tests in consolidated drained condition on granular soil samples for determination of strength parameter values like, angle of internal friction, ϕ_d .
- (i) Direct shear tests on granular soil samples for determination of strength parameter value like, angle of internal friction, ϕ_d .
- (j) Consolidation tests on cohesive soil samples for determination of settlement potentials.
- (k) Void ratio.

All or most of the above-mentioned laboratory tests on the representative soil samples were conducted as per the relevant provisions as laid down in the different sections of IS: 2720.

The result after the relevant laboratory tests on the representative soil samples have been presented in tabular form in 'Laboratory Test Result Sheet' at the end of this report.

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5. SUB-SOIL STRATIFICATION

The sub-soil profile as revealed by all the boreholes is shown in 'Sub-Soil Profile' and in 'Table 2'. On the basis of extensive field and laboratory tests on disturbed and undisturbed soil specimens and on visual inspection, the classification of different strata and engineering properties of soil are discussed below:

Stratum I Reclaimed top fill of soft grey silty clay with traces of grass roots and tree plants.

The extensions of this layer in the Boreholes are as follows: -

Borehole Mark	Measured with Respect To E.G.L. (m)	
	From	To
B.H.5	*0.00	1.00
B.H.6	*0.00	0.70
B.H.7	*0.00	0.70
B.H.8	*0.00	1.20
B.H.9	*0.00	0.50
B.H.10	*0.00	1.80
B.H.11	*0.00	1.00
B.H.12	*0.00	0.70

*During fieldwork, E.G.L. was about 1.47 m., 1.58 m., 1.65 m., 1.68 m., 1.52 m., 1.72 m., 1.73 m., 1.74 m. below D.H. Road level in B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.10, B.H.11, B.H.12 respectively.

This layer consists of reclaimed top fill of very soft / soft grey silty clay with traces of grass roots and tree plants. Field 'N' value of this layer is 03, which indicates its soft state of consistency.

I.S. Classification

= CH

Stratum II Soft / firm brownish grey / grey silty clay / clayey silt with traces of kankars and rusty brown silt spots. (Absent in B.H.5, B.H.6 & B.H.8 only).



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The extensions of this layer in the Boreholes are as follows: -

Borehole Mark	Measured with Respect to E.G.L. (m)	
	From	To
B.H.7	0.70	2.50
B.H.9	0.50	3.50
B.H.10	1.80	2.80
B.H.11	1.00	3.30
B.H.12	0.70	2.70

This layer contains of soft / firm brownish grey / grey silty clay / clayey silt with traces of kankars and rusty brown silt spots. Traces of conch shell were observed in B.H.12. Field 'N' value of this layer is 03, Combining 'N' value and 'C' values, it is understood that this layer has soft / firm state of consistency. The relevant engineering properties are mentioned herewith:

Field 'N' value	= 03
Bulk Density	= 18.22 kN/m ³ to 18.43 kN/m ³
Dry Density	= 13.60 kN/m ³ to 13.93 kN/m ³
Natural Moisture Content	= 32.25 % to 33.91 %
Liquid Limit	= 64 % to 65 %
Plastic Limit	= 23 % to 24 %
Undrained Cohesion 'C'	= 26.07 kN/m ² to 28.12 kN/m ² (From U.C.S. Test)
Undrained Cohesion 'C _u '	= 27.89 kN/m ² (From Triaxial U.U. Test)
Undrained angle of shear resistance 'φ _u '	= 0° (From Triaxial U.U. Test)
Specific Gravity	= 2.65 to 2.66
Initial void ratio 'e ₀ '	= 0.854 to 0.898

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Grain Size Analysis (Hydrometer Analysis):

Sand	= 04 %
Silt	= 52 % to 54 %
Clay	= 42 % to 44 %
I.S. Classification	= CH - MH

The following are the average coefficient of volume compressibility (m_v) values of this stratum:

Pressure Range in kg/cm^2	m_v (m^2/kN)
0.00 - 0.25	0.000498
0.25 - 0.50	0.000441
0.50 - 1.00	0.000324
1.00 - 2.00	0.000220
2.00 - 4.00	0.000147
4.00 - 8.00	0.00097

Stratum III Very soft / soft grey silty clay with varying percentage of decomposed wood.
(traces to medium to high percentage).

The extensions of this layer in the Boreholes are as follows: -

Borehole Mark	Measured with Respect to E.G.L. (m)	
	From	To
B.H.5	2.60	12.20
B.H.6	0.70	***10.45
B.H.7	2.50	14.70
B.H.8	1.20	16.00
B.H.9	3.50	***10.05
B.H.10	2.80	14.60
B.H.11	3.30	***10.45
B.H.12	2.70	13.60

***Termination depths of the boreholes.

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This layer contains of very soft / soft grey silty clay with varying percentage of decomposed wood. Traces to medium to high percentage of decomposed wood were observed in B.H.5, B.H.6, B.H.9. Traces to medium percentage of decomposed wood were observed in B.H.7, B.H.10, B.H.11, B.H.12. High percentage of peat was also observed at 2.00 m. & 2.50 m. in B.H.5 & B.H.12 respectively. Medium percentage of decomposed wood was observed only at 15.50 m. in B.H.8. Field 'N' value of this layer ranges from 02 to 03, indicating its very soft / soft state of consistency. The relevant engineering properties are mentioned herewith:

Field 'N' value	= 02 to 03
Bulk Density	= 15.22 kN/m ³ to 18.01 kN/m ³
Dry Density	= 9.04 kN/m ³ to 13.52 kN/m ³
Natural Moisture Content	= 33.21 % to 59.25 % / **64.46 % to **193.54
Liquid Limit	= 63 % to 87 % / **93 % to **103 %
Plastic Limit	= 21 % to 29 % / **34 % to **36 %
Undrained Cohesion 'C'	= 14.28 kN/m ² to 27.81 kN/m ² (From U.C.S. Test)
Undrained Cohesion 'C _u '	= 16.24 kN/m ² to 26.44 kN/m ² (From Triaxial U.U. Test)
Undrained angle of shear resistance ' ϕ_u '	= 0° (From Triaxial U.U. Test)
Specific Gravity	= 2.48 to 2.60
Initial void ratio 'e ₀ '	= 0.858 to 1.320
Grain Size Analysis (Hydrometer Analysis):	
Sand	= 01 % to 03 %
Silt	= 49 % to 53 %
Clay	= 44 % to 48 %
I.S. Classification	= CH - OH

****High values due to presence of high percentage of decomposed wood in soil sample.**

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The following are the average coefficient of volume compressibility (m_v) values of this stratum:

Pressure Range in kg/cm^2	m_v (m^2/kN)
0.00 – 0.25	0.000663
0.25 – 0.50	0.000619
0.50 – 1.00	0.000435
1.00 – 2.00	0.000299
2.00 – 4.00	0.000205
4.00 – 8.00	0.000137

Stratum IV Stiff / very stiff bluish grey / grey silty clay with traces of rusty brown silt spots & kankars. (absent in B.H.10).

The extensions of this layer in the Boreholes are as follows: -

Borehole Mark	Measured with Respect to E.G.L. (m)	
	From	To
B.H.5	12.00	17.00
B.H.7	14.70	18.50
B.H.8	16.00	19.20
B.H.12	13.60	18.30

This layer contains stiff / very stiff bluish grey / grey silty clay with traces of rusty brown silt spots & kankars. Field 'N' value of this layer ranges from 09 to 19, indicating its stiff / very stiff state of consistency. The relevant engineering properties are mentioned herewith:

Field 'N' value	= 09 to 19
Bulk Density	= 18.43 kN/m^3 to 19.56 kN/m^3
Dry Density	= 13.38 kN/m^3 to 15.61 kN/m^3
Natural Moisture Content	= 23.74 % to 32.86 %
Liquid Limit	= 62 % to 68 %
Plastic Limit	= 20 % to 24 %
Undrained Cohesion 'C'	= 42.36 kN/m^2 to 75.10 kN/m^2
	(From U.C.S. Test)

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Undrained Cohesion ' C_u ' = 35.69 kN/m² to 96.11 kN/m²
(From Triaxial U.U. Test)

Undrained angle of shear resistance ' ϕ_u ' = 0°
(From Triaxial U.U. Test)

Specific Gravity = 2.65 to 2.72

Initial void ratio ' e_0 ' = 0.645 to 0.870

Grain Size Analysis (Hydrometer Analysis):

Gravel = 03 % to 06 %

Sand = 05 % to 19 %

Silt = 34 % to 53 %

Clay = 40 % to 44 %

I.S. Classification = CH

The following are the average coefficient of volume compressibility (m_v) values of this stratum:

Pressure Range in kg/cm ²	m_v (m ² /kN)
0.00 – 0.25	0.000435
0.25 – 0.50	0.000363
0.50 – 1.00	0.000268
1.00 – 2.00	0.000184
2.00 – 4.00	0.000120
4.00 – 8.00	0.000079

Stratum IVA Stiff / very stiff mottled brown silty sandy clay with traces of mica. (Present in B.H.5 & B.H.10).

The extensions of this layer in the Boreholes are as follows: -

Borehole Mark	Measured with Respect to E.G.L. (m)	
	From	To
B.H.5	17.20	20.00
B.H.10	14.60	19.50

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This layer contains stiff / very stiff mottled brown silty sandy clay with traces of mica. It is a c- ϕ soil layer. Field 'N' value of this layer ranges from 14 to 19, indicating its stiff / very stiff state of consistency. The relevant engineering properties are mentioned herewith:

Field 'N' value	= 14 to 19
Bulk Density	= 19.32 kN/m ³ to 19.82 kN/m ³
Dry Density	= 15.32 kN/m ³ to 16.1207 kN/m ³
Natural Moisture Content	= 23.31 % to 26.05 %
Liquid Limit	= 51 % to 54 %
Plastic Limit	= 19 % to 20 %
Undrained Cohesion 'C _u '	= 73.44 kN/m ² to 90.04 kN/m ² (From Triaxial U.U. Test)
Undrained angle of shear resistance ' ϕ_u '	= 10° (From Triaxial U.U. Test)
Specific Gravity	= 2.71 to 2.72
Initial void ratio 'e ₀ '	= 0.634 to 0.705
Grain Size Analysis (Hydrometer Analysis):	
Sand	= 38 % to 39 %
Silt	= 19 % to 22 %
Clay	= 40 % to 42 %
I.S. Classification	= SC

The following are the average coefficient of volume compressibility (m_v) values of this stratum:

Pressure Range in kg/cm ²	m _v (m ² /kN)
0.00 – 0.25	0.000419
0.25 – 0.50	0.000335
0.50 – 1.00	0.000259
1.00 – 2.00	0.000177
2.00 – 4.00	0.000118
4.00 – 8.00	0.000079

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Stratum V *Medium compact brown silty clayey sand /silty fine sand with traces of mica & clay as binder.*

The extensions of this layer in the Boreholes are as follows: -

Borehole Mark	Measured with Respect to E.G.L. (m)	
	From	To
B.H.5	20.00	**20.45
B.H.7	18.50	**20.05
B.H.8	19.20	**20.45
B.H.10	19.50	**20.95
B.H.12	18.30	**20.05

****Termination depths of the boreholes.**

This layer contains medium compact brown silty clayey sand /silty fine sand with traces of mica & clay as binder. Field 'N' value of this layer ranges from 18 to 27, indicating its very medium degree of compactness. The relevant engineering properties are mentioned herewith, which may not totally representative of the entire stratum:

<i>Field 'N' value</i>	= 18 to 27
<i>Bulk Density</i>	= 18.45 kN/m ³
<i>Dry Density</i>	= 14.47 kN/m ³
<i>Natural Moisture Content</i>	= 27.49 %
<i>Liquid Limit</i>	= N.P
<i>Plastic Limit</i>	= N.P
<i>Grain Size Analysis (Hydrometer Analysis):</i>	
<i>Sand</i>	= 65 %
<i>Silt</i>	= 09 %
<i>Clay</i>	= 26 %
<i>I.S. Classification</i>	= SC - SM



TABLE - 2
AVERAGE SUB-SOIL PROFILE

<i>Stratum</i>	<i>Description</i>	<i>Average Thickness</i>	<i>Range of Field 'N' value</i>	<i>IS Classification</i>
<i>I</i>	<i>Reclaimed top fill of soft grey silty clay with traces of grass roots and tree plants.</i>	<i>0.95 m.</i>	<i>03</i>	<i>CH</i>
<i>II</i>	<i>Soft / firm brownish grey / grey silty clay / clayey silt with traces of kankars and rusty brown silt spots. (Absent in B.H.5, B.H.6 & B.H.8 only).</i>	<i>1.35 m.</i>	<i>03</i>	<i>CH - MH</i>
<i>III</i>	<i>Very soft / soft grey silty clay with varying percentage of decomposed wood. (traces to medium to high percentage).</i>	<i>11.86 m. & more than 9.75 m., 6.55 m., 7.15 m. in B.H.6, B.H.9, B.H.11 respectively.</i>	<i>02 to 03</i>	<i>CH - OH</i>
<i>IV</i>	<i>Stiff / very stiff bluish grey / grey silty clay with traces of rusty brown silt spots & kankars. (absent in B.H.10)</i>	<i>5.30 m.</i>	<i>09 to 19</i>	<i>CH</i>
<i>IV A</i>	<i>Stiff / very stiff mottled brown silty sandy clay with traces of mica. (Present in B.H.5, & B.H.10).</i>	<i>1.58 m.</i>	<i>14 to 19</i>	<i>SC</i>

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TABLE - 2
AVERAGE SUB-SOIL PROFILE

<i>Stratum</i>	<i>Description</i>	<i>Average Thickness</i>	<i>Range of Field 'N' value</i>	<i>IS Classification</i>
<i>V</i>	<i>Medium compact brown silty clayey sand /silty fine sand with traces of mica & clay as binder.</i>	<i>More than 0.45 m., 1.55 m., 1.25 m., 1.45 m., 1.75 m. in B.H.5, B.H.7, B.H.8, B.H.10, B.H.12 respectively</i>	<i>18 to 27</i>	<i>SC - SM</i>



6. BEARING CAPACITY OF SHALLOW, RIGID RAFT & DEEP FOUNDATIONS

6.1.1 Computation of net safe bearing capacities for shallow foundations

For computation of net safe bearing capacity values for design of shallow foundations, ultimate net bearing capacity formula as per "Code of Practice for Determination of Bearing Capacity of Shallow Foundations (First Revision)", IS: 6403 (1981) will be used.

As per the clause 5.1.2 of IS: 6403 (1981), the ultimate net bearing capacity of shallow foundations in case of general shear failure,

$$q_d = C_u N_c S_c d_c i_c + q(N_q - 1) S_q d_q i_q + 0.5 B \gamma N_\gamma d_\gamma S_\gamma i_\gamma w'$$

and the same in case of local shear failure,

$$q_d = 2/3 C_u N_c' S_c d_c i_c + q(N_q' - 1) S_q d_q i_q + 0.5 B \gamma N_\gamma' d_\gamma S_\gamma i_\gamma w'$$

where, N_c , N_q , N_γ and N_c' , N_q' , N_γ' are the bearing capacity factors in case of general shear and local shear failure conditions respectively.

S_c , S_q and S_γ are the shape factors.

d_c , d_q and d_γ are the depth factors.

i_c , i_q and i_γ are the inclination factors.

B is the least dimension of the foundation in metre.

q is the effective surcharge at the base level of foundation in kN/m^2 .

γ is the bulk unit weight of foundation soil in kN/m^3 .

w' is the correction factor for location of water table.

In case of foundations resting on cohesive soils, as the rate of gradual building up of load intensity at the foundation level will be more than the rate of dissipation of excess pore water pressure from the cohesive soil due to low to very low range of co-efficient of permeability values of the same, the computation of net safe bearing capacity values for foundations resting on cohesive foundation medium will be governed by undrained analysis, i.e. $\phi = 0^\circ$ analysis will prevail.

Computation of net safe bearing capacity values for shallow foundations resting on cohesive stratum, at 1.50 m. depth below Avg. E.G.L. (During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12) the following formula will be used.

$$q_{ns} = q_d / F.O.S. = C_u N_c S_c d_c i_c / F.O.S.$$



6.1.2 Foundation Settlement

Settlement is calculated on the basis of the following formulae

$$\text{Immediate Settlement (S}_i\text{)} = q_{ns} B (1 - \mu^2) I_f / E$$

Where, q_{ns} = Net foundation pressure.

B = Width of footing.

μ = Poisson's Ratio of soil.

E = Modulus of Elasticity.

I_f = Influence Factor.

$$\text{Corrected immediate settlement, } S_{ic} = S_i C_c$$

Where, C_c = Depth correction factor as per FOX.

Consolidation settlement

Assuming the influence zone for 20% stress contour, upto twice the width of footing, the total consolidation settlement is given by

$$S_c = \Sigma m_v \Delta P H \quad \text{as per IS: 8009 (Part I)}$$

Where, m_v = Coefficient of volume change.

ΔP = Pressure increment at centre of the layer.

H = Thickness of the stratum.

$$\text{Corrected consolidation settlement, } S_{cc} = S_c C_b C_c$$

Where, C_b = Pore Pressure correction factor.

C_c = Depth correction factor as per FOX.



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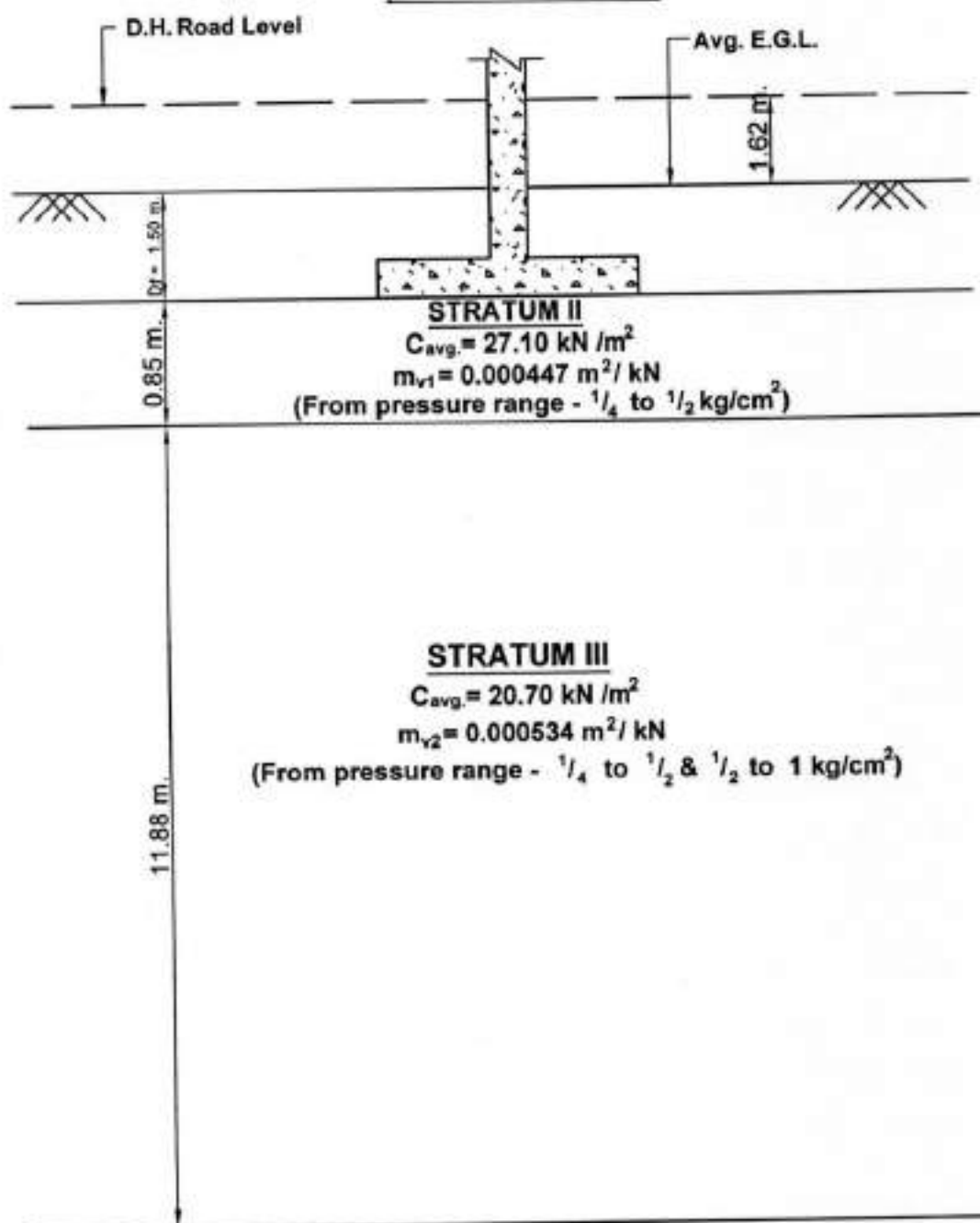
FOUNDATION DESIGN MODEL FOR SHALLOW FOUNDATION

DEPTH OF FOUNDATION = 1.50 M. BELOW AVG. E.G.L.

(During the fieldwork E.G.L. was about 1.62 m. below D.H. Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

(DRAWING NOT TO SCALE)





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6.1.3 Design Consideration for Shallow Footings resting on cohesive stratum at $D_f = 1.50$ m. depth below Avg. E.G.L. (During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

In this case, major portion of bearing capacity failure surface lies in stratum II & III. Hence the weighted avg. value of $C_u = \{[(C_1 \times d_1) + (C_2 \times d_2)] / (d_1 + d_2)\} = \{[(27.10 \times 0.85) + (20.70 \times 1.978)] / (0.85 + 1.978)\} = 22.62 \text{ kN/m}^2$, considering the weighted avg. value of C_u of stratum II ($C_1 = 27.10 \text{ kN/m}^2$) & upper reaches value of C_u of stratum III ($C_2 = 20.70 \text{ kN/m}^2$), have been used in bearing capacity equation.

$C_u = 22.62 \text{ kN/m}^2$ (Considering the weighted avg. value of C_u of stratum II & upper reaches value of C_u of stratum III)

$\phi_u = 0^\circ$ (considering worst condition of G.W. Table during Monsoons)

$E = 600 C_u = 600 \times 22.62 = 13572 \text{ kN/m}^2$ (using the weighted avg. value of C_u of stratum II & upper reaches value of C_u of stratum III)

$\mu = 0.50$ (for clay)

$m_{v1} = 0.000447 \text{ m}^2/\text{kN}$ (for relevant pressure range $-\frac{1}{4}$ to $\frac{1}{2} \text{ kg/cm}^2$)

$m_{v2} = 0.000534 \text{ m}^2/\text{kN}$ (for relevant pressure range $-\frac{1}{4}$ to $\frac{1}{2}$ & $\frac{1}{2}$ to 1 kg/cm^2)

Pore Pressure Correction Factor = 0.80 considered for N.C. Clay.

Depth correction factor, depending on D_f/B and μ

The bearing capacity and probable settlement values of shallow footings are tabulated in Table 3(A).

6.1.4 Sample Calculations of bearing capacity and probable settlement of shallow footings for $D_f = 1.50$ m. depth below Avg. E.G.L. (During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Considering the weighted avg. value of $C_u = 22.62 \text{ kN/m}^2$ where major portion of bearing capacity failure surface lies in stratum II & III.

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Example for 2.00 m x 2.00 m square footing at 1.50 m. depth below Avg. E.G.L. (During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level)(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Properties of founding stratum:

$$C = 22.62 \text{ kN/m}^2 \text{ (using the weighted avg. value of } C_u \text{ of stratum II \& upper reaches value of } C_u \text{ of stratum III)}$$

$$N_c = 5.14 \text{ (for } \phi = 0^\circ)$$

$$S_c = 1.30 \text{ (for square footing)}$$

$$d_c = 1.15 \text{ (for 2.00 m wide square footing)}$$

$$i_c = 1.00 \text{ (for vertical load)}$$

$$\text{Net ultimate bearing capacity, } q_d = 22.62 \times 5.14 \times 1.30 \times 1.15 \times 1.00 = 173.81 \text{ kN/m}^2$$

Taking a factor of safety = 2.50,

$$\text{Net safe bearing capacity, } q_{ns} = 173.81 / 2.50 = 69.52 \text{ kN/m}^2 \approx 69.50 \text{ kN/m}^2$$

Settlement

$$\text{Immediate settlement (S}_i\text{)} = q_{ns} B (1 - \mu^2) I_f / E$$

$$q_{ns} = 69.50 \text{ kN/m}^2$$

$$B = 2.00 \text{ m}$$

$$\mu = 0.50 \text{ (for clay)}$$

$$I_f = 1.12 \text{ (for square footing)}$$

$$E = 13572 \text{ kN/m}^2$$

$$\text{So, } S_i = \{69.50 \times 2.00 \times (1 - 0.50^2) \times 1.12 \times 10^3\} / 13572 = 8.60 \text{ mm.}$$

After applying Depth correction factor of 0.773 and Rigidity correction factor being not applied for flexible footings,

$$S_i \text{ corrected} = 8.60 \times 0.773 = 6.65 \text{ mm.}$$

$$\text{Consolidation settlement (S}_c\text{)} = \Sigma n_v \Delta PH$$

$$m_{v1} = 0.000447 \text{ m}^2/\text{kN} \text{ (for relevant pressure range)}$$

$$\Delta P_1 = 47.27 \text{ kN/m}^2 \text{ (assuming 2 : 1 dispersion)}$$

$$H_1 = 0.85 \text{ m.}$$

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$$m_{v2} = 0.000534 \text{ m}^2/\text{kN} \text{ (for relevant pressure range)}$$

$$\Delta P_2 = 14.19 \text{ kN/m}^2 \text{ (assuming 2 : 1 dispersion)}$$

$$H_2 = 3.15 \text{ m.}$$

$$\begin{aligned} S_o, S_c &= \{(0.000447 \times 47.27 \times 0.85) + (0.000534 \times 14.19 \times 3.15)\} \times 10^3 \\ &= 41.82 \text{ mm.} \end{aligned}$$

Applying a pore pressure correction factor of 0.80 and depth correction factor of 0.773,

$$S_c \text{ corrected} = 41.82 \times 0.80 \times 0.773 = 25.86 \text{ mm.}$$

$$\therefore \text{Total corrected settlement, } S = (6.65 + 25.86) \text{ mm} = 32.51 \text{ mm, say } 33.00 \text{ mm.}$$

Similarly, the bearing capacity-probable settlement values of other sizes and types of shallow foundations, at 1.50 m. depth below Avg. E.G.L. (During the period of field work Avg. E.G.L. was 1.62 m. below D. H. Road Level)(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12) are calculated and provided in Table 3(A) for foundation design purpose.



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Table 3(A)

Shallow Foundation: Bearing Capacity & Probable Settlement Values

Depth of foundation = 1.50 m. below Avg. E.G.L.

(During the period of field work Avg. E.G.L. was 1.62 m. below D. H. Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Factor of Safety in shear = 2.50

Type of footing	Size of footing (m x m)	Net safe bearing capacity in shear (kN/m ²)	Total expected settlement (mm)	Net allowable bearing pressure to restrict settlement within *75.00 mm (i.e. safe both in shear and settlement) (kN/m ²)
ISOLATED SQUARE	1.50 x 1.50	72.60	24.00	72.60
	2.00 x 2.00	69.50	33.00	69.50
	2.50 x 2.50	67.70	42.00	67.70
	3.00 x 3.00	66.50	51.00	66.50
	3.50 x 3.50	65.60	58.00	65.60
	4.00 x 4.00	65.00	65.00	65.00
RECTANGULAR	1.50 x 3.00	61.40	28.00	61.40
	2.00 x 4.00	58.80	38.00	58.80
	2.50 x 5.00	57.30	48.00	57.30
	3.00 x 6.00	56.30	58.00	56.30
	3.50 x 7.00	55.50	66.00	55.50
	4.00 x 8.00	55.00	75.00	55.00
SINGLE DIRECTION CONTINUOUS STRIP	1.00 m wide	60.50	23.00	60.50**
	1.50 m. wide	55.80	34.00	55.80**
	2.00 m. wide	53.50	45.00	53.50**
	2.50 m. wide	52.10	57.00	52.10**
	3.00 m. wide	51.20	68.00	51.20**
	3.50 m wide	50.50	79.00	47.90**

* As per Table - 1 of I.S.1904 (1986) for isolated foundation for Multistoried Buildings in plastic clay, like in present case.

** Bothways interconnecting strip footing, if used, should not be designed with these values of bearing capacity.

*If earth filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $18.00 \text{ kN/m}^3 \times 1.20 \text{ m} \approx 21.60 \text{ kN/m}^2$ (considering unit weight of earth fill to be 18.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 21.60 kN/m^2 from the bearing capacity of Table 3(A) during foundation design.

• Similarly, if fly ash filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $14.00 \text{ kN/m}^3 \times 1.20 \text{ m} \approx 16.80 \text{ kN/m}^2$ (considering unit weight of compact fly ash fill to be 14.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 16.80 kN/m^2 from the bearing capacity of Table 3(A) during foundation design.



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FOUNDATION DESIGN MODEL

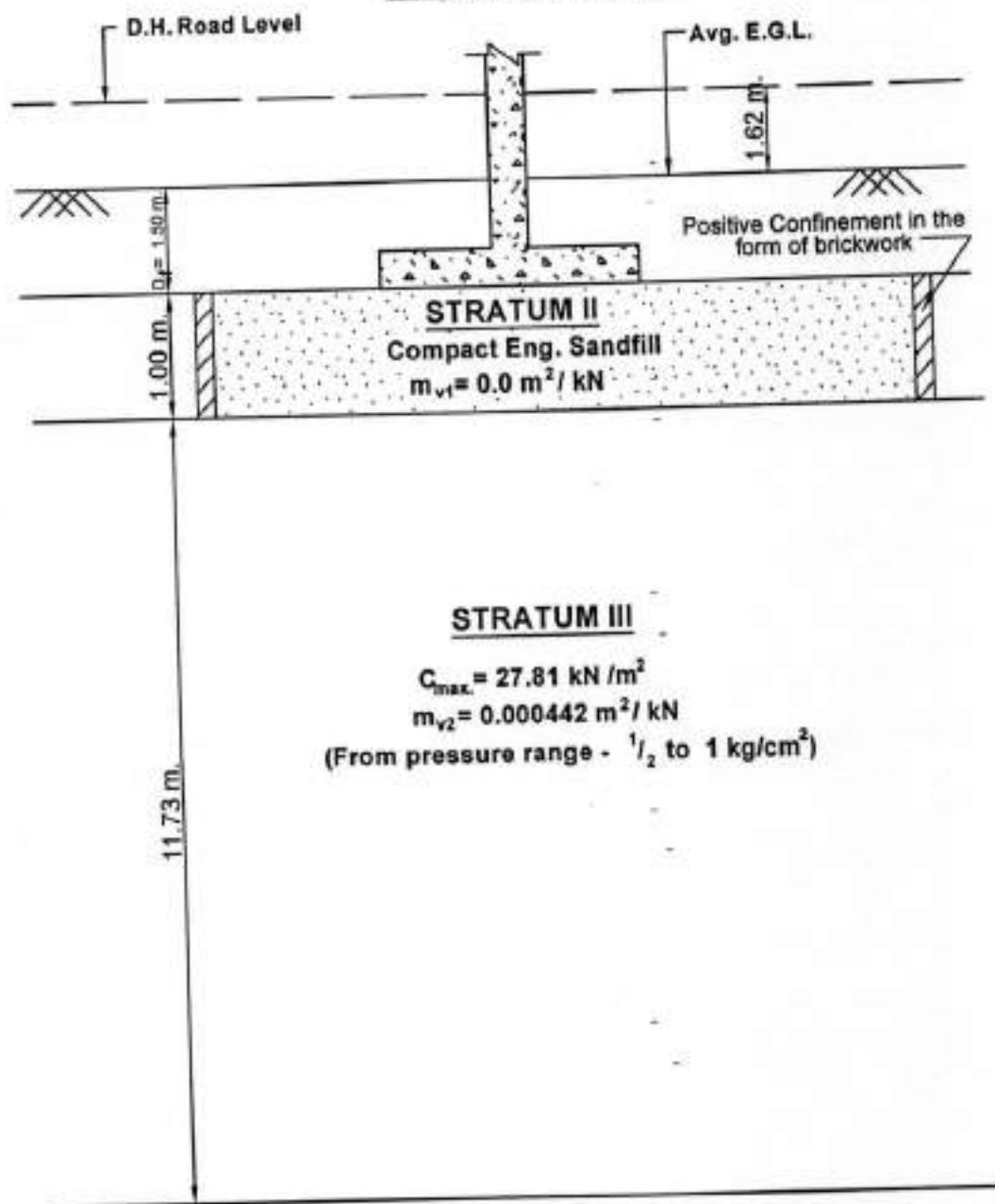
For Shallow Foundation resting on 1.00 m thick Cohesionless Compact Engineering Sand fill Layer (with positive confinement)

DEPTH OF FOUNDATION = 1.50 M. BELOW AVG. E.G.L.

(During the fieldwork E.G.L. was about 1.62 m. below D.H. Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

(DRAWING NOT TO SCALE)





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Table 3(B)

Bearing Capacity and Probable Settlement Values of Shallow Foundation resting on 1.00 m thick Cohesionless Compact Engineering Sand fill Layer (with positive confinement)

Depth of foundation = 1.50 m below Avg. E.G.L

(During fieldwork Avg. E.G.L. was 1.62 m. below D. H. Road Level)

Factor of Safety in shear = 2.50

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Type of footing	Size of footing (m x m)	Net safe bearing capacity in shear (kN/m ²)	Total expected settlement (mm)	Net allowable bearing pressure to restrict settlement within *60.00 mm (i.e. safe both in shear and settlement) (kN/m ²)	Recommended Net allowable bearing pressure for design (kN/m ²)
ISOLATED SQUARE	1.50 x 1.50	90.40	14.00	90.40	85.00
	2.00 x 2.00	80.80	20.00	80.80	80.00
	2.50 x 2.50	76.40	26.00	76.40	75.00
	3.00 x 3.00	74.10	33.00	74.10	74.00
	3.50 x 3.50	72.60	39.00	72.60	72.00
	4.00 x 4.00	71.70	46.00	71.70	70.00
RECTANGULAR	1.50 x 3.00	79.20	18.00	79.20	75.00
	2.00 x 4.00	72.10	26.00	72.10	70.00
	2.50 x 5.00	68.70	35.00	68.70	68.00
	3.00 x 6.00	67.00	44.00	67.00	67.00
	3.50 x 7.00	65.90	52.00	65.90	65.00
	4.00 x 8.00	65.20	60.00	65.20	64.00
SINGLE DIRECTION CONTINUOUS STRIP	1.00 m. wide	81.50	14.00	**81.50	**70.00
	1.50 m. wide	68.10	23.00	**68.10	**65.00
	2.00 m. wide	63.30	33.00	**63.30	**63.00
	2.50 m. wide	61.10	44.00	**61.10	**60.00
	3.00 m. wide	59.90	54.00	**59.90	**58.00
	3.50 m. wide	59.20	65.00	**54.60	**54.00

* As per Table - 1 of I.S.1904 (1986) for isolated foundation for Multistoried Buildings in sand or hard clay, like in present case.

** Bothways interconnecting strip footing, if used, should not be designed with these values of bearing capacity.

*If earth filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $18.00 \text{ kN/m}^2 \times 1.20 \text{ m} \approx 21.60 \text{ kN/m}^2$ (considering unit weight of earth fill to be 18.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 21.60 kN/m^2 from the bearing capacity of Table 3(A) during foundation design.

*Similarly, if fly ash filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $14.00 \text{ kN/m}^2 \times 1.20 \text{ m} \approx 16.80 \text{ kN/m}^2$ (considering unit weight of compact fly ash fill to be 14.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 16.80 kN/m^2 from the bearing capacity of Table 3(A) during foundation design.

Note : - At any cost, silver sand should not be used.



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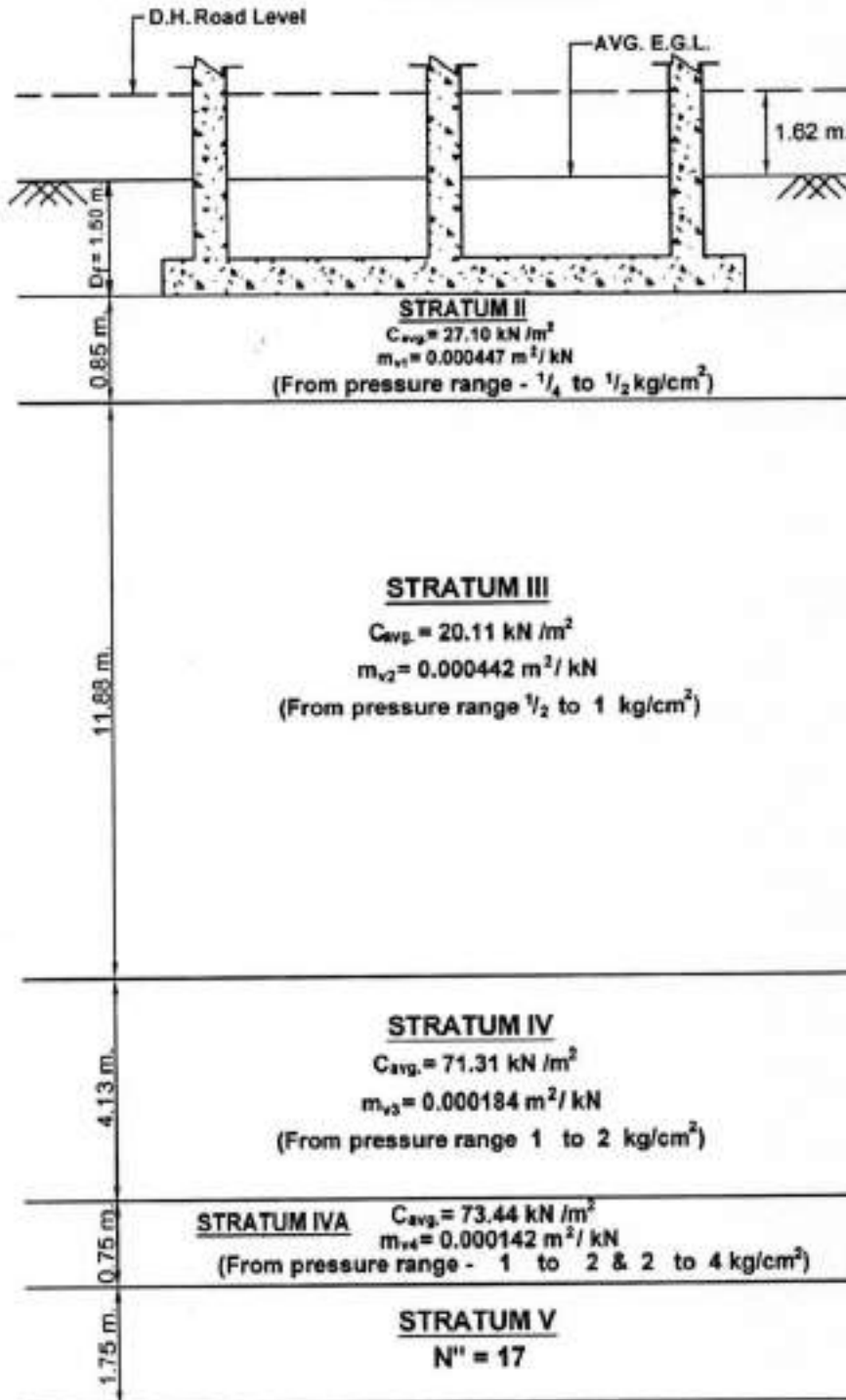
FOUNDATION DESIGN MODEL FOR SHALLOW FOUNDATION AS RIGID RAFT

DEPTH OF FOUNDATION = 1.50 M. BELOW AVG. E.G.L.

(During the fieldwork E.G.L. was about 1.62 m. below D.H.Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

(DRAWING NOT TO SCALE)





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6.2.1 Sample Calculations of 16.00 m x 21.00 m bearing capacity and probable settlement of shallow footings for $D_f = 1.50$ m. below Avg. E.G.L. (During the period of field work Avg. E.G.L. was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Given Data:

Size of foundation = 16.00 m x 21.00 m (tentative)

Depth of foundation = 1.50 m below Avg. E.G.L. (During the period of field work Avg. E.G.L. was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Depth of water table = 0.00 m (assuming the worst condition of G. W. Table during Monsoons)

Shear Criteria:

Properties of founding stratum:

$$C_{(weighted)} = \frac{[(C_1 \times d_1) + (C_2 \times d_2)]}{(d_1 + d_2)} = \frac{[(27.10 \times 0.85) + (20.11 \times 10.462)]}{(0.85 + 10.462)} = 20.63 \text{ kN/m}^2 \approx 21.00 \text{ kN/m}^2$$

[Considering the avg. weighted value of C_u of stratum II ($C_1 = 27.10 \text{ kN/m}^2$) & stratum III ($C_2 = 20.11 \text{ kN/m}^2$), $d_1 = 0.85 \text{ m}$ & $d_2 = 10.462$]

$$N_c = 5.14 \text{ (for } \phi = 0^\circ \text{)}$$

$$S_c = 1 + 0.2 (B / L) = 1.15 \text{ (for 16 m wide and 21 m long rectangular footing)}$$

$$d_c = 1 + 0.2 (D_f / B) = 1.02 \text{ (for rigid raft)}$$

$$i_c = 1.00 \text{ (for vertical load)}$$

$$\text{Net ultimate bearing capacity, } q_d = 21.00 \times 5.14 \times 1.15 \times 1.02 \times 1.00 = 126.61 \text{ kN/m}^2$$

Taking a factor of safety = 3.00,

$$\text{Net safe bearing capacity, } q_{ns} = 126.61 / 3.00 = 42.20 \text{ kN/m}^2$$

Settlement Criteria:

$$\text{Immediate settlement } (S_i) = q_{ns} B (1 - \mu^2) I_f / E$$

$$q_{ns} = 42.20 \text{ kN/m}^2$$

$$B = 16.00 \text{ m (width of rigid raft)}$$

$$\mu = 0.50 \text{ (for clay)}$$

$$I_f = 1.53 \text{ (for rectangular footing)}$$



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$$E_{\text{weighted}} = \{(16260 \times 0.85) + (10055 \times 11.88) + (53482.50 \times 3.27) / (0.85 + 11.88 + 3.27)\}$$

$$= 19260.13 \text{ kN/m}^2 \text{ say } 19260.00 \text{ kN/m}^2$$

[Where, $d_1 = 0.85 \text{ m}$, $d_2 = 11.88 \text{ m}$, & $d_3 = 3.27 \text{ m}$, and

$E_1 = 600 C_u$ (for clay) = $600 \times 27.10 \text{ kN/m}^2 = 16260 \text{ kN/m}^2$ (From V.N.S.

Murthy, Chapter – Shallow Foundation; Sub-Chapter – Foundation Settlement;

Clause-18.20) (using average value of C_u of stratum II = 27.10 kN/m^2).

$E_2 = 500 C_u$ (for clay) = $500 \times 20.11 \text{ kN/m}^2 = 10055 \text{ kN/m}^2$ (From V.N.S.

Murthy, Chapter – Shallow Foundation; Sub-Chapter – Foundation Settlement;

Clause-18.20) (using average value of C_u of stratum III = 20.11 kN/m^2).

$E_3 = 750 C_u$ (for clay) = $750 \times 71.31 \text{ kN/m}^2 = 53482.50 \text{ kN/m}^2$ (From V.N.S.

Murthy, Chapter – Shallow Foundation; Sub-Chapter – Foundation Settlement;

Clause-18.20) (using average value of C_u of stratum IV = 71.31 kN/m^2).

So, $S_i = \{42.20 \times 16.00 \times (1 - 0.50^2) \times 1.53 \times 10^3\} / 19260 = 40.23 \text{ mm}$.

After applying Rigidity correction factor of 0.80 and Depth Correction Factor of 0.888

$$S_i \text{ corrected} = 40.23 \times 0.80 \times 0.888 = 28.58 \text{ mm}.$$

Consolidation Settlement (S_c):

$$\text{Consolidation settlement, } S_c = \sum m_v \Delta P H$$

Where,

$$m_{v1} = 0.000447 \text{ m}^2/\text{kN} \text{ (for relevant pressure range)}$$

$$\Delta P_1 = 40.33 \text{ kN/m}^2 \text{ (assuming 2: 1 dispersion)}$$

$$H_1 = 0.85 \text{ m}.$$

$$m_{v2} = 0.000442 \text{ m}^2/\text{kN} \text{ (for relevant pressure range)}$$

$$\Delta P_2 = 22.40 \text{ kN/m}^2 \text{ (assuming 2: 1 dispersion)}$$

$$H_2 = 11.88 \text{ m}.$$

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$$m_{v3} = 0.000184 \text{ m}^2/\text{kN} \text{ (for relevant pressure range)}$$

$$\Delta P_3 = 12.88 \text{ kN/m}^2 \text{ (assuming 2: 1 dispersion)}$$

$$H_3 = 4.13 \text{ m.}$$

$$m_{v4} = 0.000142 \text{ m}^2/\text{kN} \text{ (for relevant pressure range)}$$

$$\Delta P_4 = 11.17 \text{ kN/m}^2 \text{ (assuming 2: 1 dispersion)}$$

$$H_4 = 0.75 \text{ m.}$$

$$\begin{aligned} S_{cl} &= \{(0.000447 \times 40.33 \times 0.85) + (0.000442 \times 22.40 \times 11.88) + (0.000184 \times 12.88 \times 4.13) + \\ &\quad (0.000142 \times 11.17 \times 0.75)\} \times 10^3 \\ &= 143.91 \text{ mm.} \end{aligned}$$

Consolidation Settlement of Sand Layer (V):

To determine the value of phi, following 'N' Values have been considered: -

B.H. No.	Depth (m)	N - Value	N' - Value	N'' - Value
7.	19.60 - 20.05	18.00	15.30	15.15
8.	20.00 - 20.45	25.00	21.25	18.13
12.	19.60 - 20.05	27.00	22.95	18.98

Avg. of corrected value of N, N'' = 17.42 $\approx$ 17.00

Corrected (due to overburden pressure and dilatancy) average 'N' of stratum V = 17.00

$\therefore$ The settlement for 100.00 kN/m² loading intensity = 13.00 mm

(Corresponding to $N_{corr.} = 17.00$ and $B = 16.00$ m from fig. 9 of IS: 8009, Part 1)

$\therefore$ The settlement for 42.60 kN/m² loading intensity = $(13.00 / 100.00) \times 42.20 = 5.49$ mm

Water table correction = 0.50

(for $d/B = 0$, considering the worst condition, i.e. water table at E.G.L)

Applying water table correction, the settlement works out to be $5.49 / 0.50 = 10.98$ mm says 11.00 mm.

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ΔP at the top portion of the stratum $V = q_{ns} B \times L / \{(B+Z)(L+Z)\}$

$$= (42.20 \times 16.00 \times 21.00) / \{(16.00 + 17.61) \times (21.00 + 17.61)\}$$
$$= 10.93 \text{ kN/m}^2$$

ΔP at the bottom portion of stratum $V = q_{ns} B \times L / \{(B+Z)(L+Z)\}$

$$= (42.20 \times 16.00 \times 21.00) / \{(16.00 + 19.36) \times (21.00 + 19.36)\}$$
$$= 9.94 \text{ kN/m}^2$$

Settlement in 19.36 m. length of cohesionless soil is computed by interpolation.

Thus settlement at top of stratum V (i.e. at 17.61 m. depth from founding level)

$$= (11.00 \times 1.75 \times 10.93) / (19.36 \times 42.20) = 0.26 \text{ mm.}$$

and settlement at bottom of stratum V (i.e. at 19.36 m. depth from founding level)

$$= (11.00 \times 1.75 \times 9.94) / (19.36 \times 42.20) = 0.23 \text{ mm.}$$

$\therefore$ Settlement in 1.75 m. thick cohesionless soil, $S_{c2} = (0.26 - 0.23) = 0.03 \text{ mm.}$

$\therefore$ Total consolidation settlement, $S_c = (S_{c1} + S_{c2})$

$$= (143.91 + 0.03) \text{ mm.} = 143.94 \text{ mm.}$$

Applying a Rigidity correction factor of 0.80, Depth Correction Factor of 0.888 and pore pressure correction factor of 0.80,

$$S_c \text{ corrected} = 143.94 \times 0.80 \times 0.888 \times 0.80 = 81.80 \text{ mm.}$$

$\therefore$ Corrected total settlement, $S = (28.58 + 81.80) = 110.38 \text{ mm, say } 110.00 \text{ mm.}$

$\therefore$ Allowable bearing capacity for 125 mm allowable settlement = 47.95 kN/m^2 say 48.00 kN/m^2

Table 3(C) gives the net allowable bearing pressure at 1.50 m. below Avg. E.G.L. (During the period of field work Avg. E.G.L. was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12) (with a factor of safety of 3.00) of the shallow rigid raft of tentative size 16.00 m x 21.00 m, for foundation designing.



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TABLE - 3(C)

Shallow Rigid Raft Foundation / both ways continuous strip footing (Virtual raft) :

Bearing Capacity and Probable Settlement Value

$D_f = 1.50$ m. below Avg. E.G.L.

(During the period of field work Avg. E.G.L. was 1.62 m. below D. H. Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Factor of Safety in shear = 3.00

Type of Footing	Tentative Size (m x m)	Net Safe bearing capacity from shear criterion (kN/m ²)	Estimated Total Settlement corresponding to Net Safe bearing pressure from shear criterion (mm)	Net Allowable bearing pressure for a total settlement of *125.00 mm (kN/m ²)	Design Net Allowable bearing pressure (least among shear and settlement criteria i.e., safe both in shear and settlement) (kN/m ²)
RIGID RAFT/ BOTH WAYS CONTINUOUS STRIP FOOTING (VIRTUAL RAFT)	16.00 x 21.00	42.20	110.00	48.00	42.20

*As per Table - 1 of I.S.1904 (1986) for Raft foundation for Multistoried Buildings in Plastic Clay, like in present case.

*If earth filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $18.00 \text{ kN/m}^3 \times 1.20 \text{ m} \approx 21.60 \text{ kN/m}^2$ (considering unit weight of earth fill to be 18.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 21.60 kN/m^2 from the bearing capacity of Table 3(C) during foundation design.

Similarly, if fly ash filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $14.00 \text{ kN/m}^3 \times 1.20 \text{ m} \approx 16.80 \text{ kN/m}^2$ (considering unit weight of compact fly ash fill to be 14.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 16.80 kN/m^2 from the bearing capacity of Table 3(C) during foundation design.



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FOUNDATION DESIGN MODEL

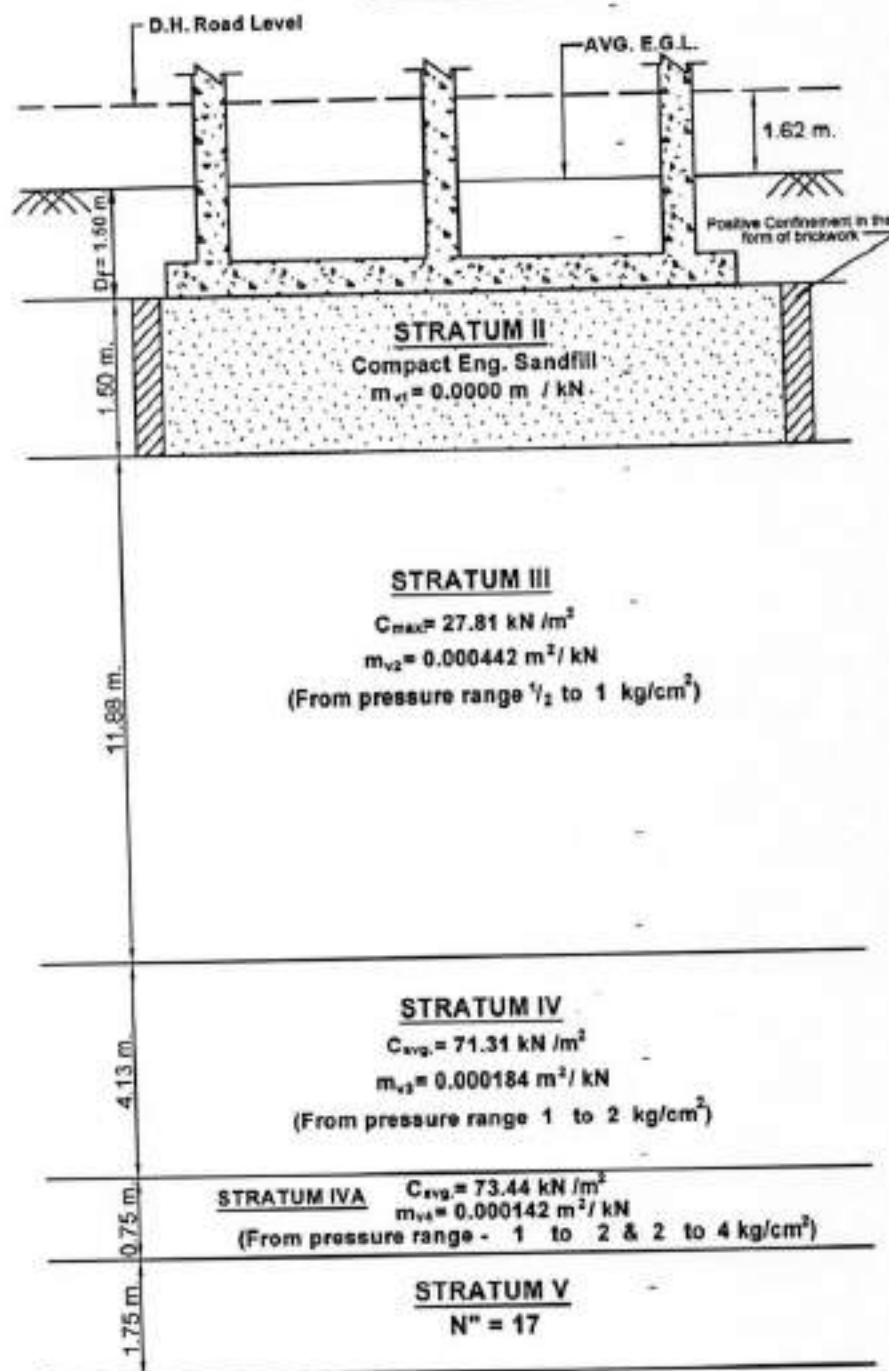
For Shallow Foundation as Rigid Raft resting on 1.50 m thick Cohesionless Compact Engineering Sand fill Layer (with positive confinement)

DEPTH OF FOUNDATION = 1.50 M. BELOW AVG. E.G.L.

(During the fieldwork E.G.L. was about 1.62 m. below D.H. Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

(DRAWING NOT TO SCALE)





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TABLE - 3(D)

Shallow Rigid Raft Foundation / both ways continuous strip footing (Virtual raft) :
Bearing Capacity and Probable Settlement Values resting on 1.50 m thick Cohesionless
Compact Engineering Sand fill Layer (with positive confinement)
 $D_f = 1.50$ m. below Avg. E.G.L.
(During the period of field work Avg. E.G.L. was about 1.62 m. below D. H. Road Level).
Factor of Safety in shear = 3.00
(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Type of Footing	Tentative Size (m x m)	Net Safe bearing capacity from shear criterion (kN/m ²)	Estimated Total Settlement corresponding to Net Safe bearing pressure from shear criterion (mm)	Net Allowable bearing pressure for a total settlement of *125.00 mm (kN/m ²)	Design Net Allowable bearing pressure (least among shear and settlement criteria i.e., safe both in shear and settlement) (kN/m ²)
RIGID RAFT/ BOTH WAYS CONTINUOUS STRIP FOOTING (VIRTUAL RAFT)	16.00 x 21.00	55.20	118.00	58.50	55.20

*As per Table - 1 of I.S.1904 (1986) for Raft foundation for Multistoried Buildings in Plastic Clay, like in present case.

*If earth filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $18.00 \text{ kN/m}^3 \times 1.20 \text{ m} \approx 21.60 \text{ kN/m}^2$ (considering unit weight of earth fill to be 18.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 21.60 kN/m^2 from the bearing capacity of Table 3(D) during foundation design.

Similarly, if fly ash filling is done to raise the E.G.L. to be at par with D.H. Road Level, an extra surcharge pressure intensity = $14.00 \text{ kN/m}^3 \times 1.20 \text{ m} \approx 16.80 \text{ kN/m}^2$ (considering unit weight of compact fly ash fill to be 14.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the super structural load or subtracting 16.80 kN/m^2 from the bearing capacity of Table 3(D) during foundation design.

Note : - At any cost, silver sand should not be used.



6.3 Deep Foundations: R.C. Bored Pile

The load carrying capacities of R.C. Bored Piles, for various dia., are calculated on the basis of I.S. Code of Practice of design and construction of pile foundations, IS: 2911 (Part I / Sec 2) – 2010.

Using the static formula,

For piles in granular soils

$$Q_u = A_p (1/2 D \gamma N_\gamma + P_d N_q) + \sum K P_d \tan \delta A_s$$

Where, A_p = Cross-sectional area of pile toe.

D = Stem diameter.

γ = Effective unit weight of soil at pile toe.

P_d = Effective overburden pressure at pile toe.

A_s = Surface area of pile shaft.

N_γ, N_q = Bearing capacity factors depending on ϕ .

For piles in cohesive soils

$$Q_u = A_p N_c C_p + \sum \alpha C A_s$$

Where, A_p = Cross-sectional area of pile toe.

N_c = Bearing capacity factor = 9.0.

C_p = Average cohesion at pile toe.

α = Reduction factor.

C = Average cohesion throughout the length of pile.

A_s = Surface area of pile shaft.



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FOUNDATION DESIGN MODEL FOR R.C. BORED CAST IN-SITU PILE

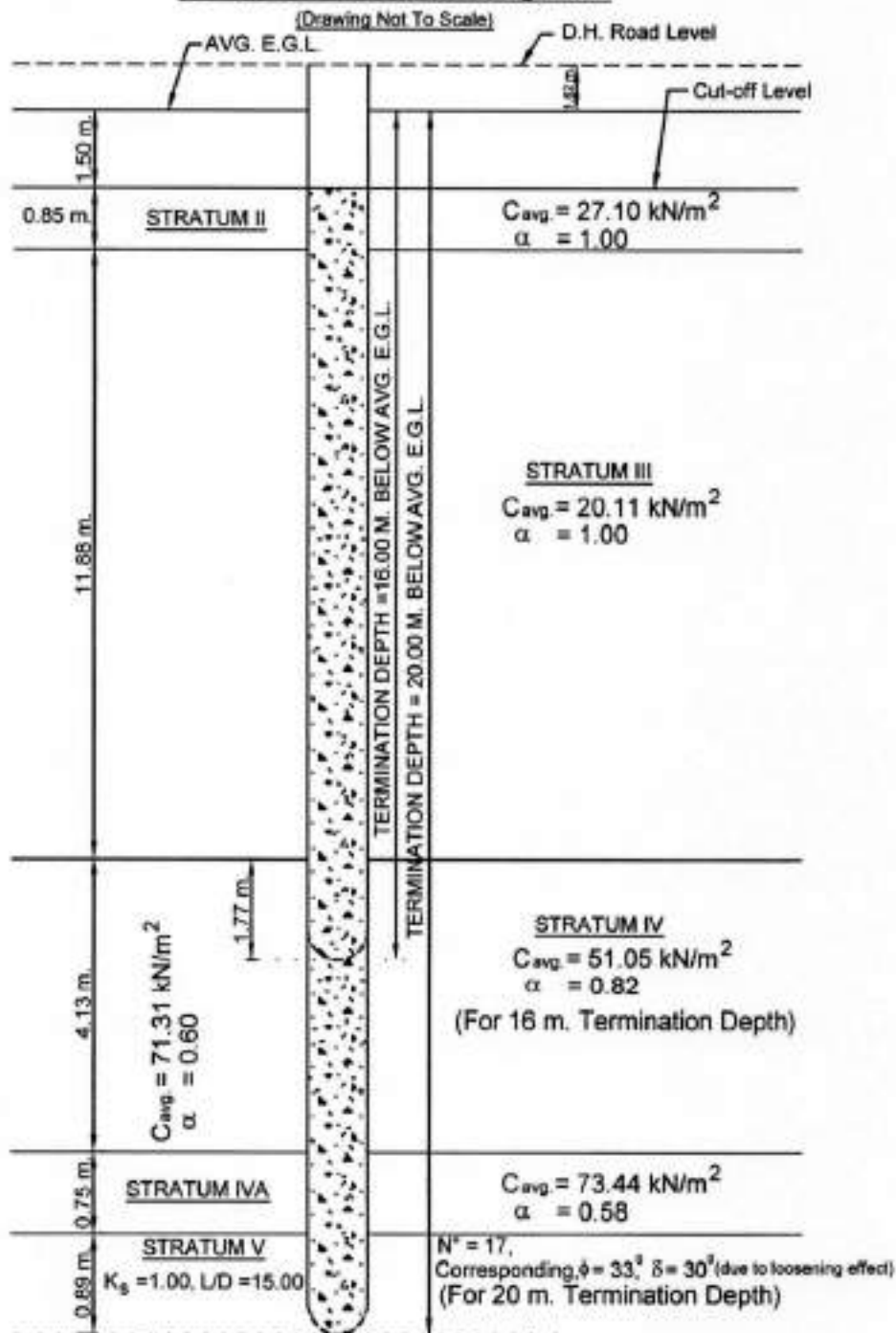
Termination Depth = 16 m. & 20 m. below Avg. E.G.L.

(During the fieldwork E.G.L. was about 1.62 m. below D.H. Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Cut-off Level = 1.50 m. below Avg. E.G.L.

(Drawing Not To Scale)





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6.3.1 Calculations of R.C. Bored pile capacity with termination depth of 20.00 m. below Avg. E.G.L.(During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Termination depth of pile = 20.00 m below Avg. E.G.L.(During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level)

Assumed cut-off level = 1.50 m below Avg. E.G.L.(During the period of field work Avg. E.G.L was about 1.62 m. below D. H. Road Level)

Effective length of pile = (20.00 – 1.50) = 18.50 m

For Skin Friction in cohesionless Stratum V, (Considering L/D = 15.00)

[IS: 2911 (Part I / Sec 2) – 2010, Annex – B: clause 6.3.1.1 and 6.3.2]

$\gamma = 17.00 \text{ kN/m}^3$ is the bulk unit weight of Stratum I foundation soil in kN/m^3 .
(assumed for design purpose)

$\gamma = 18.33 \text{ kN/m}^3$ is the bulk unit weight of Stratum II foundation soil in kN/m^3 .

$\gamma = 16.75 \text{ kN/m}^3$ is the bulk unit weight of Stratum III foundation soil in kN/m^3 .

$\gamma' = (\gamma - 10) = 7.00 \text{ kN/m}^3$ (unit weight of Stratum I in submerged condition)

$\gamma' = (\gamma - 10) = 8.33 \text{ kN/m}^3$ (unit weight of Stratum II in submerged condition)

$\gamma' = (\gamma - 10) = 6.75 \text{ kN/m}^3$ (unit weight of Stratum III in submerged condition)

Depth of water table = 0.00 m (assuming the worst condition of G. W. Table during Monsoons)

$P_d = [(7.00 \times 1.50) + (8.33 \times 0.85) + (6.75 \times 5.90)] = 57.40 \text{ kN/m}^2$ (for 450 ϕ)

$= [(7.00 \times 1.50) + (8.33 \times 0.85) + (6.75 \times 6.65)] = 62.46 \text{ kN/m}^2$ (for 500 ϕ)

$= [(7.00 \times 1.50) + (8.33 \times 0.85) + (6.75 \times 8.15)] = 72.59 \text{ kN/m}^2$ (for 600 ϕ)

For Skin Friction of Stratum V, (Considering L/D = 15.00)

To determine the value of phi, following 'N' Values have been considered: -

B.H. No.	Depth (m)	N - Value	N' - Value	N'' - Value
7.	19.60 – 20.05	18.00	15.30	15.15
12.	19.60 – 20.05	27.00	22.95	18.98



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After conducting the Overburden and Dilatancy correction,

Corrected value of N , $N'' = 17.06 \approx 17.00$ (at pile base level)

Corresponding $\phi = 33^\circ$ from Fig. 1 of IS: 6403

$\therefore \delta = \phi - 3^\circ$ (for bored piles) $= 33^\circ - 3^\circ = 30^\circ$ (due to loosening effect)

$\therefore$ Ultimate skin friction $= \sum \alpha C_u A_s + \sum K_s P_d \tan \delta A_s$ (considering $L/D = 15.00$ and $K_s = 1.00$ for stratum V)

[α (Reduction factor), used in calculation of pile capacity as per Fig - 2, Annex B, clauses 6.3.1.1 & 6.3.2 of IS: 2911 (Part 1/sec- 2)-2010]

$$= \pi D \{(1.00 \times 27.10 \times 0.85) + (1.00 \times 20.11 \times 11.88) + (0.60 \times 71.31 \times 4.13) + (0.58 \times 73.44 \times 0.75) + (1.00 \times 57.40 \times \tan 30^\circ \times 0.89)\} = 1571.07 D \text{ (for } 450 \phi \text{)}$$

$$= \pi D \{(1.00 \times 27.10 \times 0.85) + (1.00 \times 20.11 \times 11.88) + (0.60 \times 71.31 \times 4.13) + (0.58 \times 73.44 \times 0.75) + (1.00 \times 62.46 \times \tan 30^\circ \times 0.89)\} = 1579.24 D \text{ (for } 500 \phi \text{)}$$

$$= \pi D \{(1.00 \times 27.10 \times 0.85) + (1.00 \times 20.11 \times 11.88) + (0.60 \times 71.31 \times 4.13) + (0.58 \times 73.44 \times 0.75) + (1.00 \times 72.59 \times \tan 30^\circ \times 0.89)\} = 1595.59 D \text{ (for } 600 \phi \text{)}$$

End bearing will not mobilize within say upto settlement of about 5% pile dia. In present case, load test will be conducted upto 1.50 times the estimated safe load, upto a maximum of 12 mm. of pile settlement, in case of routine load test.

Even if initial load test is done, as per clause 7.1.5 of I.S. 2911(Part 4): 2013, it is seen that point no. 9) 1) of the above clause yields the lesser value, where also, for 12.00 mm. settlement, no end bearing will mobilise.

Hence end bearing is not considered in pile load capacity calculations. A factor of safety of 1.50 is used in skin friction since we apply 1.50 times the estimated safe load for 12 mm. settlement.



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This has been done to obtain a practical value of pile load capacity. On the contrary if we consider skin friction & end bearing both & use a factor of safety of 2.50, pile capacity will be available on a conservative side, which may have to be ratified after obtaining Load Test Results.

∴ Ultimate pile capacity = Ultimate skin friction, since End Bearing will not mobilize within low strain levels

$$= 1571.07 D \text{ (for 450 } \phi \text{)}$$

$$= 1579.24 D \text{ (for 500 } \phi \text{)}$$

$$= 1595.59 D \text{ (for 600 } \phi \text{)}$$

∴ Safe load carrying capacity

(a factor of safety of 1.50 has been used) = Ultimate pile capacity / 1.50

$$= 1571.07 D / 1.50 = 471.32 \text{ kN, says } 471.00 \text{ kN} = 400.00 \text{ kN Recommended (for 450 } \phi \text{)}$$

$$= 1579.24 D / 1.50 = 526.41 \text{ kN, says } 526.00 \text{ kN} = 450.00 \text{ kN Recommended (for 500 } \phi \text{)}$$

$$= 1595.59 D / 1.50 = 638.23 \text{ kN, says } 638.00 \text{ kN} = 540.00 \text{ kN Recommended (for 600 } \phi \text{)}$$

Thus, the load carrying capacities of piles with 20.00 m. below Avg. E.G.L. (During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12) with a cut off of 1.50 m below Avg. E.G.L. using a factor of safety of 1.50 are presented in Table 3(C), for foundation design purposes.

Similarly, the load carrying capacities of piles with 16.00 m. below Avg. E.G.L. (During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level) (Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12) with a cut off of 1.50 m below Avg. E.G.L. using a factor of safety of 1.50 are presented in Table 3(C), for foundation design purposes.



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6.3.2 Lateral Capacity of Pile

Design Considerations

Considering the avg. 'C' of stratum II = 27.10 kN/m^2

Therefore U.C.S. = 54.20 kN/m^2

∴ From Table 4 of Annexure C (Clause 6.5.2) of I.S. 2911 (Part 1 Sec 2):2010,

$$k_1 = 9.90 \times 10^3 \text{ kN/m}^3$$

∴ From clause c - 2.2,

$$K = (k_1/1.5) \times (0.30 / B)$$

$$= 0.44 \times k_1 \text{ (for 450 } \phi \text{)}$$

$$= 0.40 \times k_1 \text{ (for 500 } \phi \text{)}$$

$$= 0.33 \times k_1 \text{ (for 600 } \phi \text{)}$$

$$\therefore K = 4.36 \times 10^3 \text{ kN/m}^3 \text{ (for 450 } \phi \text{)}$$

$$= 3.96 \times 10^3 \text{ kN/m}^3 \text{ (for 500 } \phi \text{)}$$

$$= 3.27 \times 10^3 \text{ kN/m}^3 \text{ (for 600 } \phi \text{)}$$

Pile is considered to be fixed head and considering Grade of Concrete to be M25.

Soil and Pile Data

$$EI \text{ for 450 } \phi = 5.03 \times 10^4 \text{ kN-m}^2$$

$$EI \text{ for 500 } \phi = 7.67 \times 10^4 \text{ kN-m}^2$$

$$EI \text{ for 600 } \phi = 15.90 \times 10^4 \text{ kN-m}^2$$

$$\text{Now, } R = \sqrt[4]{EI/KB}$$

$$\therefore R \text{ for 450 } \phi = \sqrt[4]{(5.03 \times 10^4) / (4.36 \times 10^3 \times 0.45)} = 2.25 \text{ m.}$$

$$\therefore R \text{ for 500 } \phi = \sqrt[4]{(7.67 \times 10^4) / (3.96 \times 10^3 \times 0.50)} = 2.49 \text{ m.}$$

$$\therefore R \text{ for 600 } \phi = \sqrt[4]{(15.90 \times 10^4) / (3.27 \times 10^3 \times 0.60)} = 3.00 \text{ m.}$$



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Depth of fixity

From Fig. 4 of IS: 2911 (Part 1 / Sec 2): 2010, for fixed Head Piles in clay,

$$L_i / R = 0, L_f / R = 2.20$$

$$\therefore L_f \text{ (for 450 } \phi) = 4.95 \text{ m.}$$

$$\text{(for 500 } \phi) = 5.48 \text{ m.}$$

$$\text{(for 600 } \phi) = 6.60 \text{ m.}$$

Lateral Pile Capacity

From clause C - 4.2 of IS2911 (Part 1/Sec 2): 2010

Deflection $Y = \{H(e + Z_f^3) / 12EI\} \times 10^3$ for fixed Head Pile

Now for deflection $Y = 5.00 \text{ mm}$ and $L_i = 0$

$$H = (Y \times 12EI \times 10^{-3}) / (L_i + L_f^3) \text{ for fixed Head Piles.}$$

$$e = L_i \text{ and } Z_f = L_f$$

$$H \text{ (for 450 } \phi) = (5.00 \times 12 \times 5.03 \times 10^4 \times 10^{-3}) / (4.95)^3 = 24.88 \text{ kN says 24.90 kN.}$$

$$H \text{ (for 500 } \phi) = (5.00 \times 12 \times 7.67 \times 10^4 \times 10^{-3}) / (5.48)^3 = 27.96 \text{ kN says 28.00 kN.}$$

$$H \text{ (for 600 } \phi) = (5.00 \times 12 \times 15.90 \times 10^4 \times 10^{-3}) / (6.60)^3 = 33.18 \text{ kN says 33.20 kN.}$$

The lateral capacities of piles are presented in Table 3(C).

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TABLE 3(E)

Load Carrying Capacities of R.C. Bored Piles of Straight Shaft

Assumed Cut – off level = 1.50 m. below Avg. E.G.L.

(During the period of field work Avg. E.G.L was 1.62 m. below D. H. Road Level)

(Based on data from B.H.5, B.H.6, B.H.7, B.H.8, B.H.9, B.H.11 & B.H.12)

Termination Depth of pile below Avg. E.G.L. (m)	Cut-off Length Below Avg. E.G.L. (m)	Shaft Length (m)	Dia. of Pile (mm)	Recommended working Load on single pile in Axial Compression (kN)	Safe vertical Pull-out capacity with F.O.S. = 3.00 (kN)	Depth of fixity Below cut off Level (m)	Safe Lateral Capacity* (kN)
16.00	1.50	14.50	400 ϕ	240.00	154.00	4.38	22.40
			450 ϕ	270.00	174.00	4.95	24.90
			500 ϕ	300.00	196.00	5.48	28.00
20.00	1.50	18.50	450 ϕ	400.00	256.00	4.95	24.90
			500 ϕ	450.00	289.00	5.48	28.00
			600 ϕ	540.00	356.00	6.60	33.20

N.B. : These are the capacities of pile derived from sub-soil properties, not the structural capacities.

* Corresponding to 5.00 mm deflection of pile head at pile cap base level under fixed head condition, as per clause 8.4. of I.S.2911(Part4):2013.

Pile is considered to be fixed head and considering Grade of Concrete to be M25.



7. RECOMMENDATIONS

From the information, as supplied by the client, it is known that, there is a proposal for the construction of a Housing Complex comprising of 7 nos. (G+4) storied buildings at the site. However, whatever be the type, height and nature of the buildings, their foundation design should satisfy two basic criteria. They are as follows:

- (a) There must be adequate factor of safety against shear failure.*
- (b) The settlement of footings must be within permissible limits as defined in IS: 1904 (1986).*

Considering the above mentioned criteria and sub-soil condition in situ, it is suggested to try to use shallow foundations in the form of isolated and or combined and or single direction continuous strip footings or a judicious combination of some or all of them depending on column position and spacing, to be designed with the bearing capacity values as given in Table 3(A) of the previous chapter, for foundation design purposes.

However, if it is not possible to do the foundation designing operation with the bearing capacities as given in Table 3(A), by providing adequate footing area within available space, without over lapping, it is suggested to try to use shallow foundation in the form of a rigid raft foundation / both ways continuous strip footing (Virtual raft), to be designed with the bearing capacity and probable settlement value as given in Table 3(C), of the previous chapter, for foundation design purposes.

But, if it is not possible to do the foundation designing operation with the bearing capacities as given in Table 3(C), it is suggested to try to resort to shallow foundations only after doing the Ground Improvement work by 1.00 m. thick 'Compact Engg. Sandfilling'. The bearing capacity of shallow foundations resting on 'Compact Engg. Sandfilling' are provided in Extreme Right Hand Side Column of Table 3(B) of previous chapter for foundation design purpose.

In this case, for Ground Improvement by 'Compact Engg. Sandfilling' to be done, it is suggested to excavate the existing soil upto 2.50 m. below Avg. E.G.L. (During the period of field work Avg. E.G.L was about 1.32 m. below Diamond Harbour Road Level) and filling is



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to be done with medium sand [having uniformly co – efficient greater than 4. Sand to be used should conform to Grading Zone I or II as per Table 4 of I.S. 383-1970 (Reaffirmed 2002)] upto 1.50 m. below Avg. E.G.L.(During the period of field work Avg. E.G.L was about 1.32 m. below Diamond Harbour Road Level)with positive Confinement in the form of brick – work, so that thickness of sand fill layer is 1.00 m. Sand to be used should be well graded with little or no fines, classified as SW as per I.S. 1498(1970).

The compaction of sand pad having total thickness of 1.00 m, is to be done in two layers of 0.50 m thickness each and is to be compacted by plate vibrator.

Field 'N' value of Compact Engineering Sand fill should be minimum 20, which has to be verified at site after sand filling & Compaction.

At any cost, silver sand should not be used. Also, the SBC is to be verified by conducting sufficient number of Plate load tests, before foundation construction.

Normal conventional shallow footings, if used, should be properly connected in all directions by suitable tie beams to arrest / check differential settlement.

But, if it is not possible to do the foundation designing operation with the bearing capacities as given in Table 3(B), it is suggested to try rigid raft foundation/ both ways continuous strip footing (Virtual raft) to be placed at a foundation depth of 1.50 m. below Avg. E.G.L. (with underlying 1.50 m. thick Compact Engg. Sandfill layer) to be designed with bearing capacity and probable settlement values as presented in Table 3(D) for foundation design purposes.

In this case, for Ground Improvement by 'Compact Engg. Sandfilling' to be done, it is suggested to excavate the existing soil upto 3.00 m. below Avg. E.G.L.(During the period of field work Avg. E.G.L was about 1.32 m. below Diamond Harbour Road Level) and filling is to be done with medium sand [having uniformly co – efficient greater than 4. Sand to be used should conform to Grading Zone I or II as per Table 4 of I.S. 383-1970 (Reaffirmed 2002)] upto 1.50 m. below Avg. E.G.L.(During the period of field work Avg. E.G.L was about 1.32 m. below Diamond Harbour Road Level)with positive Confinement in the form



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of brick – work, so that thickness of sand fill layer is 1.50 m. Sand to be used should be well graded with little or no fines, classified as SW as per I.S. 1498(1970).

The compaction of sand pad having total thickness of 1.50 m, is to be done in three layers of 0.50 m thickness each and is to be compacted by plate vibrator.

Field 'N' value of Compact Engineering Sand fill should be minimum 20, which has to be verified at site after sand filling & Compaction.

At any cost, silver sand should not be used. Also, the SBC is to be verified by conducting sufficient number of Plate load tests, before foundation construction.

If fly ash filling is done to raise the existing ground level, an extra surcharge pressure intensity = $14.00 \text{ kN/m}^3 \times 1.20 \text{ m} \cong 16.80 \text{ kN/m}^2$ (considering unit weight of compact fly ash fill to be 14.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the superstructural load or subtracting 16.80 kN/m^2 from the bearing capacity of Table 3(A), 3(B), 3(C) & 3(D) during foundation design.

Similarly, if earth filling is done to raise the existing ground level, an extra surcharge pressure intensity = $18.00 \text{ kN/m}^3 \times 1.20 \text{ m} \cong 21.60 \text{ kN/m}^2$ (considering unit weight of earth fill to be 18.00 kN/m^3 and height of fill to be 1.20 m) would act extra on the foundation, which is to be considered in foundation design, either by adding with the superstructural load or subtracting 21.60 kN/m^2 from the bearing capacity of Table 3(A), 3(B), 3(C) & 3(D) during foundation design.

But finally, if it is not possible to do the foundation designing operation with the bearing capacities as given in Table 3(A), 3(B), 3(C) & 3(D) or if the designer wants to avoid the settlement of raft, then deep foundation is suggested to be used in the form of R.C. Bored Pile, the load capacities of which are given in Table 3(E) of previous chapter, for foundation design purposes.

For R.C. Bored Piles, the entire job is to be done as per relevant provisions of IS: 2911 (Part 1 / Sec 2), 2010.



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For pile having termination depth of 16.00 m. below D. H. Road Level, if done, the entire piling job can be done by Hand Auger piling method, although DMC piles would produce better quality pile.

But if piles having termination depth of 20.00 m. below D. H. Road Level is used, D.M.C. method of piling is strongly suggested.

It is strongly recommended that the design load carrying capacities should be tallied as per load test on test / working pile done as per IS: 2911 (Part 4):2013, if R.C. Bored Piles are used.

For R.C. bored piles, if provided, while considering the seismic effect, the skin friction component the pile load capacities, as given in Table 3(E) may be increased by 25 % as per Table 1 (clause 6.4.2.1) of IS: 1893 (Part I): 2016. However, no percentage increase in SBC given in Table 3(A), Table 3(B), Table 3(C) & Table 3(D), are allowed under seismic condition for shallow foundations, in present type of sub soil condition, as per (clause 6.4.2.1) of IS: 1893 (Part I): 2016.

Now-a-days, to ensure and assess quality control in R.C. piling job, load test as per IS: 2911 (Part 4) is not solitarily enough. It is suggested that load tests should be supplemented by low-strain non-destructive integrity tests on piles, in early, intermediate and last stage of piling job, if pile foundation is used.

Standing water level during the period of fieldwork was observed at an average depth of about 0.71 m. below Avg. E.G.L. (During the period of field work, Avg. E.G.L. was 1.62 m. below D. H. Road Level). Since the fieldwork was done in the months of January & February, it can be considered as moderate water table location, which can be normally expected to go down in dry season. So it is envisaged that, constant pumping will be necessary to keep the excavation water free, due to the position of water table above the founding depth in case of shallow foundation & above the cut off level in case of concrete piles.

No local information regarding highest position of ground water table during Monsoon was available from the site. However, for design purposes, it is advisable to consider the worst



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possible condition of standing water level to merge with E.G.L., which has been done in present case. Hence our results of bearing capacity of conventional shallow foundation, rigid raft foundation & deep foundation, as given in Table 3(A), 3(B), 3(C), 3(D) & 3(E) respectively, will not be affected by fluctuation of Ground Water Table position, since those were determined under worst condition.

Back filling of foundation pits should be done by good quality earth and by proper compaction.

However it is to be noted that, in the location of B.H.10, even if conventional shallow foundations are provided in the form of isolated square, rectangular, one way continuous strip footings or rigid raft / both ways interconnecting strip footings, with 1.50 m. founding depth, then also, a nominal sand cushion of 0.30 m. thickness, is suggested to be provided below the footings.

Last but not the least, due considerations should be given to open excavation of any sort. All sorts of precautionary measures like earth retainment by any suitable method, are to be adopted to avoid excessive ground settlement and damage to adjoining structures.

For and on behalf of GEOTEST ENGINEERS PVT. LTD.


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ANNEXURE

