# OPTIMUM AND ECONOMICAL GEOTECHNICAL INVESTIGATION

By:

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## INTRODUCTION:

Though significant progress has been made in accepting importance of geotechnical exploration in past 2 decades by planners, designers and National Building Code, there are many aspects which generalized tender documents, needs to be corrected.

The soils, being heterogeneous, amsotropic, sensitive to environmental changes and sampling technology, the extrapolated design foundation parameters including depth of foundation could be, a) over conservative, b) unsafe in some cases and c) unwise from cost of exploration / benefit to design ratio.

There are instances where application of exploration proved to disprove realities regarding stability for stable structures built when exploration was unknown.

## PROFESSIONAL PROBLEMS ON EXPLORATION:

Opinion poll of users mostly structural engineers, builders, planners can be summarized as:

a) Very expensive,

b) Time consuming,

c) Doubtful degree of credibility of the recommendation as judged from local observations or similar nearby problems,

d) Interpretation is subjective and variable

There is wide spread belief that “Art of foundation engineering (Pre 1950) was safer, economical and less time consuming” (Desai 1996).

This in my opinion due to

i)limited role of geotechnical engineers to exploration,

ii)generalized preliminary parameters in absence of specific structural data / problem, and

iii)Scanty details of site environment.

The lack of feedback of final design, performance of structure over years and professional transparency has throttled progress in Ground Engineering & Foundation Engineering.

## CRISIS IN GROUND ENGINEERING:

The crisis can be attributed to:

1. Poor qualified professional practice,
2. Poor control over tools, samplers & field test without keeping specific problems in view,
3. Establish non technical decision or prove otherwise

Codes for guiding design parameters are non existing or vague or outdated.

e.g. permissible differential settlement, interpretation of SPT, DCPT, CPT etc.

## a) Fresh Engineers:

Last decade of 20th century saw large demand for fresh graduates and post graduates in field of Geotechnical engineering. The employer recruited them as experts. Activity of such experts in public / private / government / academic sectors have led to crisis in soil engineering. The situation is similar to 1958 crisis in Europe (Henry Lossier, 1958).

An engineer, on large job, eager to practice theories just learnt, finds essential facts concealed in tables, graphs, field and laboratory notes representing a tropical jungle. Such deposits are inevitable with large numbers of drillings and sampling by number of agencies. Bulky volumes of data is a must for clients satisfaction for his large investment.

## b) Experts:

“Theory can only be used as a guide for judgement. Capacity for judgement is acquired by years of field contact - feedback of actions and reactions of soil behavior. Both the judgement and experience mends and sharpen tools of interpretations. Calibration of prediction by the feedback generates confidence and increase capacity to take calculated risk for cost and time optimizations.

Terzaghi (59) rightly stated: “Expertness required innate qualifications and desire over which we have no control”.

Engineers in field practised soil engineering without basic knowledge of subject and majority of scholars practised as consultants with little interaction with soils in field. This type of environment deepened crisis further.

For projects of thousands of crores every year, expected turnover will be in Tens of crores for exploration. With reputed 5 to 6 exploration agencies available in country, the scenario is critical. The high profit margin and shortage of agencies brought out mashroom growth of small drilling firms as soil explorer. They are ill equipped, non professional and have little interest in what they do. In fact there are few reports for sites never explored and there is no firm whose report of exploration was being scrutinized by the writer.

## c) Consulting Engineers:

Using exaggerated limitations and knowledge of Geotechnical engineering consultants developed a “Topo culture”. They pre-decided foundation for structure based on worst subsoil condition elsewhere.

It is this over-safe uneconomical design which consulting firms are interested in. The exploration in general multiplied crisis of confidence.

## Equipment industry:

## Professional Ethics:

Talks of using the art of foundation over scientific approach of Geotechnical engineering should not be taken lightly. All post - Terzaghi efforts will go waste if serious attempt to tackle the crisis is not perused by the profession.

## d) Real Experts:

In earth work and foundation engineering, a novice enters tropical jungle of data. His hopes, to obtain readymade (Text Book) solutions to problems using theory and tutorials learnt in class, will soon vanish. He will be soon frustrated. Only one with inquisitive mind, dares to plunge into the jungle with a bush knife (knowledge). He gets the taste of soil mechanics in action. He soon will realize that it will take years of welding his knife to attain competency. Latter in practice he may rise to expertness, not necessarily successful.

Habits of

(a) observations,

(b) analysis of causes and feedback of performance,

(c) renouncing old convictions and

(d) evolving appropriate new realistic models, will reduce gap between theoretical and actual behaviour.

It is a continuous process. This requires initiative, imagination, dedication, resourcefulness and through knowledge of basics of related soil mechanics. In quick buck spinning world surrounding, personal with such qualification will be few. These few experts cannot cope up with work in time.

Terzaghi (‘59) correctly stated, “Soil mechanics will not consistently serve its purpose until profession realize that it is supplement to and not substitute for common sense combined with knowledge acquired by experience. “Engineer must get used to the idea that he has to use his brains and judgement all the time even if he knows theory by heart.”

Failures if analysed recorded unbiased, are more valuable than success, as latter could be over-safe & uneconomical.

## EXPLORATION DATA FOR PROJECT:

The back bone of Geotechnical Engineering is soil exploration. It forms the basis for discussion which influence cost, time and safety of projects.

General complaints against exploration of soil as foundation materials are:

(a)Very expensive,

(b)Time consuming,

(c)Poor in reliability,

(d)Interpretation is personal and hence variable.

Bulkiness of generalized explorations records are inevitable for projects involving earthwork, roads, large industrial plots etc. Part of bulkiness is considered essential to satisfy the owner to justify large investment.

“Percentage significant information may range from 0 % to close to 100% depending on the qualifications of man who planned sub soil exploration. Even excellent records, undigested and un-condensed, cannot serve useful purpose. This task requires weeks and months of efforts, for which most of the cases had little time and personal” (Terzaghi’59).

Time lag between collection of data and use by the designer, leads to serve contamination. Crisis of confidence in profession, is an easy excuse to defend one's inability to digest. The analysis for reliability of data requires time and experience based judgements. The field layman’s classifications to laboratory tests and performance of structures around, present many contradictions. Pruning of the data or rechecking, though obligatory, is rarely done.

Laboratory CH soil could be Insitu layered clay and sand strata or expansive clay below water table which has different insitu behavior. The range of shear and compressibility parameters, based on standard investigation specifications, irrespective of subsoil, creates more confusion unless redundant or irrelevant results are discarded. Drainage conditions of triaxial, SPT, density from UDS or SPT, vane shear in layered or moist sandy clay etc. needs closer scrutiny. Expansive potential, in a soil report do not mean sub soil needs treatment, if subsoil is below water table or has equilibrium moisture. Swelling potentials has misguided designers to treat even swollen deposits, or deep moist deposits with no access for moisture.

Also universal standard specifications evolved, by consulting, firms do not consider soil at site. Thus bulk redundant data above and below the stressed zone, though useless, is inevitable.

# COMMON PRACTICES:

**Table 1**
**Code IS 1892 Investigation for Foundation - Objectives**

- Divide plot into zones having homogeneous subsoil.
- Provide vertical section (profile each zone showing layers A, B, C, D).
- For soil in each layer provide classification, structure, dry density, moisture, consistency, or relative density, shear parameters (UU-CU), compressibility, permiability by laboratory testing of adequate representative samples.
- Observe variations of ground water table.
- Check environmental aspects - expansive, collapsible, loessic soil by special tests and provide special properties - Swell, Shrinkage, Geology, Sesmicity, etc.
- Provide field log and observations to permit decisions on foundation system.

## Table 2
## Applications of Exploration

- Decide type of foundation – Shallow or deep; depth of foundation.
- Examine techno-economical feasibility of GRIMTECH for soft, expansive, collapsible, loessic soil.
- For saturated clay, SBC is f(w) and parameters $C_c$, $m_v$, $C_v$, $p'_c$ (pre-compressive stress), $H_{comp.}$ are required to predict settlement (St).
- For $c' - \phi$ soil unsaturated clay and silty find sand, SBC $q_s = f(R_d \cdot \phi - d_f - B - L)$, safe bearing pressure for allowable settlement, SBP is function of $R_d$, B, D/B and modulus E.

## Table 3
## Other Specifications and Exploration Cost

| | |
|---|---|
| Procedure | Pit hole by Augur, shell, wash boring |
| Extent | Every node of 60 m grid in vast area. 4 corners and one centre of large Building. For closely spaced structure maximum depth 4.5 × B or 1.5 × L. For B = 3 m explore 14 m<br><br>For Raft one or two 20 to 30 m deep bores for weak layers if any at depth. |
| Types | C = 0 Soil SPT at 1.5 m interval in stress zone. $\left(N_s - p_o' - N_s'' - R_d - \phi - E - q_{p40}\right)$<br>• $\phi$ = 0 Soil UDS @ 1.5 m interval in stress zone, 20% points replace by in situ vane test if soil is soft, sensitive<br>• C – $\phi$ soil suitable combination of above to obtain critical parameters (Table 2)<br>• Disturbed samples of each layer 6 to 8 per bore |
| Exploration for for 14,000 Sq.m. | 5 bores, B = 3 m, $D_f$ = 2 m, depth of exploration 10 m, 4 bores 10 m deep – One 15 m bore, 55 m of drilling, 2545 $m^3$ of soil represented by 1 m drilling |

Continue…

# Table: 3 Continue....

<table>
<tr><th rowspan="2">Test</th><th colspan="3">Tests</th><th>Remarks</th></tr>
<tr><th>Per Bore</th><th>Total</th><th>Per Layer</th><th></th></tr>
<tr><td>SPT</td><td>06</td><td>30</td><td>07</td><td>@ 1.5m interval</td></tr>
<tr><td>Undisturbed samples</td><td>05</td><td>25</td><td>06</td><td>@1.5 m interval</td></tr>
<tr><td>In situ Vane</td><td>01</td><td>05</td><td>2.5</td><td>Only for top 2 layers</td></tr>
<tr><td>Classification</td><td>06</td><td>30</td><td>07</td><td>Standard tests</td></tr>
<tr><td>Special Classification</td><td>02</td><td>10</td><td>05</td><td>DFI, swell potential, % clay SI etc. for top 2 layers</td></tr>
<tr><td>Structure</td><td>05</td><td>25</td><td>06</td><td>Density, moisture</td></tr>
<tr><td>Triaxial shear</td><td>02</td><td>10</td><td>2.5</td><td>On selected UDS</td></tr>
<tr><td>U C C</td><td>01</td><td>05</td><td>02</td><td>Sat. cohesive soil top 2 layers</td></tr>
<tr><td>Odeometer</td><td>02</td><td>10</td><td>2.5</td><td>—</td></tr>
<tr><td>OMC-MDD-CBR</td><td colspan="4">Total 10 to 15 Tests per project</td></tr>
<tr><td>Cost estimate</td><td colspan="4">Field and laboratory work 25000/-<br>+ Mobilisation (Varies) 10000/-<br>+ Report 5000/- i.e. Rs. 8000/- per bore or<br>0.2 to 0.3 Rs. per $m^3$ of soil explored</td></tr>
<tr><td>Time</td><td colspan="4">Normally 2 to 4 months</td></tr>
</table>

**Table 4**
**Practice of Field Exploration**

| No | Project | Drilling | S P T | | | U D Samples | | |
|---|---|---|---|---|---|---|---|---|
| | | | Depth per test (m) | Per bore | Per layer | Depth per sample (m) | Per bore | Per layer |
| 01 | IBP CO. Ltd., Hazira | 180 | 02 | 15 | 22 | 02 | 15 | 22.5 |
| 02 | Searle (I)Ltd., Ankleshwar | 100 | 1.6 | 06 | 15 | 1.7 | 0.6 | 15 |
| 03 | Cynides and Chem., Olpad | 40 | 1.6 | 12.5 | 6.0 | 10 | 02 | 01 |
| 04 | ONGC Gandhar | 855 | 1.7 | 11 | 83 | 4.3 | 4.5 | 50 |
| 05 | ONGC Hazira Phase-II | 260 | 1.4 | 11.8 | 47.5 | 4.3 | 3.8 | 15 |
| 06 | Rajula, Bhavnagar | 60 | 04 | 42 | 42.5 | 03 | 05 | 05 |
| 07 | Petro-Chem, Auraya (UP) | 1410 | 02 | 11 | 116 | 4.3 | 05 | 81 |
| Range in practice | | 40 – 1400 | 1.4 – 02 | 11 – 15 | 15 – 83 | 02 – 10 | 04 – 06 | 05 – 08 |
| Approximate by IS: | | Min. 55 | 1.8 | 06 | 07 | 2.2 | 05 | 06 |

In limited cases UDS is replaced by insitu vane test (2 tests per bore)

20% projects have prescribed cyclical load tests 2 nos. for design of machine foundations.

Even for vast area 83 results of SPT or 81 UDS per layer and 15 shear- odeometer tests per layer are bound to consume time and cost.

The range of parameters will be, for a jungle of data, very wide.

## INTERPRETATION:

1. Zoning of area in plan & model profile of soil for each zone showing critical stratification.
2. Location of water table.
3. Engineering properties of soil stratifications namely classification, plasticity, consistency, density (relative density), water content, shear parameters with appropriate drainage compressibility, permeability, CBR etc. Basis field, lab data, scrutinized & digested by judgment.
4. Type of structure, depth at which foundation is feasible, stress zone.
5. SBC, SBP, for permissible settlement and ABP, special problems for shallow foundation – footings, Raft.

6. Preliminary design report:

   There are no standard interpretations for codes. There are text books adopting over conservative local interpretations. My 2 books in references have brought out anamolies in performance and predictions.

7. Recommended approach:

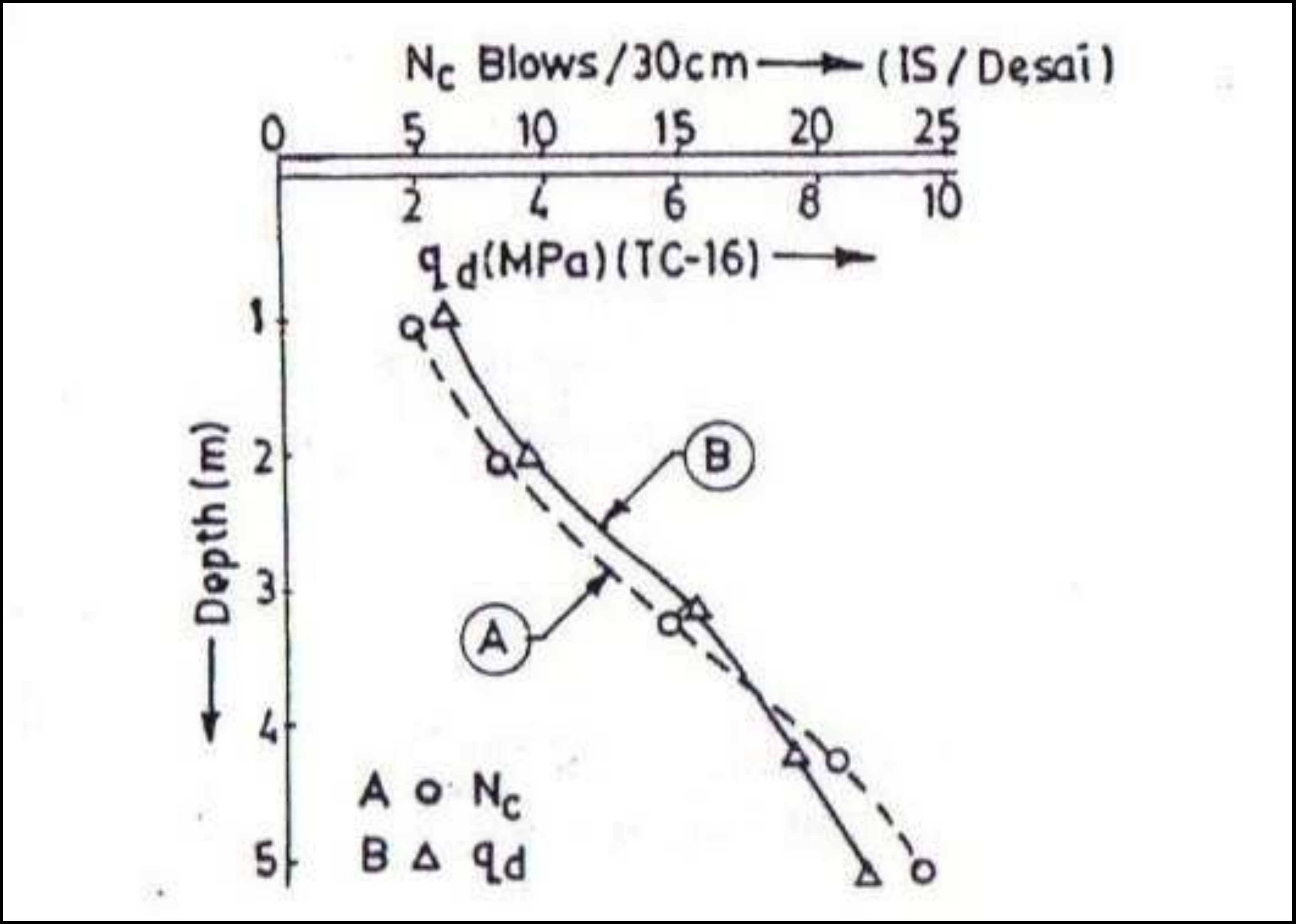
   IS code is guide and not rule. The local conditions, needs and specific studies requires application of mind, common sense and judgment based on experience. This will provide time and cost optimized programme to provide more reliable data for design of shallow foundation.

## SUGGESTED STEPS:

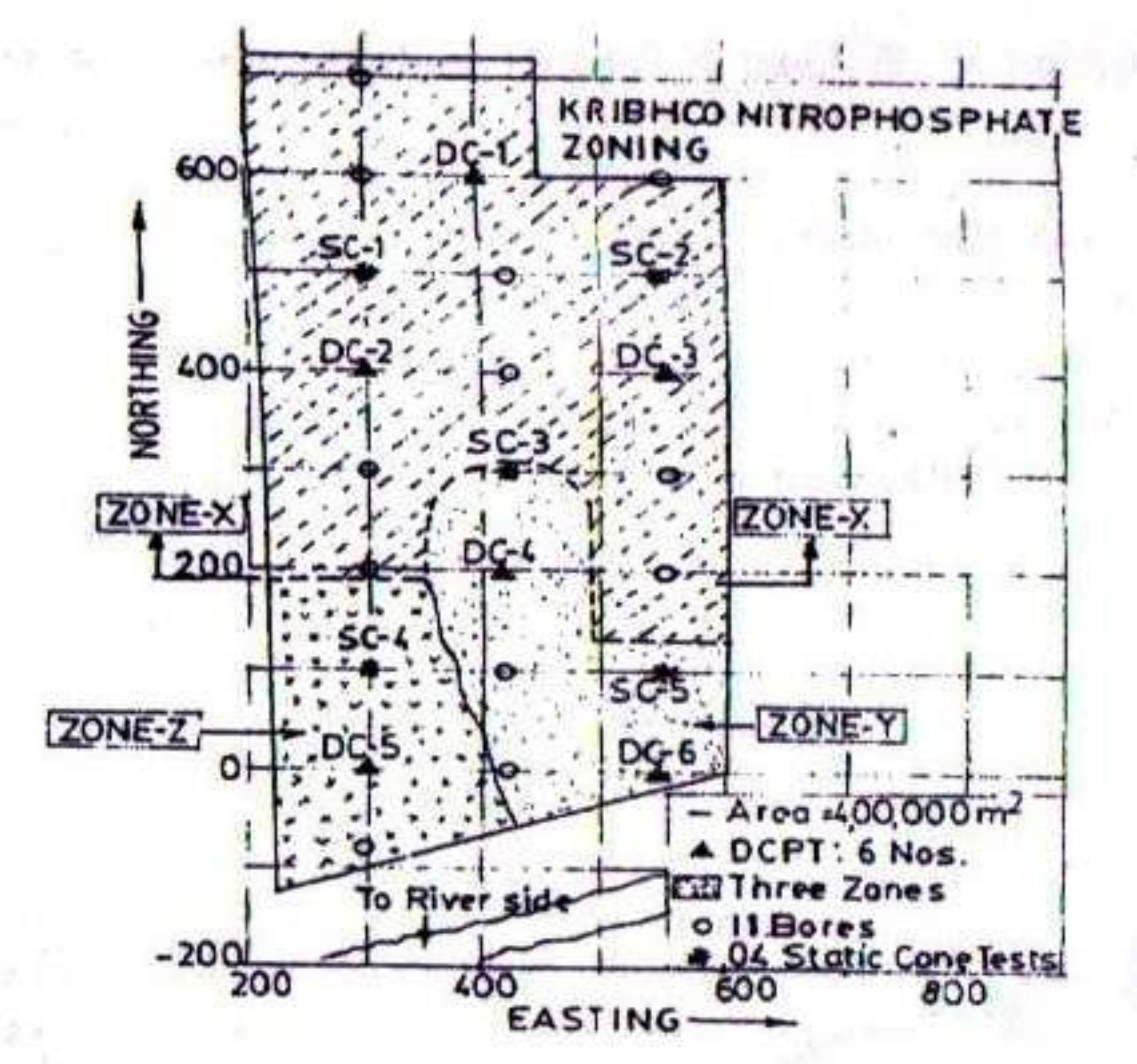
1. For large plots evolve 5 DCPT (4 corners & centre) which will indicate need for additional tests. The profile of Nc vs depth will be used to evolve

   a) Homogeneity of subsoil with depth

   b) Plan zoning plot to similar subsoil stratification

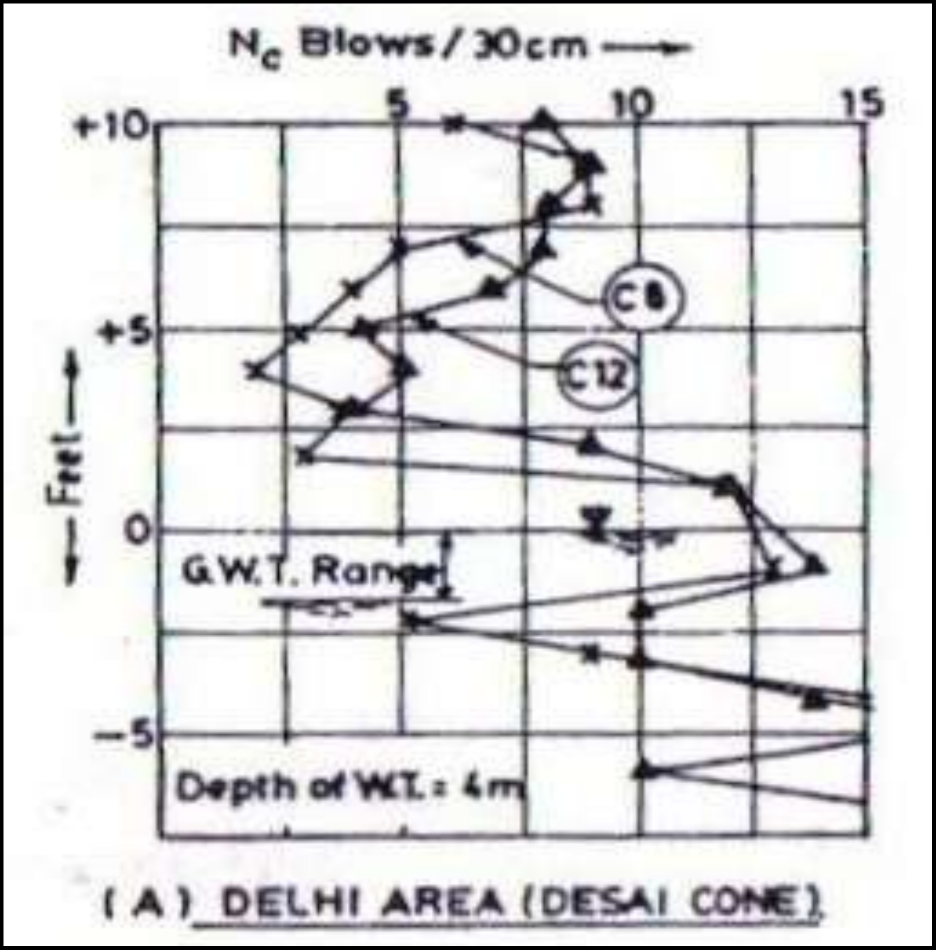



**Comparison of Trends of $N_c$ and $q_d$ for a site.**



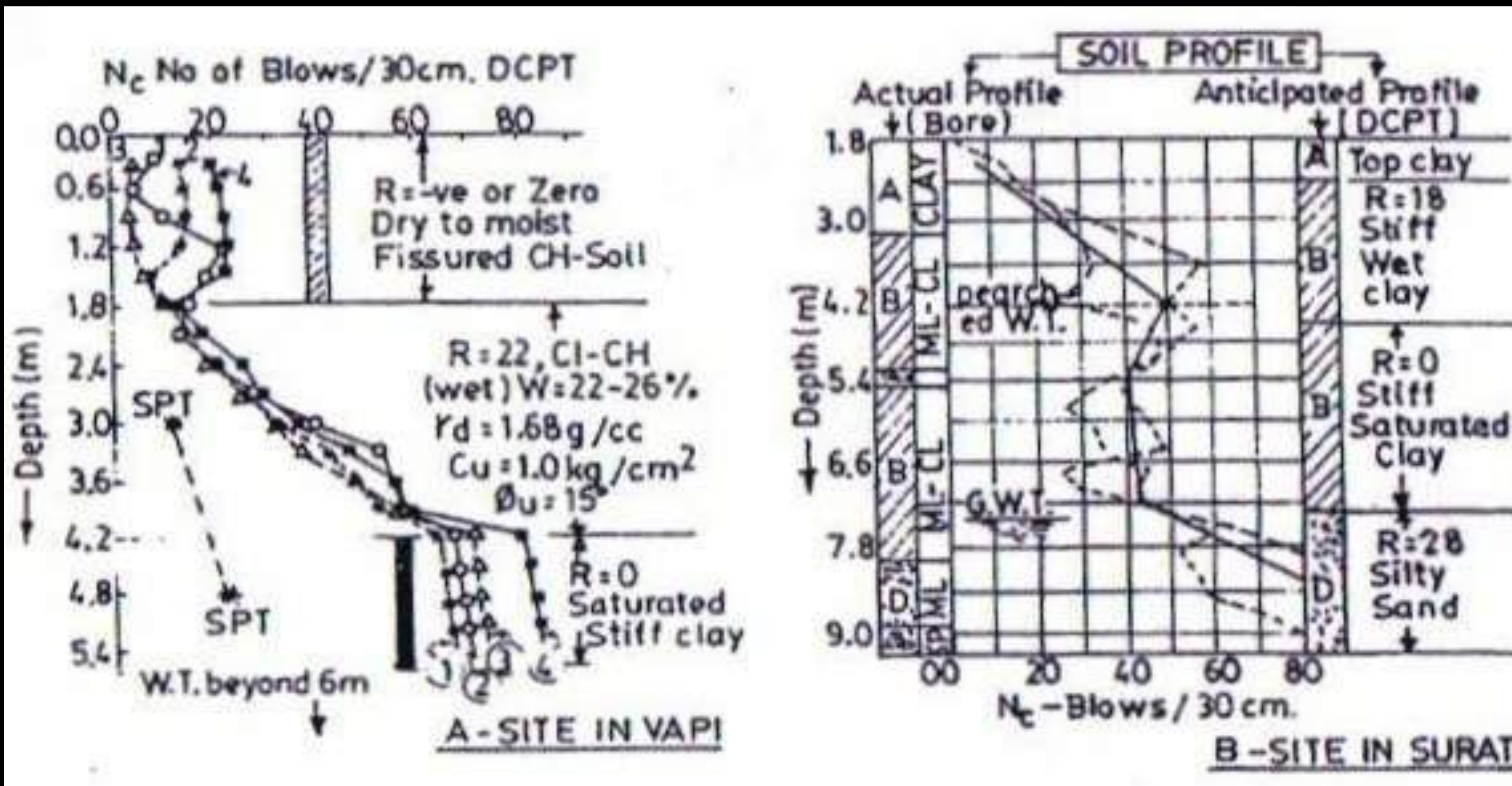

# Zoning Plan by DCPT to plan Detail Exploration (Desai, 1982)

2. Using proposed interpretation (Desai M.D.) estimates G.W.L. at site to be cross checked with available nearby bore / well data.




**Predicted and Actual Water Table**

3. Compute R = $\Delta N_c$/depth for slopes of data Nc vs depth and log the soil probability using local experience & recommendations.




## Soil Profile Developed by DCPT

4. For each strata in model profile depending on cohesive & non cohesive subsoil predict the engineering parameters.

   a) Saturated cohesive soil (R = _), Nc vs Cu, Øu = __, CBR = __, E = __.

   b) NC soils (R = __ ), avg. Nc, $P_0$', $R_d$, C – Ø, for type of sand fine and coarse.

5. The data (Nc > 10) indicate depth to found ($D_f$) the 6 storied common type or medium span factory structures. Consider the $D_f$ + 4 to 6 m as stressed zone.

Note: Soil below shows improving / decreasing Nc / strength!

6. Considering soil type, water table and Nc – $P_0$' derive SBC, PBC for settlement safe for structure and allowable bearing capacity.

e.g.

(1) Soil: Saturated Clay, Øu = 0,

As per Dr. M.D. Desai interpretation (Book - Ground Property Characterization from Insitu Testing):

Cu = 8 x Nc < 150 kPa, E = 600 x Nc upto depth 5 m

Observed: average or representative Nc = 10,

This gives Cu = 80 kPa, E = 4800 kPa, SBC = 250 kPa,

## (2) Soil: Silty fine sand (SM),

| Nc=10 | Rd | Ø | E | SBC | PBC, St=25mm | Liquefaction |
|---|---|---|---|---|---|---|
| $P_0$=0 | 80% | > 30º | 15 MPa | - | 400 kPa | Low |
| P0=50 kPa | 50% | 28º | - | - | 250 kPa | Medium |

7. If capacity of soil is inadequate marginally examine going deeper, feasible ground treatment for soil (such as raft, pile, dynamic compaction, static compaction, sand wicks, geo-fabrics / grids reinforcing etc.)

8. Plan bore / two per zone with specific critical parameters. (Personally supervised)

   e.g. Subsoil saturated clay with sand strata below 7 m, $D_f$ = 2 m, require $C_u$, $E_u$ from proper UDS at foundation level and 2 & 4 m below (only one SPT for check). UDS not reliable if $N_s$ > 25 adopt pit block or other method for UDS. Soil $\rho_d$ – water content profile with depth (with note of season) will be required.

   If the strata is clay to 2 m overlaying SM – ML non plastic silt and sand above W.T. model average $N_c$ at proposed depth of foundation ($N_c$ > 10), plan bore hole with SPT at 2.0, 3.0, 4.5, 6.0 m depth conducted carefully keeping personal supervision for drop, piping in borehole etc. UDS do not provide any reliable data. Two samples may give some range of $\rho_d$, water content. As shear and compressibility are sensitive to $\rho_d$ – w and strata could be dilatants in sampling data needs careful analysis before testing as UDS/ recommended for engineering properties.

9. Compare model data with bore, after scrutiny and digesting data to derive final design parameters. In case of marginal design bearing capacity and stress for the structure additional check test vane / static cone / pressurometer can be adopted.

10. In case of large gap between practice and design, derived a prototype test can be conducted on 1.0 to 1.5 m square footings to get non challengeable design parameters, checked by pressurometer, CPT or by model load tests for UBC, PBC etc.

# CASE STUDIES

# NHAI,

# DWARAKA, DELHI

**Significance of Experience Based Judgement.**

**(2008)**

# SOIL EXPLORATION FOR NHAI, PLOT - G 3, SECTOR – 10, DWARAKA, DELHI – CASE STUDY OF SIGNIFICANCE OF EXPERIENCE BASED JUDGEMENT. (2008)

## Introduction:

Building: Seven stories, Plot area 6086 sq.m.

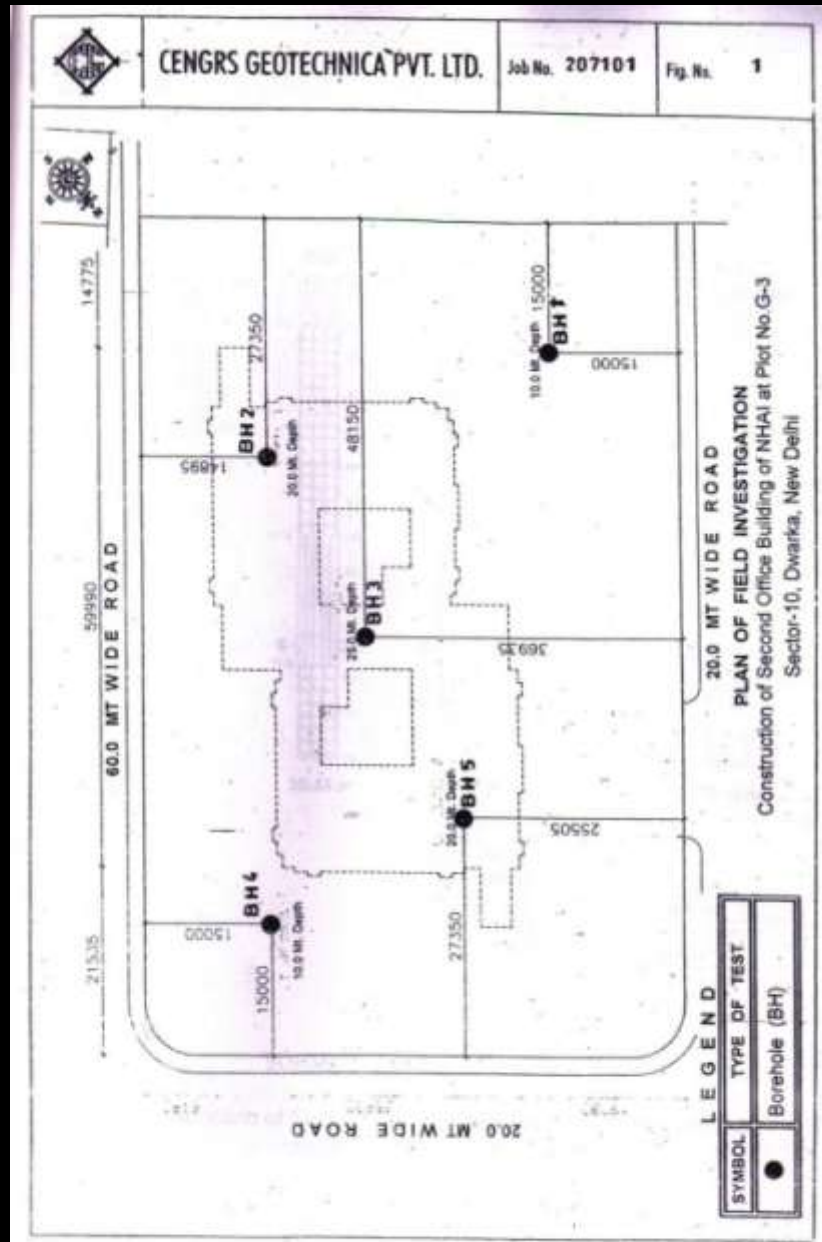
Soil exploration: 5 boreholes for geotechnical profile.

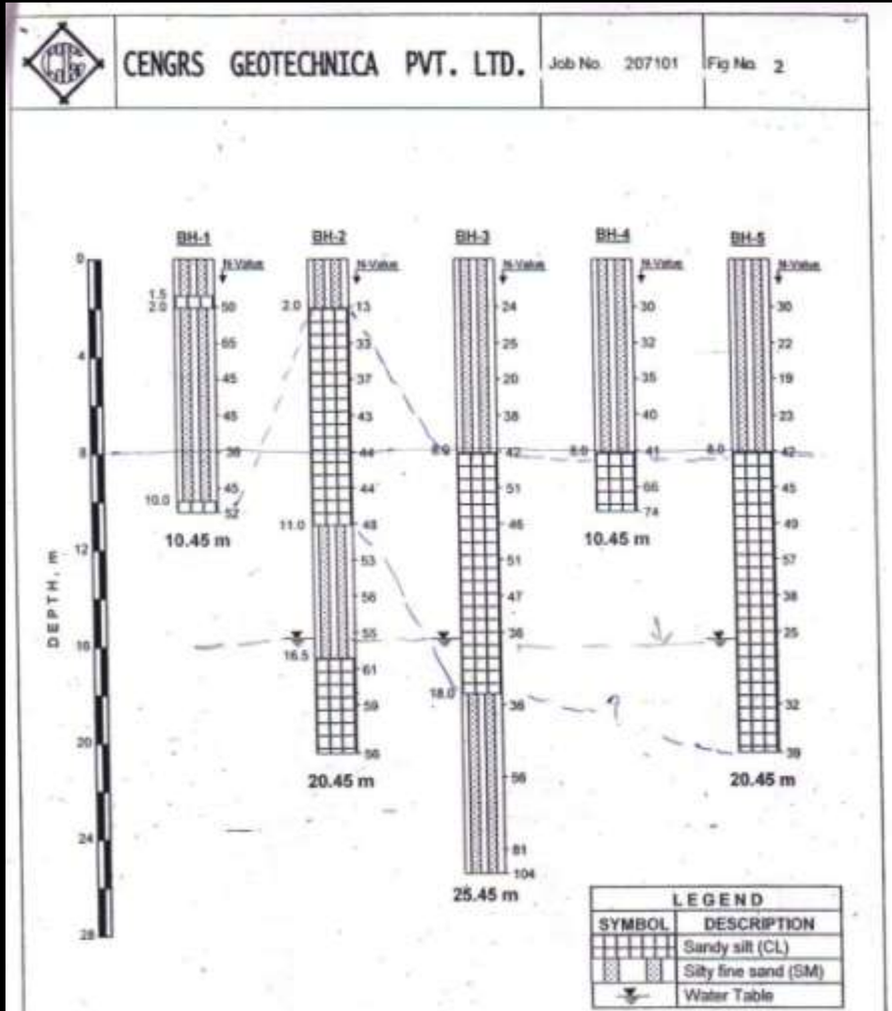
Fig. 1 – Plan

Fig. 2 – Note BH: 2, exception to soil profile 0 – 10 m of SM,
Silty fine sand, BH-2, Clay 12 %, silt 45 %, Check data,
effect (differential settlement)

Fig. 3 – SPT – BH: 2 (Red), W.T. @ 16.5 M (Sept 2007)

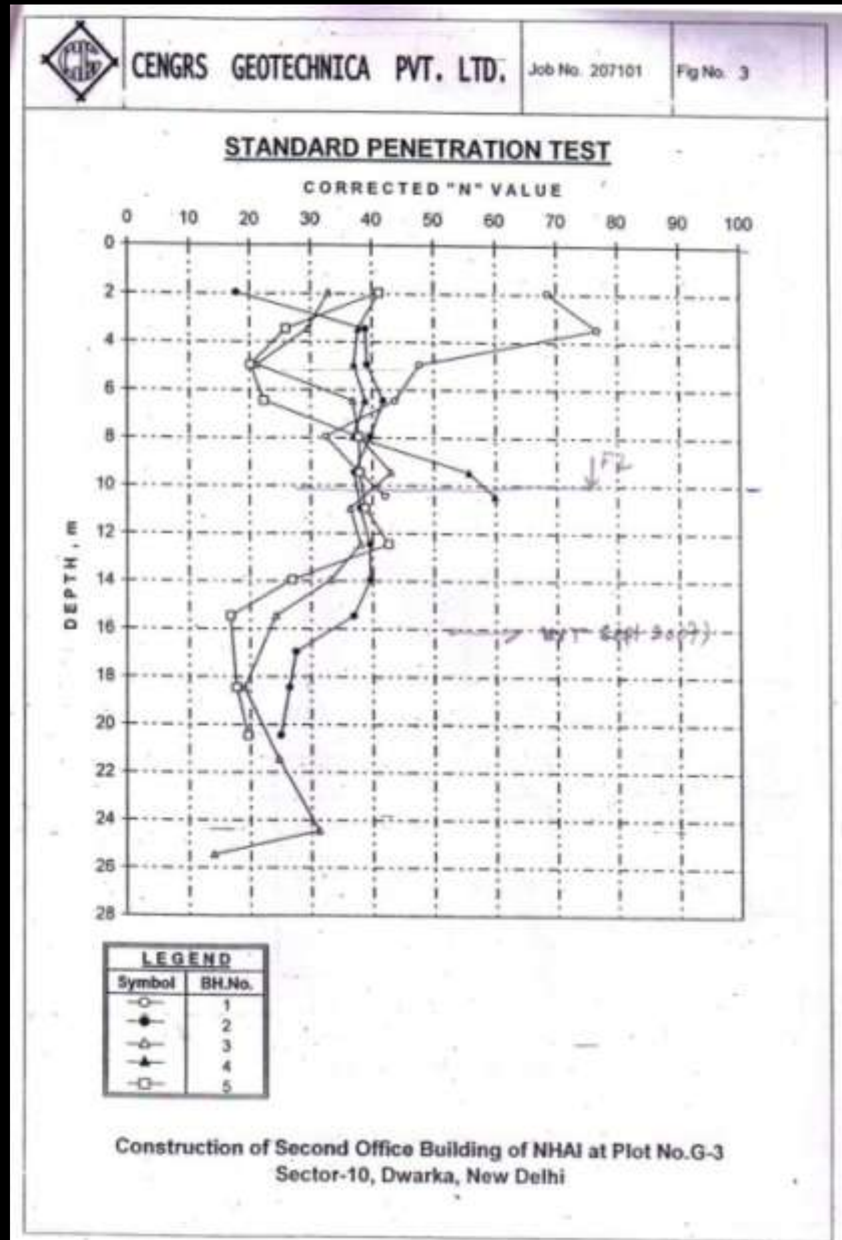
Liquefaction Potential … ?


CENGRS GEOTECHNICA PVT. LTD.
Job No. 207101
Fig. No. 1
60.0 MT WIDE ROAD
20.0 MT WIDE ROAD
20.0 MT WIDE ROAD
BH 1
BH 2
BH 3
BH 4
BH 5
PLAN OF FIELD INVESTIGATION
Construction of Second Office Building of NHAI at Plot No.G-3
Sector-10, Dwarka, New Delhi
LEGEND
SYMBOL
TYPE OF TEST
Borehole (BH)




SUMMARY OF BOREHOLE PROFILE

Construction of Second Office Building of NHAI at Plot No.G-3
Sector-10, Dwarka, New Delhi


CENGRS GEOTECHNICA PVT. LTD.
Job No. 207101
Fig No. 3
STANDARD PENETRATION TEST
CORRECTED "N" VALUE
0 10 20 30 40 50 60 70 80 90 100
DEPTH, m
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28
LEGEND
Symbol
BH.No.
1
2
3
4
5
Construction of Second Office Building of NHAI at Plot No.G-3
Sector-10, Dwarka, New Delhi

# Analysis:

## Main structure:

| | | | |
|---|---|---|---|
| SPT at, | 2.0 m, | varies | 13 to 30, |
| | 9.0 m, | ” | 19 to 40, |
| | Up to 25 m, | ” | 38 to 41. |

Depth for foundation – 9 to 10 m

## Note:

1) Data above 9.0 m: Not required if foundation depth 9.0 m

2) Isolated footings:

Soil below 9 m – CL, depth: variable 2 to 3 m thick,

$N_{SPT} > 30$, $C = 0.9$ kg/cm$^2$, $\text{Ø} = 5^{\circ}$, $W_L = 27$, $PI = 9$, $W = 9\text{-}10$ %, $\rho_d = 1.5$,

## Design report:

Net allowable bearing capacity = 25 T/m$^2$, k = 2.2 kg/cm$^3$, F.S. = 2.5, $S_t$ < 50 mm

(Data? --- Specific UDS / $C_u$ on Pit samples / w - $\rho_d$)

## Dr M.D. Desai interpretation:

Based on UDS, SBC = 25 T/m$^2$,

Based on $N_{SPT}$ > 30, Cu > 12 T/m$^2$, SBC > 33 T/m$^2$, recommended.

## Light structure:

## Design report:

$D_f$ = 1.2 m on SM, Allowable bearing capacity = 10 T/m$^2$ as per report.

## Dr M.D. Desai interpretataion:

$D_f$ = 2 m, Minimum $N_{SPT}$ = 13, $N_{avg}$ = 15, $P_O$ = 30 kPa, $R_d$ = 70 %, Ø = 35º,
Permissible bearing capacity for $S_t$ 40 mm = 50 T/m$^2$ (No W.T.)

Net design bearing capacity = 30 T/m$^2$.

Liquefaction not likely…

## Cost aspects:

100 T/ 10 = 10 sq.m. x 0.7 = 7 m$^3$/cc

100 T/ 25 = 4 sq.m. x 0.6 = 2.4 m$^3$/cc per footing cost !!!!

# PANIPAT

**Need to seek second/third opinion on decision affecting society. Common sense- forecast of liquefaction at Panipat by Jain Aswin et at (2007): interpretation of SPT**

## SPT?

- The SPT test results are sensitive to equipment operator efficiency and age of equipment (rope).

- To evaluate relative density of non-cohesive sand NSPT observed blow/30 cm is interpreted with corrections for efficiency of kit, tools ware & tear by IS code & ASTM etc.

## Considered correlation is function of effective overburden pressure at test depth around 1960 by,

- DR. M.D. Desai
- Gibbs et al
- Schultze

# Soil Profile for Site: 1 (Panipat Division)

| Sr. No. | Depth (m) | IS Class$^n$ of soil | Ø in Degree | $D_{50}$ (mm) | N-value | Rd (%) | Unit weight γ (kN/m$^3$) | Remarks |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.75 | ML | 25 | 0.160 | 8 | 28.0 | 20.0 | |
| 2 | 1.50 | ML | 25 | 0.155 | 8 | 28.0 | 20.0 | W.T. at 1.5m |
| 3 | 3.00 | ML | 25 | 0.150 | 8 | 28.0 | 20.0 | |
| 4 | 4.50 | ML | 31 | 0.090 | 8 | 28.0 | 20.2 | |
| 5 | 6.00 | CL-ML | 31 | 0.150 | 13 | 39.5 | 20.2 | |
| 6 | 7.50 | ML | 33 | 0.140 | 13 | 39.5 | 20.1 | |

| Sr. No. | References for interpretation. | $N_s$ considered as corrected | $R_d$ % |
|---|---|---|---|
| 1 | UD samples not fully reliable in Non cohesive soils (ML) – Ns (Table 1) | Observed by UDS data | 28 |
| 2 | Jain et al (2007) $(N_i)_{60}$ (Table 2) | 9 | 11-38 |
| 3 | Desai M.D. (1970) $N_{cor}$ (Table 3) | 18-22 | 51-60 |
| 4 | Gibbs et al (1957) $N_{cor}$ (Table 3) | 25 | 60-67 |
| 5 | Schultze $N_{cor}$ (Table 3) | 20+ | 60-76 |

# Comparison of $R_d$ and corrected N-value by different methods:

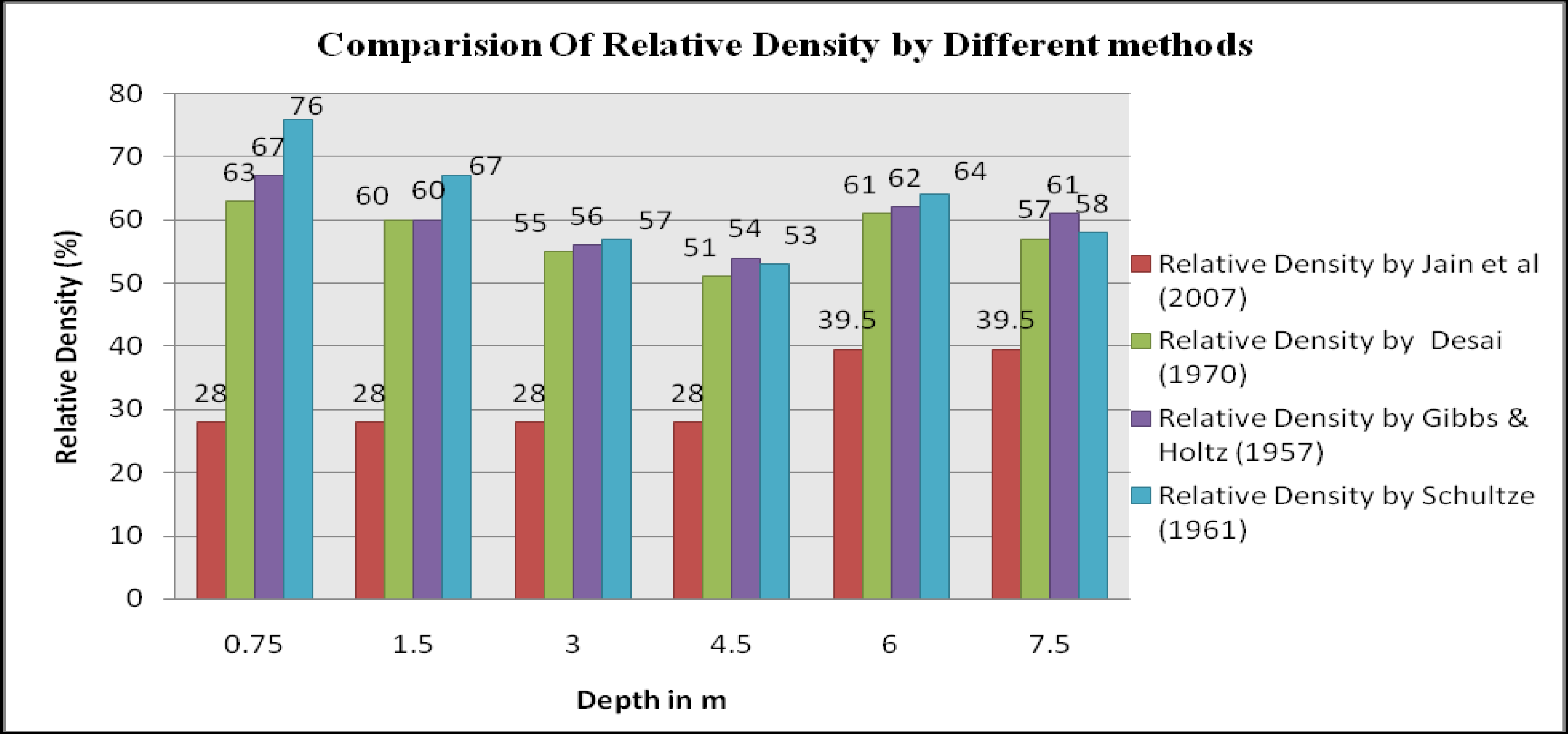
| Depth (m) | Author(2007) Jain et al | | Desai(2005) | | Gibbs & Holtz (1957) | | Schultze (1961) | |
|---|---|---|---|---|---|---|---|---|
| | $R_d$(%) | $(N_1)_{60}$ | $R_d$(%) | $N_{corr}$ | $R_d$(%) | $N_{corr}$ | $R_d$(%) | $N_{corr}$ |
| 0.75 | 28.0 | 9 | 63 | 24 | 67 | 31 | 76 | 31 |
| 1.50 | 28.0 | 9 | 60 | 22 | 60 | 25 | 67 | 26 |
| 3.00 | 28.0 | 7 | 55 | 18 | 56 | 20 | 57 | 19 |
| 4.50 | 28.0 | 7 | 51 | 16 | 54 | 19 | 53 | 14 |
| 6.00 | 39.5 | 10 | 61 | 22 | 62 | 26 | 64 | 22 |
| 7.50 | 39.5 | 10 | 57 | 19 | 61 | 26 | 58 | 19 |

# Ranges of $R_d$ predicted and denseness of soil:

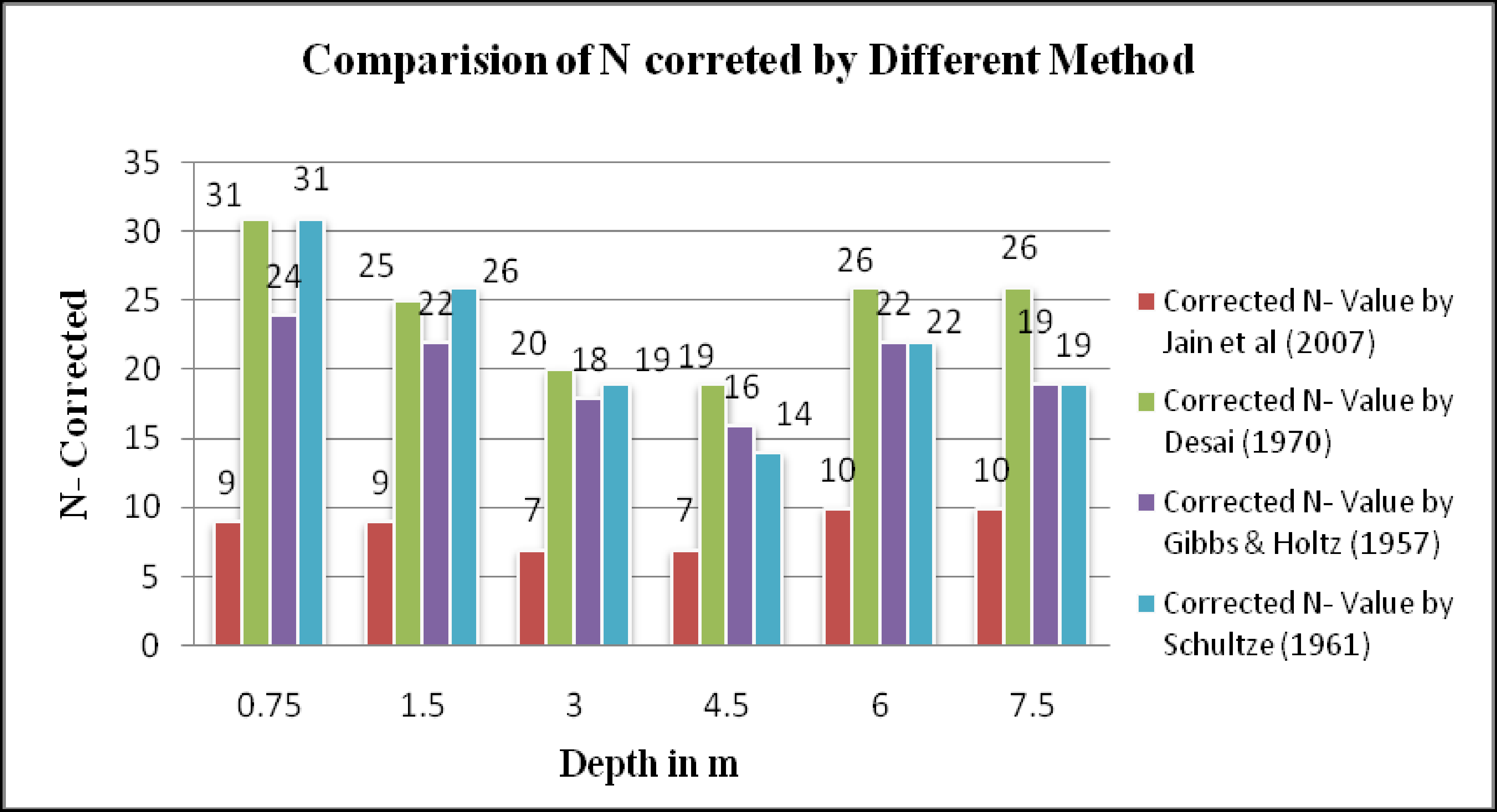
| Site No. | Jain et al | | India | | USA | | Germany | |
|---|---|---|---|---|---|---|---|---|
| | Jain et al (2007) | | Desai(2005) | | Gibbs & Holtz(1957) | | Schultze(1961) | |
| | $R_d$ (%) | Dense ness | $R_d$ (%) | Dense ness | $R_d$ (%) | Dense ness | $R_d$ (%) | Dense ness |
| 1 | 28 to 39 | Loose | 51 to 63 | Medium | 54 to 67 | Medium | 53 to 76 | Medium |
| 2 | 11 to 38 | Loose | 28 to 60 | Medium | 29 to 62 | Medium | 29 to 66 | Medium |
| 3 | 11 to 39 | Loose | 28 to 57 | Medium | 38 to 61 | Medium | 30 to 60 | Medium |
| 4 | 18 to 36 | Loose | 33 to 56 | Medium | 46 to 55 | Medium | 42 to 55 | Medium |
| 5 | 11 to 35 | Loose | 28 to 57 | Medium | 37 to 60 | Medium | 30 to 60 | Medium |
| | 11-36 % | | 30- 60 % | | 36 to 62% | | 30to 65 % | |

# State of denseness of non cohesive ml (silt and fine sand):

- The analysis by Jain et al shows sand is loose. Undisturbed sample of dilatants sand also shows loose state (Rd = 30%). The researches by Desai, Gibbs & Schultze (1970) suggested Rd > 60% i.e. mid, dense to very dense. The graphics depicting N SPT corrected by different methods and relative density predicted



# Comparison of corrected N-value:



# Need for check test / model prototype test:

- This text book/R&D in geotechnical design cannot be applied successfully & economically without judgment experience based on strong common sense.

- The feedback, prototype expensive tests must be accepted for a generalized dangerous predictions such as liquefaction potential low bearing capacity justifying pile/ raft at high cost for project.

## Liquefaction potential assessment by seed at al (1985):

- The predicted data of density based on Ns at given depth has been analysed using Seed et al (1985) approach.
- The computed stress ratio & shear resistance as well as likely hood to liquefy or no are tabulated in Table 4.
- The NSPT corrected by Desai (1970) is shown in Table 4A. Similarly potential for liquefaction by Schultze (1961), Gibbs & Holtz (1957) approach to correct N are presented

# Liquefaction Potential by Desai (2005) Ncorr

| Depth (m) | $\sigma'_v$ (kN/m$^2$) | $\tau_{av}$ (kN/m$^2$) | N | $N_{corr}$ | Stress Ratio | $\tau_h$ (kN/m$^2$) | Liquefaction | Remark |
|---|---|---|---|---|---|---|---|---|
| 0.75 | 15.00 | 2.3137 | 8 | 24 | - | - | No | |
| 1.50 | 30.00 | 4.5747 | 8 | 22 | - | - | No | W.T at 1.5m |
| 3.00 | 45.00 | 8.9388 | 8 | 18 | 0.290 | 13.050 | No | |
| 4.50 | 60.30 | 13.1359 | 8 | 16 | 0.265 | 15.980 | No | |
| 6.00 | 75.60 | 17.1204 | 13 | 22 | - | - | No | |
| 7.50 | 90.75 | 20.8713 | 13 | 19 | 0.320 | 30.640 | No | |

# Liquefaction Potential by Schultze (1961) Ncorr

| Depth (m) | $\sigma'_v$ kN/m$^2$) | $\tau_{av}$ (kN/m$^2$) | N | $N_{corr}$ | Stress Ratio | $\tau_h$ (kN/m$^2$) | Liquefaction | Remark |
|---|---|---|---|---|---|---|---|---|
| 0.75 | 15.00 | 2.3137 | 8 | 31 | - | - | No | |
| 1.50 | 30.00 | 4.5747 | 8 | 26 | - | - | No | W.T at 1.5m |
| 3.00 | 45.00 | 8.9388 | 8 | 19 | 0.32 | 14.400 | No | |
| 4.50 | 60.30 | 13.1359 | 8 | 14 | 0.23 | 13.869 | No | |
| 6.00 | 75.60 | 17.1204 | 13 | 22 | - | - | No | |
| 7.50 | 90.75 | 20.8713 | 13 | 19 | 0.32 | 30.640 | No | |

# Liquefaction Potential by Gibbs & Holtz (1957)

| Depth (m) | $\sigma'_v$ (kN/m²) | $\tau_{av}$ (kN/m²) | N | $N_{corr}$ | Stress Ratio | $\tau_h$ (kN/m²) | Liquefaction | Remark |
|---|---|---|---|---|---|---|---|---|
| 0.75 | 15.00 | 2.3137 | 8 | 31 | - | - | No | |
| 1.50 | 30.00 | 4.5747 | 8 | 25 | - | - | No | W.T at 1.5m |
| 3.00 | 45.00 | 8.9388 | 8 | 20 | 0.360 | 16.200 | No | |
| 4.50 | 60.30 | 13.1359 | 8 | 19 | 0.320 | 19.296 | No | |
| 6.00 | 75.60 | 17.1204 | 13 | 26 | - | - | No | |
| 7.50 | 90.75 | 20.8713 | 13 | 26 | - | - | No | |

# Liquefaction Potential by Seed et al (1985) with data of Jain et al.

| Depth (m) | $\sigma'_v$ (kN/m$^2$) | $\tau_{av}$ (kN/m$^2$) | N | $(N_1)_{60}$ | Stress Ratio | $\tau_h$ (kN/m$^2$) | Liquefaction | Remark |
|---|---|---|---|---|---|---|---|---|
| 0.75 | 15.00 | 2.3137 | 8 | 9 | 0.165 | 2.4750 | No | |
| 1.50 | 30.00 | 4.5747 | 8 | 9 | 0.165 | 4.9500 | No | W.T at 1.5m |
| 3.00 | 45.00 | 8.9388 | 8 | 7 | 0.145 | 6.5250 | Yes | |
| 4.50 | 60.30 | 13.1359 | 8 | 7 | 0.145 | 8.7435 | Yes | |
| 6.00 | 75.60 | 17.1204 | 13 | 10 | 0.175 | 13.2300 | Yes | |
| 7.50 | 90.75 | 20.8713 | 13 | 10 | 0.175 | 15.8813 | Yes | |

# Other methods to predict liquefaction potential:

- There are three methods by,
  - Youd et al (2001),
  - Idriss and Boulanger (2004)
  - Iwasaki et al (1984)

  to evaluate potentiality of liquefaction.

# Liquefaction Potential by Youd et al (2001) with data of Jain et al.

| Depth (m) | $\sigma'_v$ (kN/m$^2$) | $\tau_{av}$ (kN/m$^2$) | N | $(N_1)_{60}$ | Stress Ratio | $\tau_h$ (kN/m$^2$) | Liquefaction | Remark |
|---|---|---|---|---|---|---|---|---|
| 0.75 | 15.00 | 2.3137 | 8 | 19.0 | 0.320 | 4.800 | No | |
| 1.50 | 30.00 | 4.5747 | 8 | 13.0 | 0.215 | 6.450 | No | W.T at 1.5m |
| 3.00 | 45.00 | 8.9388 | 8 | 11.0 | 0.195 | 8.775 | Yes | |
| 4.50 | 60.30 | 13.1359 | 8 | 9.0 | 0.160 | 9.648 | Yes | |
| 6.00 | 75.60 | 17.1204 | 13 | 13.0 | 0.215 | 16.254 | Yes | |
| 7.50 | 90.75 | 20.8713 | 13 | 12.0 | 0.215 | 19.511 | Yes | |

# Liquefaction Potential by Idriss and Boulanger (2004) with data of Jain et al.

| Depth (m) | $\sigma'_v$ (kN/m$^2$) | $\tau_{av}$ (kN/m$^2$) | N | $(N_1)_{60cs}$ | Stress Ratio | $\tau_h$ (kN/m$^2$) | Liquef$^{\underline{n}}$ | Remark |
|---|---|---|---|---|---|---|---|---|
| 0.75 | 15.00 | 2.3137 | 8 | 15 | 0.1535 | 2.3022 | No | |
| 1.50 | 30.00 | 4.5747 | 8 | 14 | 0.1459 | 4.3760 | No | W.T at 1.5 m |
| 3.00 | 45.00 | 8.9388 | 8 | 13 | 0.1385 | 6.2319 | Yes | |
| 4.50 | 60.30 | 13.1359 | 8 | 12 | 0.1313 | 7.9177 | Yes | |
| 6.00 | 75.60 | 17.1204 | 13 | 16 | 0.1614 | 12.2006 | Yes | |
| 7.50 | 90.75 | 20.8713 | 13 | 15 | 0.1535 | 13.9285 | Yes | |

# Liquefaction Potential for Site: 1 by Iwasaki et al (1984) with data of Jain et al.

| Depth (m) | $D_{50}$ (mm) | $\sigma'_v$ (kN/m$^2$) | L | $(N_c)_{60}$ | R | FL | Liquefaction | Remark |
|---|---|---|---|---|---|---|---|---|
| 0.75 | 0.160 | 15.00 | 0.2373 | 8 | 0.347 | 1.462 | No | |
| 1.50 | 0.155 | 30.00 | 0.2346 | 8 | 0.329 | 1.402 | No | W.T at 1.5m |
| 3.00 | 0.150 | 45.00 | 0.3056 | 8 | 0.315 | 1.032 | No | |
| 4.50 | 0.090 | 60.30 | 0.3351 | 8 | 0.351 | 1.048 | No | |
| 6.00 | 0.150 | 75.60 | 0.3484 | 13 | 0.346 | 0.994 | Yes | |
| 7.50 | 0.140 | 90.75 | 0.3353 | 13 | 0.336 | 1.003 | No | |

# CONCLUSION:

- Similar forecast of low Rd leads of ground treatment by vibroflotation, compaction pile, blasting. A blasting prototype test to examine liquefaction at Ukai satisfied surcharge correction and analysis by Desai, Schultze & Gibbs et al.

- Foundations for Barrage, ISBT Delhi, many structures were provided simple foundations ignoring predictions of low Rd by codal approach. Akshardham was suspected to be liquefiable by SPT value.

- Check by field density & DCPT, CPT test permitted foundations without piles. Check tests or even prototype load tests to conform realistic denseness are advisable before deciding foundations. Over safe uneconomical design is non engineering and corrodes economy for decades. “TOPO” of success due to over safe design normally over rules real economical design.

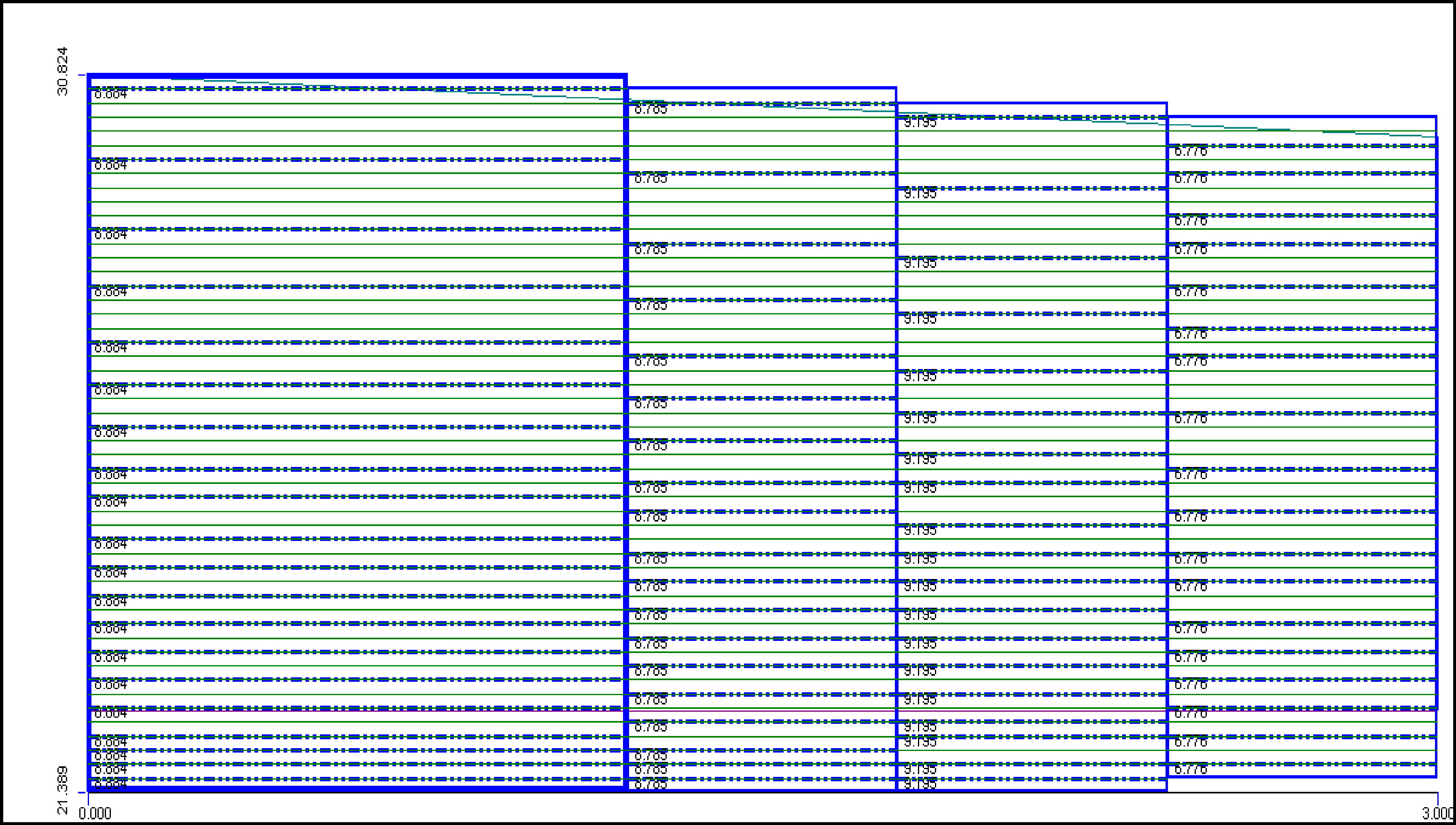
# REINFORCED

# RETAINING EARTH WALL

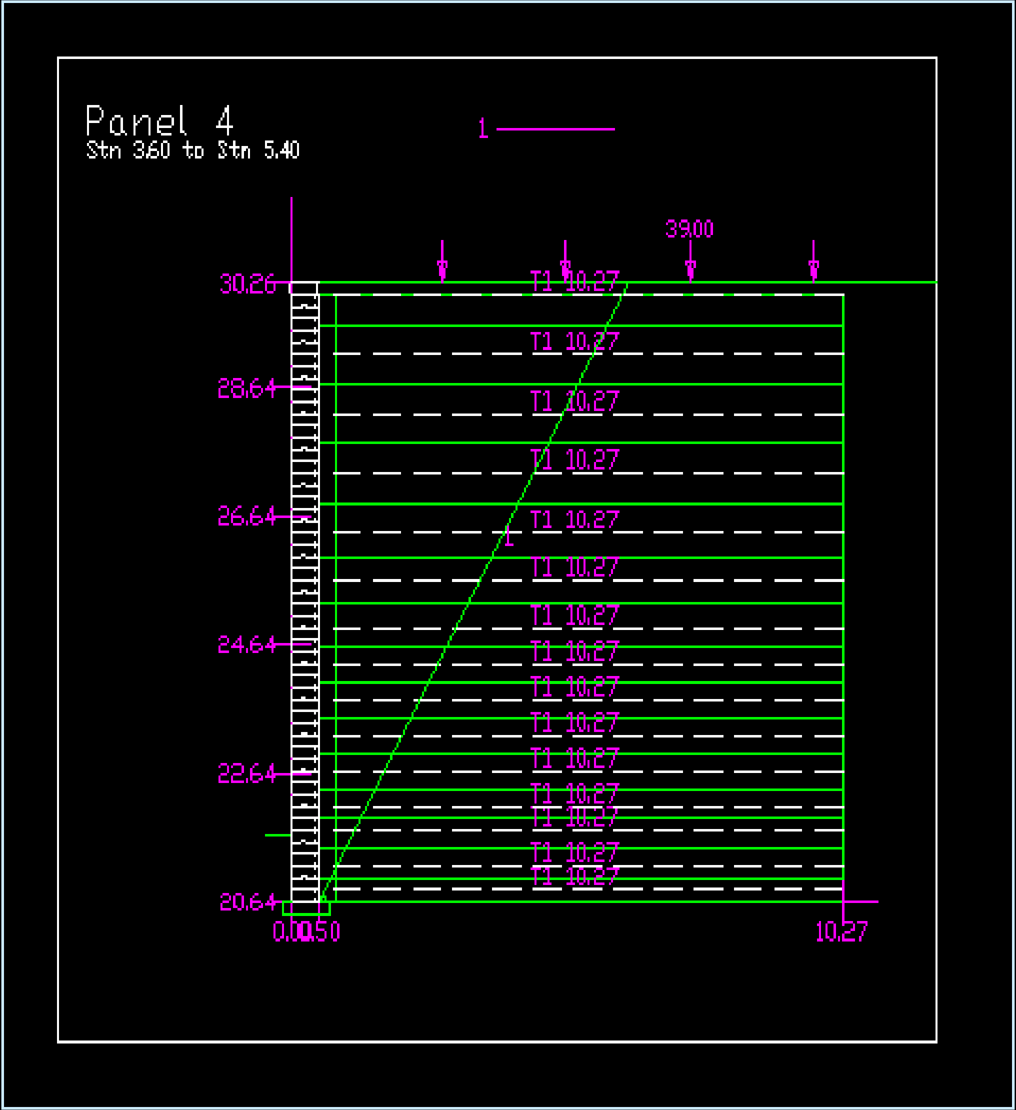
# Component of Reinforced Retaining Wall:

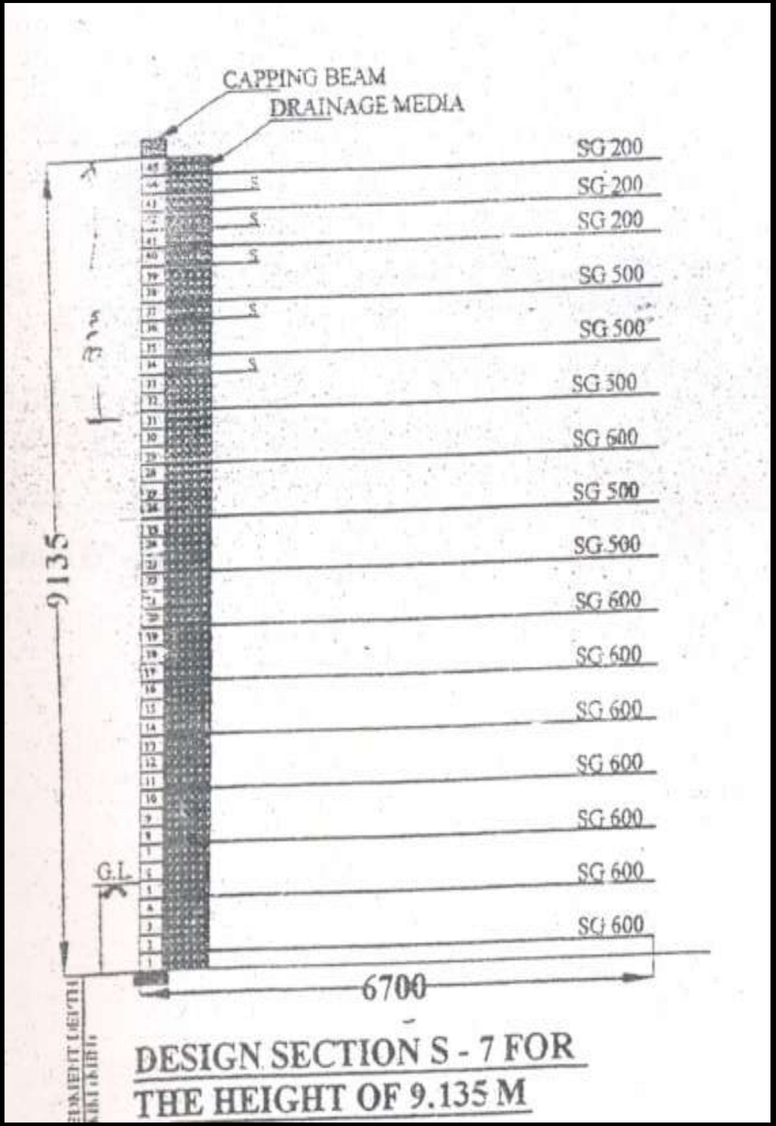
- Original ground.
- Reinforcing element.
- Facing element.

# Elevation of Reinforced Retaining Earth Wall.



# Reinforced Retaining Earth Walls:



## Reinforced Retaining Earth Wall Foundations:

- RE zone C' = 0, $\phi = 35^0$, $\gamma_d$ = 18 kN/ m$^2$.
- Retained soil with C' = 0, $\phi = 35^0$, $\gamma_d$ = 20 kN/ m$^2$.
- Foundation soil with C' = 0, $\phi = 30^0$, $\gamma$d = 18 kN/ m$^2$.

The walls designed at times of excavation were found to rest expansive CH soil. Report of soil investigation No:708094 Unique Engineering Testing & Adv. Services.

- The filled up or road work soil strata shows 1.5 mt, 1.8 mt, 1.5 mt & 2.7 mt in Bore Hole 1,2,3 & 4 respectively.

- These strata could be improved if site permits to place footings at 1.6 mt below Ground Level, the SBC of improved soil has to be evaluated by technology.

- Prima-facie it could give design bearing capacity of 150 Kpa for RE Walls on saturated cohesive soils overlaying expansive soils.

- The strata below up to 4.0 mt in Bore Hole 1, 2 & 3 is CH type expansive soil above Water table at 3.0 mt. During summer, soil could shrink if Water Table is not permanent.

- This normally options are to by pass & place footings of RE Wall at 4.0 mt. The Water Table being at 3.0 mt, the footings can be provided at Water Table with care not to allow the subsoil to dry during excavation.

- The design will consider CH saturated soil with avg. Ns = 10 (Bore hole 1, 2 & 3) & Ns = 25 for Bore Hole 4, w = 25 ± 2 %, $\gamma$d = 1.5, Cu = 7 T/mt2 (Bore Hole 1, 2 & 3) Cu = 10 T/mt2 (Bore Hole 4).

- The design SBC for above two cases will be net 160 Kpa (Bore Hole 1, 2 & 3) & 250 Kpa (Bore Hole 4). Recommended SBC will be net 160 Kpa, UBC = 240 Kpa. The Water Table fluctuations could cause (differential) settlement due to swell/shrinkage depending on the surrounding.

# Soil improvement technique:

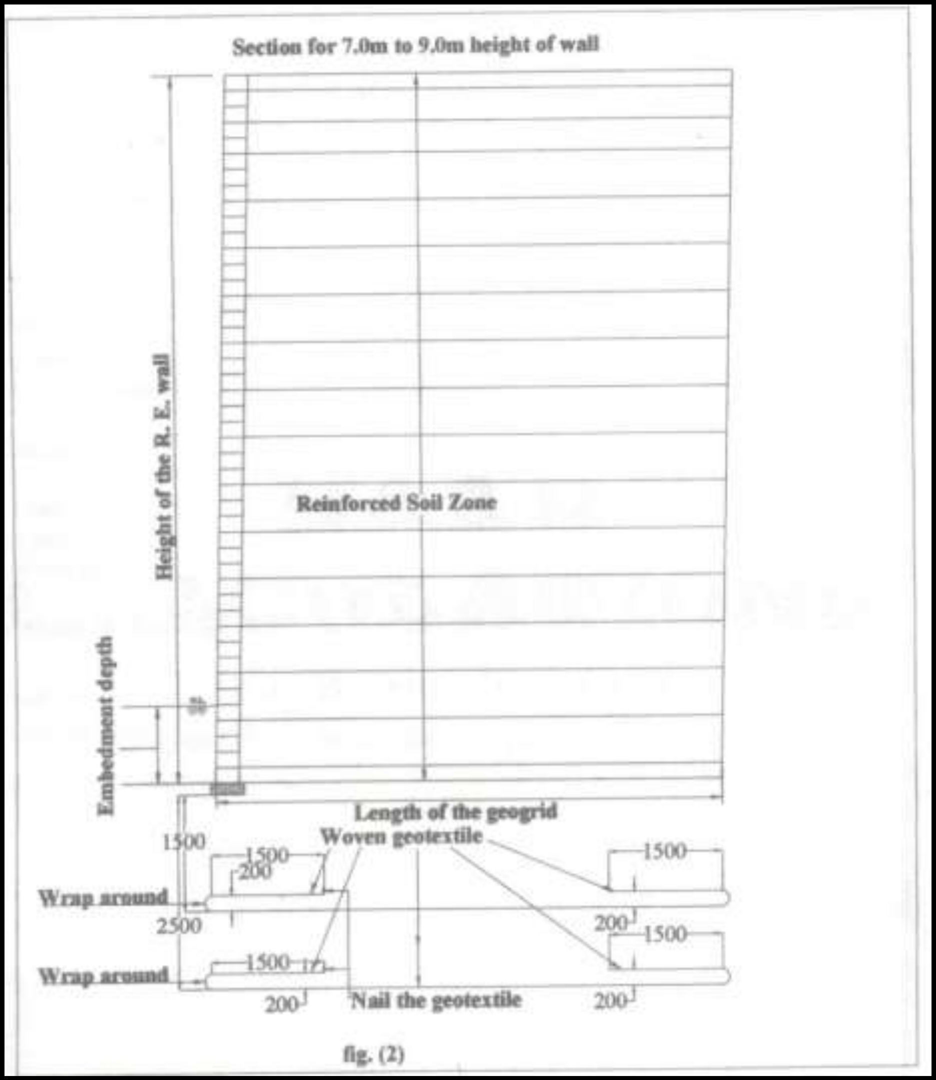
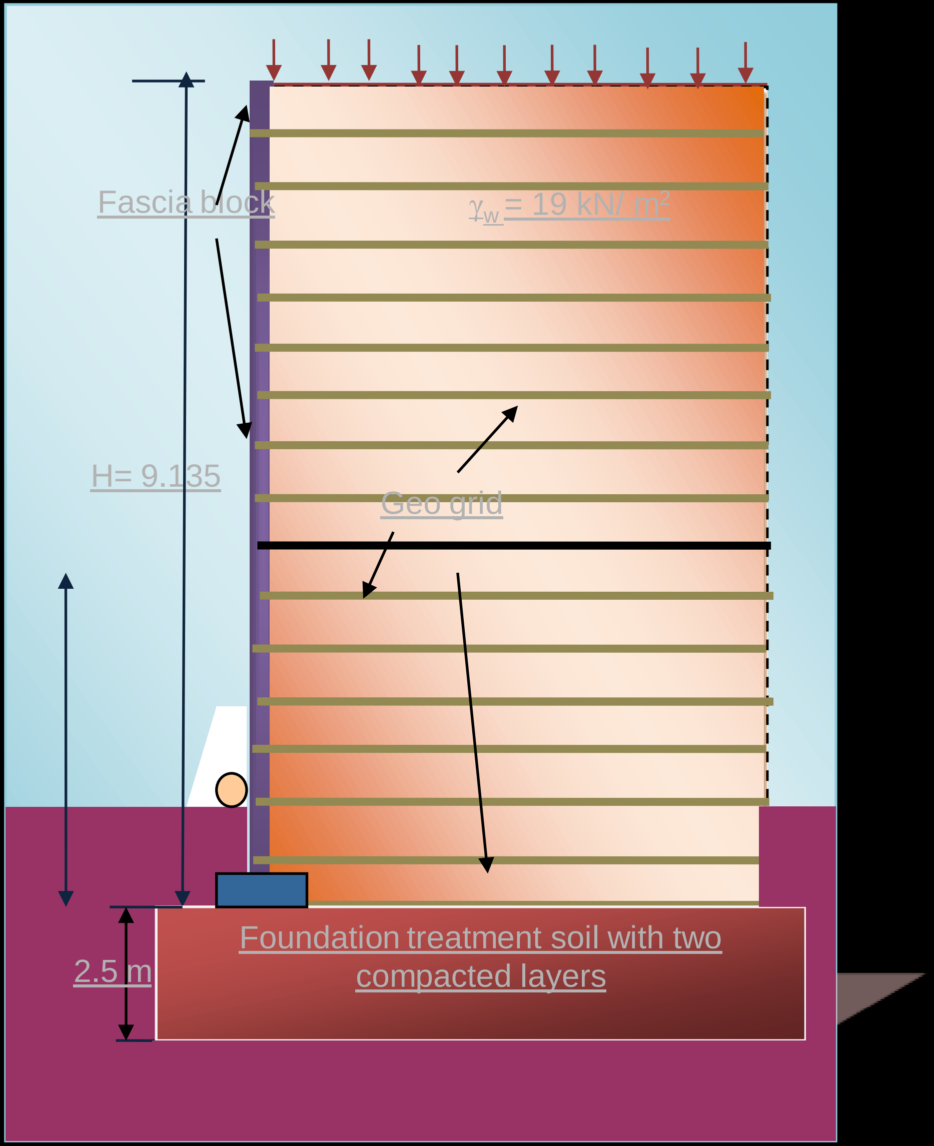



fig. (2)

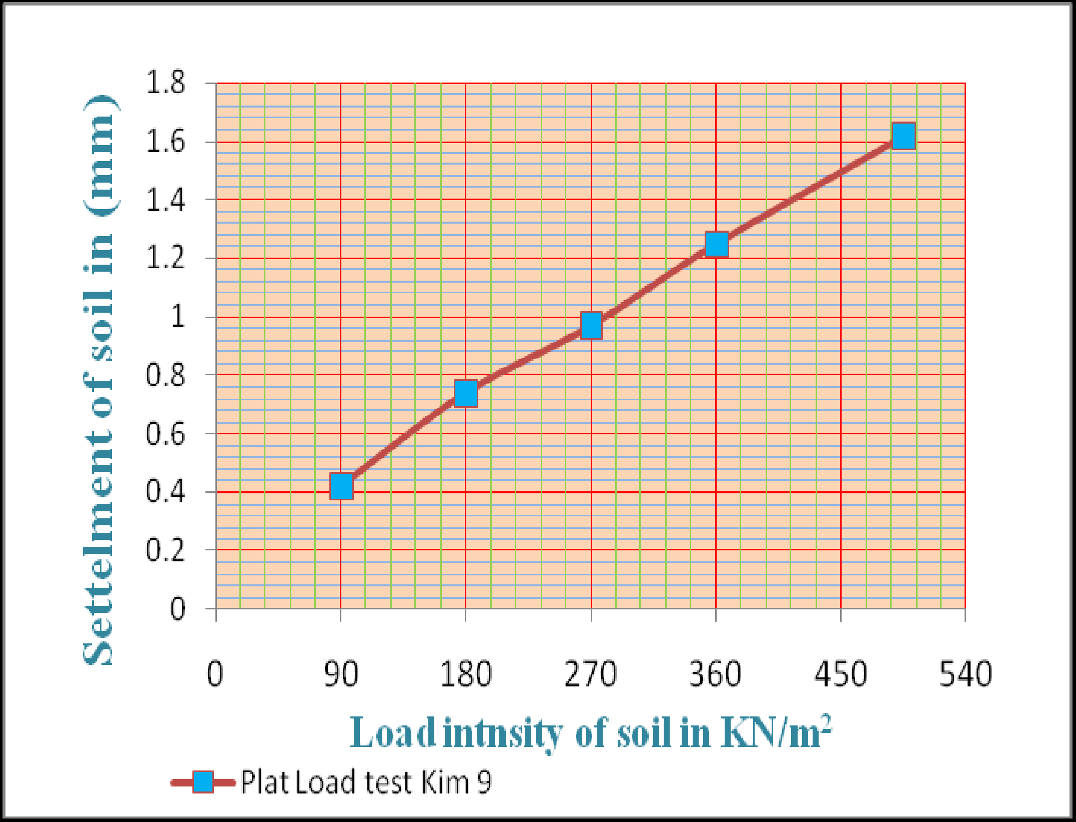


- The Ultimate bearing capacity will be 400 Kpa at 3m below G.L. 3m strata below will be made up by selected earth fill reinforced by Geotextile 200 gm/m2 or so in 2 layers The stress analysis of RE wall max height will create bearing pressure of 247 Kpa. Factor of safety against bearing capacity will be 1.67 < 2.5 required.

- To attain minimum bearing pressure 2.5 m soil was replaced by Geotextile reinforced fabric (200-300 gm/ m2) in 2 layer as shown fig 3, to avail minimum of FS = 2.5. The backfill soil was specified as sandy soil C' = 0, $\phi$ = 340, $\gamma$d = 19 kN/ m2 at OMC. The site was load tested by plate load test on 0.45 x 0.45 x 0.025. Plate which gave load settlement curve Fig. 7.5. The loading to 495 Kpa reached settlement of 1.60 mm. the FS would be more then 2.0.

# Load intensity vs Settlement curve

| Load intensity in soil kN / m$^2$ | Corresponding Avg. settle$^t$ in ( mm) |
|---|---|
| 90 | 0.42 |
| 180 | 0.74 |
| 270 | 0.97 |
| 360 | 1.25 |
| 495 | 1.62 |



# Plate Load Test :

# Conclusion:

- **The need for model test and proper geotechnical site data interpretation is justified by above case study.**

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THANK YOU