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Role of Geotechnical Investigations in Engineering Judgement*

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Introduction

In addition to analysis and design, engineering involves decision making on the basis of available information. It is primarily an exercise in judgement regarding,

* the technical feasibility of the project,
* the economic implications and the time requirement for completion,
* the acceptability of the solution anticipated, and
* the performance of the final outcome.

All structures except those which float or fly, rely upon soil deposits and/or rock formations for their support. Geomaterials and water exert pressure on, or utilized in construction, thus affecting the safety of all structures. Despite this significance, soil generally implies that it makes up the ground on which we live and makes us dirty. Most people take soils for granted and are not overly concerned. However geotechnical engineers are one such group, who are deeply concerned with the soil, the others being geomorphologists, geologists, hydrologists, agronomists, etc.

The inherent nature and diversity of the geological processes involved in soil formation are responsible for the wide variability in its in-situ state. In the continuous geo-material spectrum, very soft and soft clays form one end with extremely hard rock at the other end.

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Only a minute fraction of soil can be sampled and tested because of practical and economical constraints. For example, even if the spacing of bore holes is as close as 10 m and 50 mm diameter sample is tested for every second meter, only one millionth of the total volume would have been actually explored. In contrast to many other situations in engineering where one would normally specify the requirements of the materials used, geotechnical engineer usually has to adjust his design to accommodate to the prevailing properties of the in-situ soils. Hence with innovative approaches which are rational and simple, practical problems can be tackled satisfactorily and economically, if the soil variability in terms of soil parameters due to erratic conditions can be realistically arrived at and appropriately accounted for.

In-Situ Soils in Geo-Material Spectrum

The in situ soil formations might arise due to sedimentation or may originate as non-sedimentary residual deposits in origin. Although soils, primarily, are particulate media, the stresses to which they are subjected to, the environment in which the deposits are formed and the time, in the geological time scale, that has elapsed, have all been recognized as potential factors to impart their effects to the in-situ soil systems encountered. It is very well known that the equilibrium state of the in situ deposits are the resultant effects of stress, time and environment. They are neither mutually exclusive processes nor a simple superposition of their influences is tenable in the analysis and assessment of their engineering behaviour.

Subsurface investigations to know the relative disposition of earth materials in the substratum, field and laboratory investigations to determine the engineering properties of geo-materials, analysis and design with the incorporation of appropriate material properties culminating in the construction of contemplated structure are a chain of events in the practice of geo-engineering. The discussions in this lecture mainly pertain to the first two aspects only.

Broadly, the field of Geotechnical Engineering has grown very fast since its introduction in the second half of this century. Considerable achievements are apparent in the areas of field exploration, basic understanding of the behaviour of soils, developments in the methods of analysis, construction methods and instrumentation.

In the professional practice, in our country Geotechnical exploration, design and consultancy picked up slowly since 1980. Its growth has been rather slow and much more is desired. In a vast country like ours, with expenditure of thousands of crores of rupees per year being in various sectors, having only a handful of major exploration agencies shows a poor scenario and infrastructure. Exploration services, provided at high cost and considerable time in providing the needed information, have failed to establish their credibility and degree of accuracy needed at engineering level. Further analysis of soil data culminating in the design of structures have not been developed as full fledged involvement in consultancy.

Field and construction aspects of Geotechnical engineering have undergone a limited growth. On a national scenario, piling, dewatering or ground engineering as an industry is practically non-existent. Thousands of successfully built earth dams and other pertinent structures are the achievements about which the profession can take pride. These projects involved extensive studies on geotechnical designs, analyses, and ground improvements.

There have been rapid developments in geotechnical instrumentation industry which in a way has turned the tide from imports to self sufficiency and has taken up the challenges of new economic policy by exporting equipment. The instrumentation for performance studies has not yet been systematically organized under consulting services. Vast opportunities at our doorstep have not yet motivated people to provide planning, procuring, monitoring and related services to dams, retaining walls and such other geo-structures.

Soil Mechanics in Action

Terzaghi (1959) defined "Soil Mechanics in Action is application of soil mechanics as a tool in foundation engineering practice". Tools like Computer or Calculator can be expertly and quickly mastered by reading a manual. Same is not the case with geophysical exploration which can only be used with tolerable confidence after many years of experimentation.

Civil engineer is expected to be conversant with various facets of practice. In fact structural engineers freely work as geotechnical designers. Today in India, Geotechnical Engineer in most of the cases has to accept a role of assisting a consulting engineer or a structural designer.

Although every aspect of geotechnical engineering has undergone phenomenal change and has contributed significantly to the soil mechanics in action, the discussions mainly center around subsurface exploration, its role and significance in the practice of geotechnical engineering. The specific aspects of presentation with considerable bias towards subsurface exploration are:

- Subsurface exploration – critical appraisal,
- Economical aspects of foundation system,
- Legal implications in geotechnical engineering,

- Significance of second opinion and prototype studies, and
- Prediction versus performance.

Subsurface Exploration

The back bone of Geotechnical Engineering, is soil exploration. It forms the basis for engineering decisions 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 subjective and hence questionable.

"Percentage of significant information may range from 0 to close to 100 percent depending on the qualifications of the person who planned sub-surface exploration. Even excellent records, undigested and un-condensed, cannot serve useful purpose. This task requires weeks and months of efforts, which most often had little time and personnel" (Terzaghi 1959). Time lag between collection of data and use by the designer, leads to undue delay. The analysis to check reliability of data requires time and experience based on judgement. The layman's classifications to laboratory tests and performance of structures, present many contradictions. Pruning of the data or rechecking, though obligatory, is rarely done.

Laboratory CH soil could be in situ layered clay with alternate sand strata or altogether expansive clay below water table which has different in-situ behaviour. The range of shear and compressibility parameters, based on standard investigation specifications, irrespective of subsoil, create more confusion rather than clarity unless redundant or irrelevant results are discarded. Drainage conditions in triaxial testing, SPT, density from UDS or SPT, vane shear in layered or in moist sandy clay, etc. need much closer scrutiny. Expansive potential of the clay met with, in a soil report does not mean that subsoil does not need treatment, unless subsoil is below water table or has equilibrium moisture. Swelling potentials have misguided designers to treat even swollen deposits, or deep moist deposits with no access for moisture.

Although universal standard specifications have evolved by consulting, firms do not consider soil at site. Thus bulk of redundant data above and below the stressed zone, though useless, is inevitable. The site investigation as per IS1982 is summarized in Tables 1 to 3. It covers objectives, applications, design parameters, procedures, extent, depth, type and approximate cost for comparison. The area of 14000 m^2 with four layered deposit is considered here for comparative studies.

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 $q_s = f(R_d - \phi - d_f - B - L)$, safe bearing pressure for allowable settlement, SBP is function of R_d, B, D/B and modulus E.

Tables 4 and 5 detail statistical data of practices adopted by different consulting agencies for exploration. This data is based on tenders. These practices are compared with theoretical requirements. Data of practices for selecting test sample per bore or a sample representing 6 to 7 m depth, in four layer profile, are unknown. This data does not ensure the selection of appropriate representative samples. Normal practice of filling tables evenly, leads to random selection of soil samples for critical tests by investigator. It is clear from Table 4 that 15 to 83 blows SPT results per layer and 5 to 8 undisturbed samples per layer, will have to be classified by selecting two samples per bore. The soil structure is identified by 2 tests, shear parameters evaluated by 2 to 3 tests, shear and oedometer tests on 1 to 2 samples per bore. For a layer,

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 $4.5 \times B$ or $1.5 \times L$. For $B = 3$ m explore 14 m 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. $\left(N_s - p_o' - N_s'' - R_d - \phi - E - q_{p40}\right)$ • $\phi = 0$ Soil UDS @ 1.5 m interval in stress zone, 20% points replace by in situ vane test if soil is soft, sensitive • $C - \phi$ soil suitable combination of above to obtain critical parameters (Table 2) • 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

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

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.

Table 5
Analysis of Lab. Tests Prescribed by Tender (Project Serial Numbers same as Table 4)

Sr. No	Classification			Soil Structure*	Shear Tests Triaxial + UCC No./Bore	Consolidation Tests No./Bore	CBR-OMC-MDD No./Project
	Usual		Special				
	No./Bore	Depth m/sample	No./bore				
01	03	4.5	-	01	01+00	01	02
02	01	10	0.5	10	01+01	01	-
03	02	20	01	-	01+01	02	01
04	03	06	01	06	04+02	02	20/10
05	04	03	2.5	05	02+02	02	03
06	15	0.5	2.5	01	60+04	01	-
07	4.5	4.5	0.8	4.2	0.5+1.4	1.5	40/50
Range	2-15	3-20	0.5-2.5	01-10	T: 01-04 U: 01-02	01- 02	02 - 50
Theory	06	01-02	02	02	03-04	02-03	10

* i.e. Dry density, moisture etc.

Notes:
1. Table assumes 4 layered subsoil profile 50% cohesive, 50% non-cohesive.
2. Can soil profile based on one sample per 3 m depth be representative? Field classification by layman is usually misguiding.
3. The range in most of the practices shows false economy of testing in laboratory.
4. If top layer of 2 meters is unsuitable for founding structure and stress zone is 4 m, the samples tested in laboratory in zone 2 to 6 m depth gives adequate significant data only by chance. Depending on range of results, one or two tests can give uneconomical or unsafe design.

decisions therefore will be mostly guided by SPT. The wide range of results of SPT, shear strength and compressibility for a layer provides a challenge of selecting design parameters truly representing subsoil in action. Thus usefulness of exploration depends on the person who executes exploration in field and laboratory. Statistics do not help.

The judgement based on experience and contact with site conditions can only provide a parameter which is linked to factor of safety. Lower range adopted is with F.S. = 1.5 but upper bound data may require F.S. = 2.5. This is rarely, done, thereby introducing uncertainty in designs.

This report based on experience of the author's practice is presented. Thus review, at later stage forces a designer to ignore report in cases of contradictions, and adopt oversafe, uneconomical solution by art or adopting successful foundations. Such a decision becomes a prestige issue later on. The general problems and limitations related to explorations are listed in Table 6.

Table 6
Comments on Exploration

1.0	For surface, shallow or deep foundation, 2/3 of the collected details will be redundant.
2.0	The subsoil and loads are unknown. a common tailor made exploration for all types of probable foundations is based on alternate SPT and UDS or vane schedule. This may provide SPT in clay (ϕ = 0) or UDS in non-cohesive sands. Such data has to be pruned during interpretations. Thus significant information for specific stressed sub-soil may very from 0 to 100%.
3.0	Vane and UDS in layered alluvium needs scrutiny as it could misguide designer.
4.0	Table 5 presents wide range in practice compared to recommendation of code. The depth for most of the cases is 15 to 20 m for drilling against required range of 7 to 9 m for fairly good number of sites. Number of SPT and UDS are showing wide range 11 to 15 and 4 to 6 against normal 5 to 6 per bore. The UDS per layer varies from 5 to 81 against normal 6 numbers.
5.0	Most specifications indicate very small numbers of samples for laboratory tests (Table 5) Classification tests are for samples every 3 to 20 meters and special tests are 0.5 to 2.5 per bore. The density and moisture are available at 5 m interval in most of the cases. Important shear tests are 1 to 2 per bore – not even one per layer. The range of field and laboratory data for each bore, analysed for layer, will provide wide range with enough contradictions making final selection of single design parameter difficult. Adopting lowest is non-engineering. There is no scientific approach to reduce range for a layer.
6.0	The data analysed presents chaotic practice which has led to crisis, loss of faith and low reliability. Design ignoring soil reports led to professional's ego and prestige issues. The outcome is increasing trends to legal redresses and manipulative interpretations to justify design conceived before exploration.

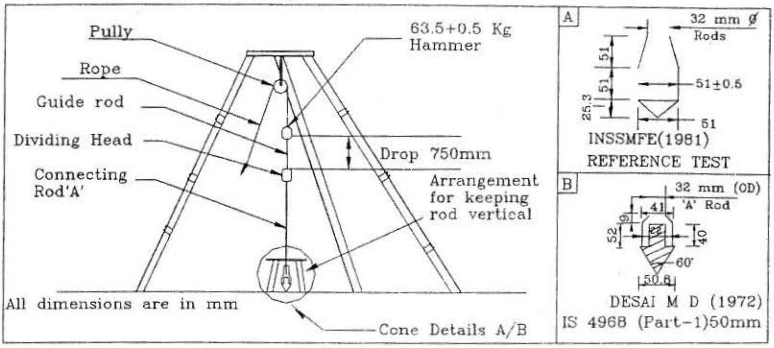


FIGURE 1 : Set-up for Dynamic Cone Penetrometer (DCP) Test showing Cones as per TC 16

A Critical Appraisal – Subsurface Exploration

Having critically analyzed problems and prevalent practices, a need for acceptable alternatives is examined. The approach based on extensive experience only can play a dominant role to rebuild faith in geotechnology. The following are possible avenues.

Pilot Exploration :The quick pilot exploration is first run in 2 to 3 days by DCPT. The data is interpreted for preliminary foundation system. Then the stressed zone, so assessed is explored by bore holes at the rate of one or two per zone, for obtaining critical information and samples for testing to arrive at the design parameters.

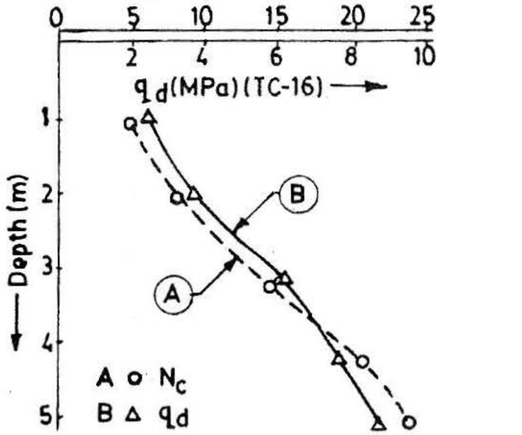


FIGURE 2 : Comparison of Trends of N_c and q_d for a site

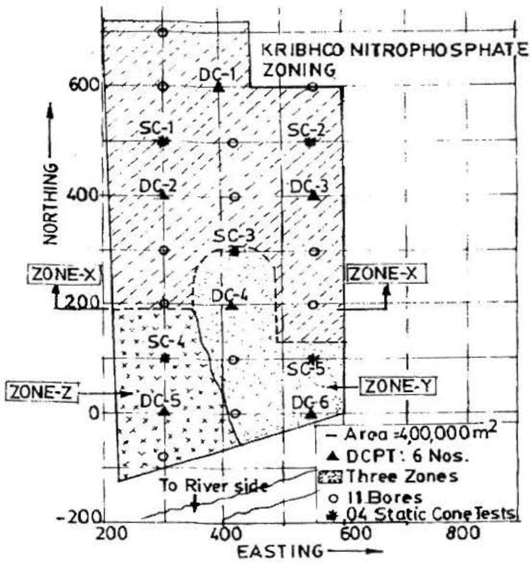


FIGURE 3 : Zoning Plan by DCPT to Plan Detail Exploration (Desai, 1982)

To minimize limitations there is a need to forecast type of subsoil layers and obtain structural details for foundation design. The author used 51 mm – 60° uncased dynamic cone penetration test (IS: 4968 Part-I) as a sounding tool. This cone and assembly are shown in Fig. 1. The recommended DPSH test by INSSMFE (TC-16) is also shown in the above figure. This enables to carry out a similar test. Fig. 2 shows the results of N_c blows per 30 cm or q_d plotted against depth (reduced level). The DCPT and DPSH have comparable results of N_c with depth (Desai, 1982).

Single zone is assumed if results show similarity. In large areas, 200 m grid points are used to divide plan area into zones with similar N_c – depth profiles. The grid is selectively narrowed to delineate boundary between the zones. The typical zoning derived for Nitrophosphate plant of KRIBHCO is shown in Fig. 3.

Identification of Profile : For each of the zones a pilot soil profile is projected as shown in Fig. 4. R is ratio of change in N_c per meter or slope of N_c – depth plot (Pratima and Desai, 1989). R = 0 to 8 shows saturated to moist-wet fissured clayey soil. R = 8 to 20 indicates cohesive silts and normally consolidated clays and R more than 20 is non-cohesive sandy soils. Thus number of layers, thickness and its probable behaviour is evolved from pilot DCP tests. This interpretation by regular calibration, provides a code for a region.

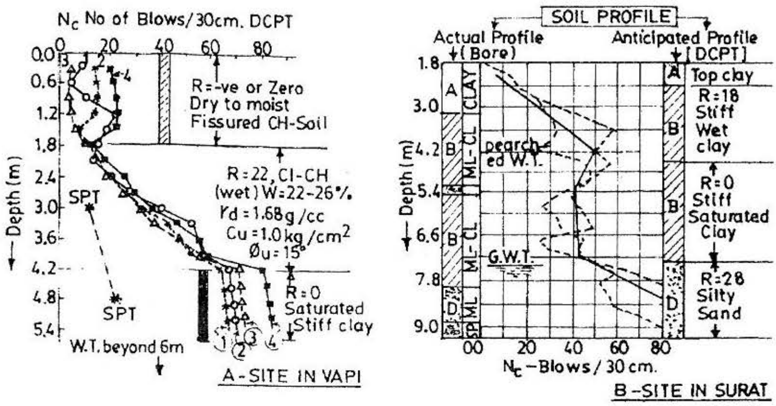


FIGURE 4 : Soil Profiles Developed by DCPT

Water Table Location : The depth to ground water table is predicted as per Fig. 5. Normally soil above water table has a sharp drop in N_c up to 40 percent and the same soil below water table shows sharp increase. Figure 4 shows even perched water table. The prediction is fairly accurate in medium to loose sands. The decrease of N_c at water table varies with denseness and over burden (Desai, 1970).

Properties of cohesive soils : Most of the soils with R = 2 to 4, nearer to the surface, could be desiccated fissured clods of expansive CH soil. The soils with R = 0 to 2, are saturated cohesive mass. The critical parameter $C_u = 0.83\ N_c\ t/m^2$ for depths up to 5 m ($N_c < N_s$) and $E_u = 75\ N_c\ t/m^2$ are suggested as first approximation. (Pratima, 1991). For higher N_c ($N_s < N_c$),

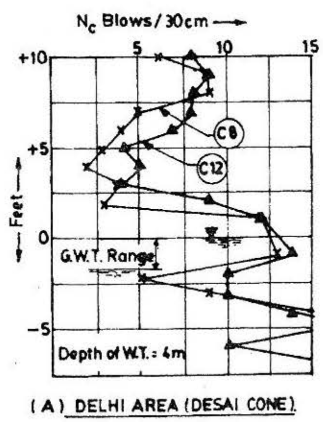


FIGURE 5 : Predicted and Actual Water Table

stiff clays $C_u = N_c$ but not more than 12 t/m^2 is used. E_u is obtained by the above approximation.

For cohesionless soil, the N_c with corresponding effective surcharge pressure is used to estimate relative density, liquefaction potential, safe bearing pressure for a settlement of 25 mm and angle of shearing resistance using the information detailed in Fig. 6. Modulus of elasticity for different non-cohesive soils can be directly read from Fig. 7. CBR can be evaluated using Fig. 8 (Desai and Desai, 1979).

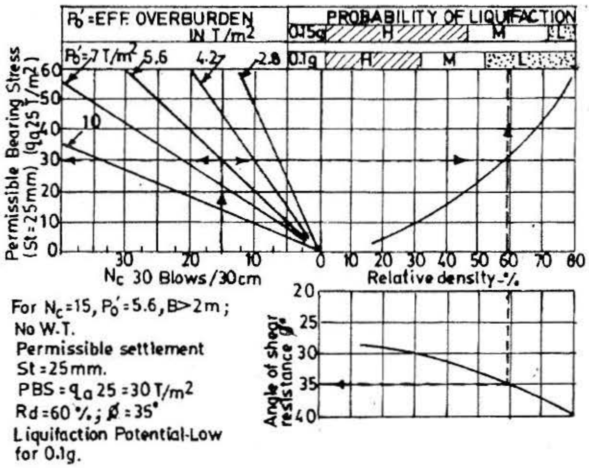


FIGURE 6 : Engineering Properties of Non-cohesive Subsoil by DCPT

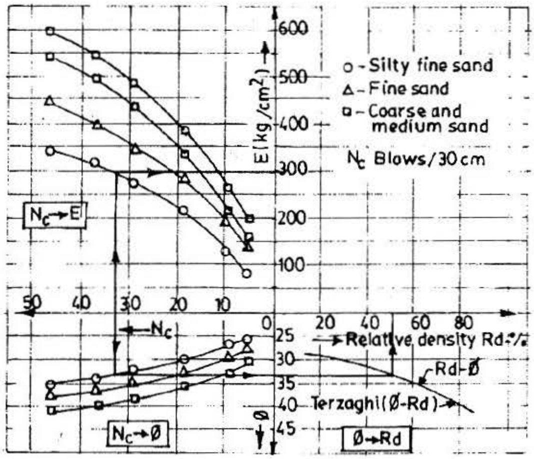


FIGURE 7 : Interpretation of DCP N_c for $Rd - \phi - E$ for Different Sands

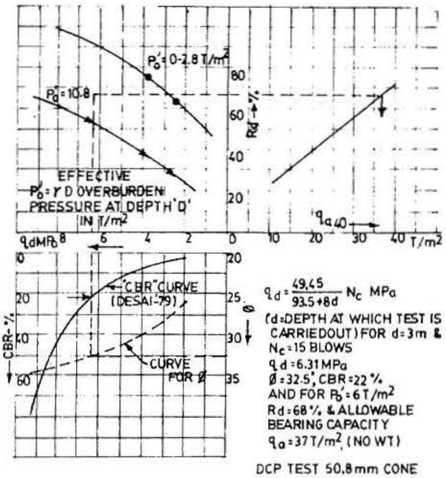


FIGURE 8 : Engineering Properties of Non-cohesive Soils Including CBR by DPSH

On the other hand the engineering properties of fine grained soils is difficult to assess only on the basis of check tests and actual boring data unless it is backed up by local experience and judgement. In this regard the advantage of DCP over SPT is illustrated for Kandla soil in Fig. 9 (Desai, 1990). The soft clay up to 12 m shown by SPT could be subdivided into very soft clay up to 3.6 m, normal clay in the layer 3.6 to 9 m and over consolidated clay beyond 9 m. In the case of layered deposits field observations mainly guide the course of action.

Critical samples are selected on the basis of consistency inferred from

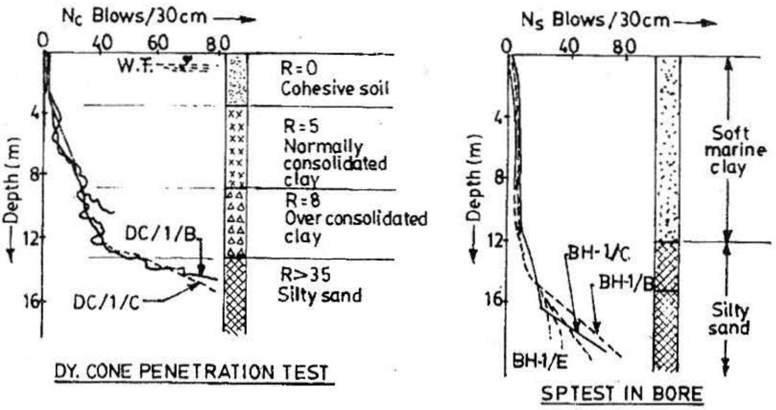


FIGURE 9 : Comparison of Soil Profile by DCP and SPT at Kandla Site

N_c. This approach provides a small number of critical appropriate samples making precise testing, with personal supervision, feasible. The parameters obtained by DCP are then modified, if required, to finalize design. In the case of borderline problems of footings, raft or pile, precise evaluation of variable parameters is done by special exploration adopting appropriate tests such as plate load test, pressuremeter, static cone (CPT), nuclear density depth probe and such other techniques. Still there would be situations where actual prototype tests had to be carried out (Desai, 1970).

Advantages of Above Approach : The above approach, involving pilot survey, requires 3-4 days for sounding, one day for analysis and three weeks for detailed exploration by bores. The exploration, to confirm critical parameters on selected few samples, around lowest N_c in layer, reduces test time considerably. In addition to this cross check, special explorations can be planned if sensitive parameters, interpreted by pilot and detailed exploration are inconsistent. Even load test on large model or prototype has been resorted to, to arrive at logical solution.

This approach is less expensive and can be executed more scientifically in less than half the time. Eliminating irrelevant data and cross checks on predicted design parameters, adoption of the method detailed improves reliability. For the same cost more DCPT can be conducted and hence better coverage is possible. The input leading to variable personal interpretations are also minimized. The set up is low capital intensive unit and job is practically unskilled. It provides continuous data. A model soil profile based on DCPT, moderated by bore data and special exploration by CPT (Static cone) is illustrated in Fig. 10. It incorporates field observations as well. Selecting parameters for the worst state (lowest mean N_c), a lower factor of safety is recommended. The reduced exploration cost is Rs. 0.18 to 0.24 per

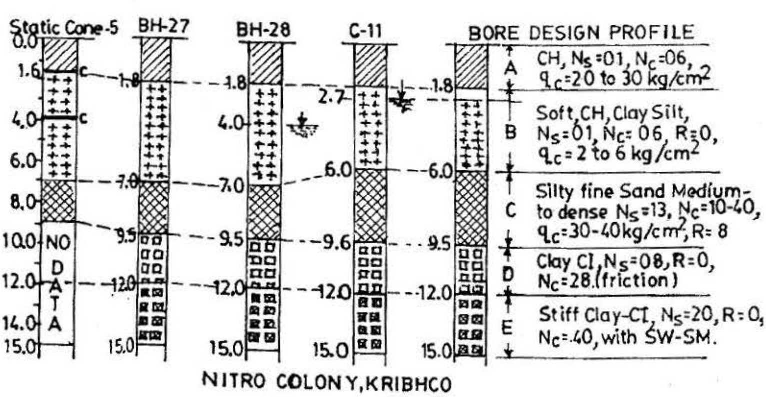


FIGURE 10 : Design Soil Profile Evolved by DCPT Modified by Bores and other Tests

Table 7
Prediction and Performance of Tank on Sand near Delhi (1990).

Dairy site (1990), Capacity = 10000 T, Diameter = 30 m Explored by 4 bores, 2 DCP, Ns – Observed SPT, N_s'' Corrected N_s

Meters	Depth log Soil	N_s (P_o' t/m^2)	N_s''		Mean N_s''	St Max. stress 16 t/m^2 mm	Safe bearing pressure St = 40mm (t/Sq.m.)	Estimated E (Desai)	Remarks
			IS Code	Desai					
00	Filled up	14	—	—	—	—	—	—	—
1.5	SM ML LOW PI	(03)	—	—	—	—	—	—	conservative 50% reduction for WT. is adopted
4.0	W.T Silty Sand W = 13, t_0 22%	11 (08)	14	25	14 General Practice	Using IS 8009 and WT Correction 64	q_{a40} = 08	3640	* Mean N_s'' for Depth = width of foundation by Desai
6.0	Fine Sand SW	15 (12)	12	27	26 (Desai)	38	q_{a40} = 15	4100	** E adopted in soil report = 1150 t/m^2
20	Fine Sand SP								
30	W = 7 to 16%	22 (20)	16.5	26				4700	

NOTE: For 40 mm settlement q_a = SE / 0.7 × Hs, S = 0.04, H_s = 9 m, q_a = 7 t/m^2 (Report)

$$\text{Mean } N_s'' = \frac{14 \times 6 + 12 \times 4 + 16.5 \times 10}{20} = 14$$ for IS and mean N_s'' by Desai = 26

Mean E = 3900 t/m^2.

cubic meter. Thus this approach reduces time of exploration as well as cost. It is more reliable and consistent.

Disputes on Design : In spite of the above approach, designer or client may still disagree with the recommendations as it may not confirm with their intuitive or Topo design. In such a case, report based on writer's approach has a better sustainability for technical arbitration or legal battle.

Case Studies of Subsurface Exploration

Observations presented are based on number of investigations and review of reports for which second opinion of the author was sought. Only typical cases are presented.

Tank around Delhi

The records of exploration are summarized in Table 7 showing soil profile, N_s – observed SPT, safe bearing capacity, as per prevailing practices. For settlement of S_t = 100 mm, SBP varied from 20 to 36 t/m^2. The exploration report recommended a value of SBP 16 t/m^2.

The incorrect designation of sand as "Loose" interpretion of N_s, incorrectly, substitution of B (value and unit) corrected for surcharge, not considering average N_s'' for stressed zone, adoption of submergence factor when N_s is below water table, are spelled out in Table 8. Though soil is classified as sandy silt (field) the tests indicate it as silty sand.

Table 8
Analysis of Soil Report for Tank Near Delhi

The report appendix gave following expression for St = 100 mm:

$$q_a = 0.553(N-3)\frac{(B+30)^2}{4B^2} \times R'_w$$

This itself is incorrect. The correct expression for St = 100 mm is

$$q_a = 3.5(N-3)\frac{(B+0.3)^2}{4B^2} \times R'_w$$

where B is in m.

Substitution of B is width or diameter of tank in meters, The value of B = 450 cm adopted against 30m diameter is questionable.

Value is of N is corrected N_s''. It is taken at foundation level. The average N_s for the subsoil in stress zone is not considered by many which is incorrect.

The E value reported is arbitrary. Such assumption could be mistaken as manipulation to obtain specific answer of settlement.

Table 9
Prediction and Performances of Tank Delhi

Data: Diam. = 30 m, Max Contact Stress = 16 t/m^2, u = 0.3, Influence Factor for Edge = 0.64, E Variable Width Depth = 1800 to 3900 t/m^2.

Agency	Soil report	N_s''= 14 (IS)		N_s''= 26 (MDD)		Based on E	
		A1	A2	B1	B2	C1	C2
Theory	$St = \frac{0.7 \times q \times B}{E}$	—	—	N_s'' $- IS4009$	N_s' $- q_{a40}$	$St = \frac{q\,B\left(1-u^2\right)\times 0.64}{E}$ $= 9.2 \times 10^3\ B/E$ mm	
Parameters							
B m	9	30	30	30	30	12	30
E t/m^2	1150 (Adhoc)	—	—	—	—	1800	3900 (Table 7)
Predicted St mm	88 say 100	64	80	38.4	43	61.3	140
$\frac{\text{St (Predicted)}}{\text{St (Actual)}}$	0.83	0.54	0.68	0.32	0.36	0.52	1.19

A1 : Based on N_s'' – St as per IS Code.

A2 : $N_s'' - B - q_{a40}$ charts.

All Settlements are doubled for submergence consideration as per practice

Table 10
Time Rate (Edge) Settlement of Tank during Hydro-testing – Delhi

Day	Stress applied t/m^2	Settlement		Remarks
		P_1 mm	P_5 mm	
14/05/90	2	20	20	Loading started on 12/05/90
18/05/90	4	62	38	Wind on 16/5/90 observed
20/05/90	8	84	60	Average settlement = 118 mm
22/05/90	12	105	70	Differential Settlement = 55 mm
01/06/90	16	138	90	
06/06/90	16	138	88	

Notes:
1. Considering average settlement of 118 mm, B = 30 m dia, Mean Foundation Modulus E in-situ works out as 3100 t/m^2 against 1800 to 3900 t/m^2 estimated by different practices
2. Tilt of 24 mm caused by heavy winds on 16/5 increased differential settlement.
3. Incorrect applications and interpretations affects economy or safety hence survival of GE at end of 20^{th} century.
4. Permissible differential settlement criteria are very conservative
5. 5 to 8 mm settlement per t/m^2 stress and rate of settlement indicate foundation behaviour of sand and not silty sand.

The computed settlements, by IS code using $N_s'' = 14$, $N_s'' = 26$ and predicted E considering compressible strata of 9 meters (report) (B = 30 m) are compiled in Table 9. The estimated settlement ranged from 38 to 140 mm as against actual observed average edge settlement of 118 mm. The approaches using $N_s'' = 14 - qa_{40}$ (IS Code) and E based on N_s'' (Desai, 1980), with compressible zone equal to diameter of the tank, predicted values closest to actual. The trends of settlement with time for two points (maximum and minimum) are shown in Table 10. The settlement against hydrostatic stress and differential settlement observed are given in Fig. 11. The differential settlement is 50% of total settlement. This case illustrates wide gap in practices strictly adhering to codes. Such feed-backs of prototype help improved soil modeling and interpretations. The in-situ prototype test gave E = 3200 t/m^2 against predicted E = 3900 t/m^2 (Desai, 1980) and E = 1140 t/m^2 of soil report. This case depicts usual practices are unsafe in predicting settlement in most of the cases. The depth of compressible zone in settlement analysis plays significant role. To conclude, the need of creating awareness even in the qualified professionals and to update practices, proper uses of codes and analysis of performances must get priority over the R&D to arrest possible erosion of faith in geotechnical engineering.

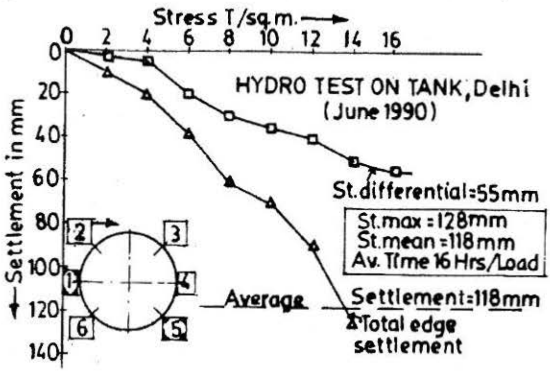
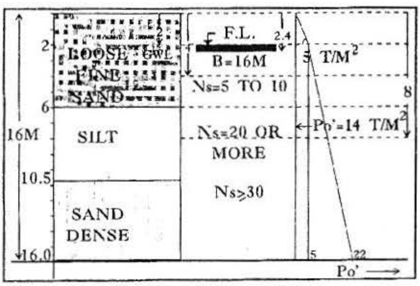


FIGURE 11 : Stress Vs. Settlement (St) of Tank near Delhi

Case Study of ESR around Ghaziabad

Terzaghi emphasized that Soil Mechanics is a guide to judgement, which could be obtained only by years of experience with field observations and realization of the limitations of exploration and theories.

Table 11
Foundation for Tank and Housing Complex – Ghaziabad



The Soil data is summarized here. G.W. is 2 m below GL in lo se fine sand layer

Foundation for Tank and Housing

Water table at 2 m
Loose sand ($N_s < 7$)
Footing infeasible

Permissible St = 50 mm,
SBP = 9 t/m^2 at 2.4 m
below GL for 16 m dia raft

Piles 9 m deep
resting on dense
sand recommended

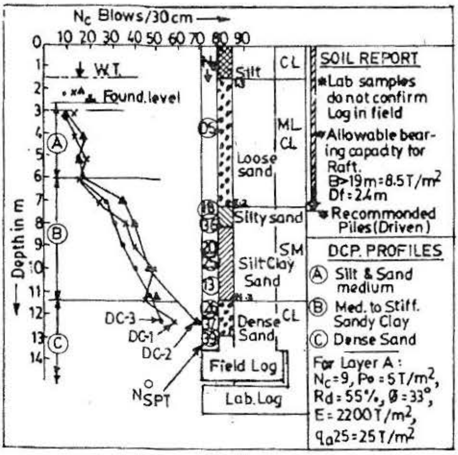


FIGURE 12 : Subsoil Characteristics for Ghaziabad Site for Buildings and Tanks

The relevant data from a soil report for a Ghaziabad site is compiled in Table 11 and Fig. 12. The analysis of SPT – Soil profile showed allowable bearing capacity of 8.5 t/m^2 for a raft foundation. The report therefore recommended 8 to 9 m deep driven piles as the foundation system. For the same data, the writer based on Table 12, obtained allowable bearing capacity of 20 t/m^2 and considered shallow foundation at depth of 2.2 m as technically feasible. The experience and judgement of ISBT and Yamuna Barrage at Delhi as well as other publications (Desai, 1970, 1972) formed the basis for such an opinion.

The casual approach or lack of self confidence or use of geotechnical prescription for other than technical reasons, as discussed earlier, could result in erroneous decisions.

Terzaghi's statement is valid for many such cases. In all such problems, the interpretation and recommendations cannot ignore ground improvement feasibility and check tests (e.g. in-situ density to verify looseness and corresponding N_s at 2.0 m depth) to evolve not only safe but cost effective solutions. Even prototype test is justified as precedent for piling, and if adopted, can influence future course of decision in the region.

Project in Rajasthan

For site Y, a preliminary exploration for a mega project was carried out by agency (say X). Subsequently turnkey job contractor (say Z) carried out detailed explorations. The contractor's designs were disputed by project

Table 12
Writers Review of Data-based Experience – Ghaziabad

Analysis for raft

Mean $Ns_{(o)}$ for 16m dia. Raft

$$= \frac{6\times6+4.5\times20+5.5\times30}{16}$$

$= 18$ Blows / 30cm

P_o' mean $= 14$ t/m^2,

$N_s'' = Cn \times Ns_{(o)} = 27$

SBP for St = 50 mm = 33t/m^2
on safe side w.t. Correction applied
SBP = 2/3 × 33 = 22 t/m^2

Smaller raft feasible.

Analysis for Footing

Depth = 2.2 m, wt = Dw = 2 m

N_s average = 7 for 2.2 to 6.0m (Table 11)

$N_s'' = [35/5+7]\times N_s$ is 20

Sand is medium dense

Rd > 50%, 0 = 33°, E = 900 t/m^2

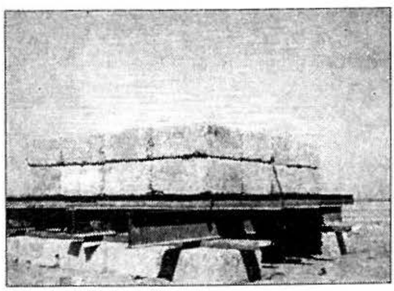
PBS = q_{p40} = 20 or more t/m^2

$\Downarrow$

Footings are also feasible.

RECOMENDATIONS

1. Conformatory 4 DCP test and a plate load test.
2. Test for 2 × 2 m prototype to 100T load or prototype load test.
3. Experience at ISBT, Yamaha Barrage and other publications (Desai '70, '72) Confirms Feasibility of Shallow foundations.



consultants based on soil consultant's (X) preliminary report. Such instances brought out are intended to avoid repetitions and alert the professionals against accepting recommendations of soil report on its face value. Structural consultant, guided by soil experts or vice verse, ultimately reaches to a point of no return ultimately resulting in clash of egos. These types of problems referred to court or arbitration, result in loss of years to avail benefits of mega projects.

The case studies with such predicament reminds one of Terzaghi's statement: "Soil mechanics is supplement to and not substitute for common sense combined with knowledge acquired by experience". Engineer has to use his judgement despite being fully conversant with principles of soil mechanics. Besides these, even non-geotechnical reasons such as errors in reduced levels (Fig. 13) subsequently corrected (Fig. 14) matter considerably in practice. The resulting influence on depth and consequently resulting economically feasible foundation system, is obvious as shown (Fig. 13). Raw data with over safe

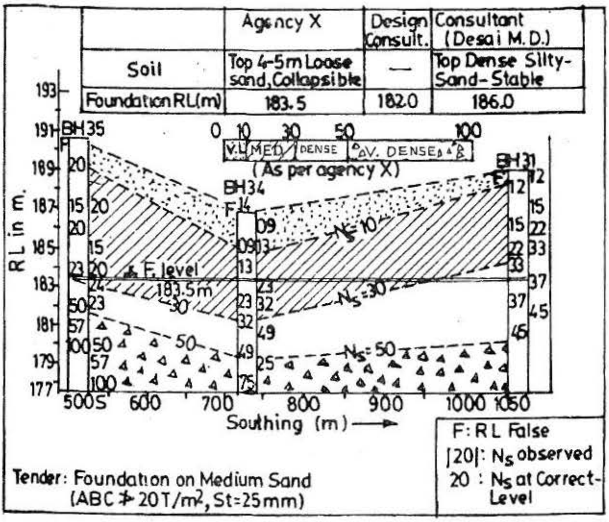


FIGURE 13 : Soil Profile based on Preliminary Exploration of Party X, Project Y with Errors in Levels.

interpretation, misled the consultant to place foundation at RL 183.5 (Fig. 13). The corrected levels and analysis of relative density by author indicate that foundation can be placed at RL 186 (i.e. 2.5 m above).

Detailed exploration and direct analysis : The contractor's detailed exploration data and analysis for density using $N_s - P_o' - Rd$ are shown in

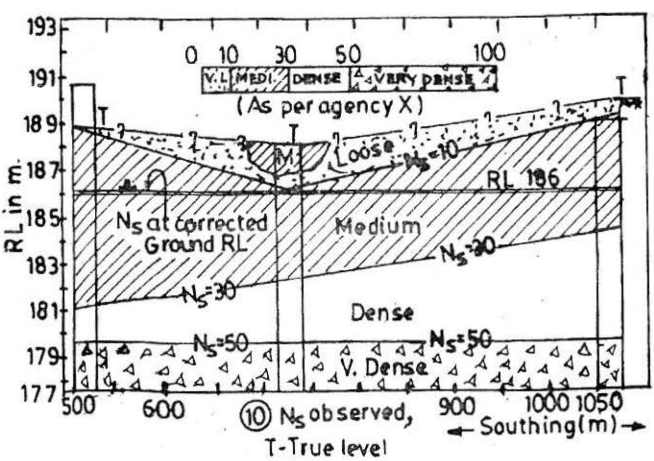


FIGURE 14 : Soil Profile based on Data of X Corrected for Levels and Review of N_s Project Y

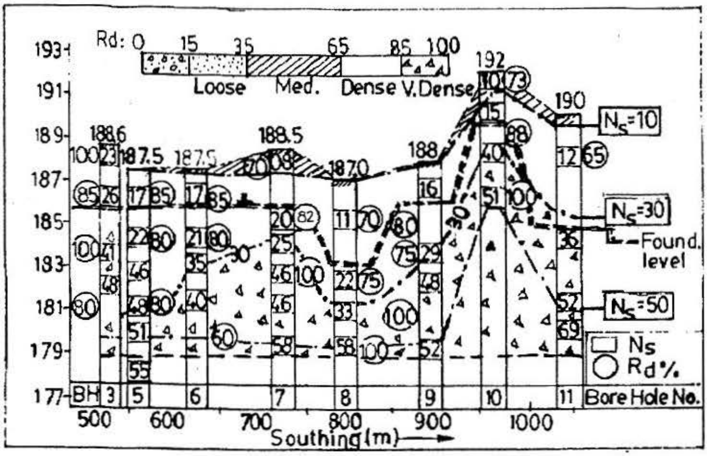


FIGURE 15 : Soil and Rd Profile, Data of Detailed Exploration by Party Z, Project Y

Fig. 15. The entire misleading top loose layer of Fig. 14 vanished. At depth beyond one meter, the sand exhibits 70% or more relative density. The turn key tender specifies foundation to be placed on medium dense (R_d = 30%) sand adopting maximum safe bearing capacity of 20 t/m^2. Writer recommended a foundation level of RL 186 (2 m below ground level). If indirect criteria of N_s is adopted, the foundation levels recommended are as shown in Fig. 15.

Need for strong common sense and scrutiny of data : As explained, if N_s between 9 and 14 in one of the bore holes was considered unreliable and discarded or rechecked, the phobia of loose sand would not have crept in. Relative density of 70 to 85% could not have been indicated as 15 to 35%. The density reported in the same soil report also does not confirm looseness. The obsession that sand is collapsible – aeoline of Rajasthan was never cross checked. On the contrary, report of a poor analysis, attempted to justify the obsession.

Wrong classification by misquotation

The sand can be wrongly classified as loose deposit at deeper levels. This could be a pre-exploration obsession either based on literature or lack of confidence. The report wrongly cited works of Alam Singh et al. (1986) to justify that sand is collapsible. Slightly higher settlement in a plate test in flooded condition and decrease of N_s in water filled deep bores have been often cited in support of classifying sand as collapsible.

Citing improper references and misinterpreting data are quite common to soil reports. This practice is used to accommodate assumption a designer

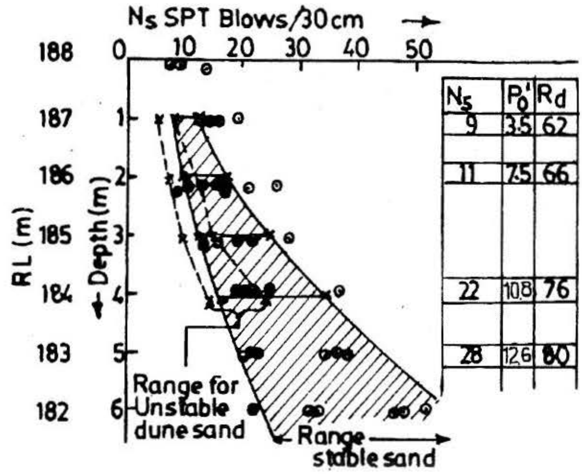


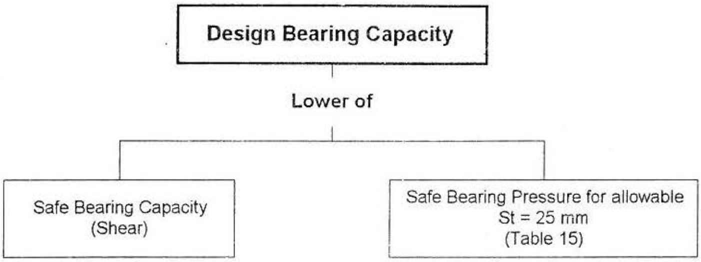
FIGURE 16 : Classification of Desert Sand into Stable, Unstable Groups. Data of Party X and Project Y

has made prior to exploration. This type of activities are most disgusting and damaging for profession. The work of Alam Singh on sand of Rajasthan is presented in Fig. 16. The data of N_s versus depth for site by both the explorations are shown therein. Only one point falls in unstable zone, 6 of

Table 13
Interpretation of Denseness of Sand Project Y (Rajasthan)

Tender specifications: Foundation on Medium Dense Sand	
PARTY – X PROJECT CONSULTANT	DESAI M.D. BASED ON EXPERIENCE
Errors in levels introduced wrong soil profile (Fig. 13)	R L checked corrected (Fig. 14)
Instead of rechecking/rejecting isolated value N_s 9 below N_s 14, it was given undue weightage in top layer	Lowest N_s = 9 ignored and checked by additional bores by party Z (Fig. 15)
Use of outdated (1948) Ns – Rd correlation by ignorance or desire to confirm committed opinion	Analysis data of other tests DCP plate and UDS for checking Rd (Fig. 17)
Reduction in Ns in flooded bore wrongly attributed to collapsible structure @ 8 m depth. In fact given Rd, N_s of dry sand is always more than submerged sand (P_o' effect)	Use of well known internationally accepted $N_s - P_o' - Rd$ (Desai '92) indicate sand has Rd minimum 65% @ 1 m below ground level. Dense sand can not be loose and hence collapsible.
More settlement in plate test on flooding (2 to 4 mm) do not indicate collapsible structure.	Proposed foundation level for SBP 20 t/m^2 is R.L. 185-186 m against project proposal of R.L. 182 m

Table 14
Design Safe Bearing Capacity : Project Y



R.L. (m)	Depth (m)	Data	R.L.	Avg. N_s	Min. N_s
188		Foundation Level	184	22.4	15
	03	3m × 3m Footing	182	38.2	22
185		Design N_s mean = 23	181	38.8	28
	37	P_o' mean = 11 t/m^2	178	more than 50	50
148		St permissible = 25 mm			
		Water Table			

Parameter	Party X	Writer	Plate test	DCPT
Rd (%)	35	70	65-80	67 -76
ϕ^o	32°, $\phi' = 23$	36°	—	32° (N_c = 17, P_o' = 5)
Shear Type	Local	General	General	General
Factor of safty	2.5	2.5	2.0	2.5
Net SBC t/m^2	19.4	128	more than 60	74 (Desai '92)

26 in overlap zone and 19 in stable zone. This reference does not prove unquestionably that sand is collapsible. Even the physical properties (percent of $CaCo_3$, silt content and moisture) did not conform to sand for which above chart has been evolved. As explained in Table 13, load – settlement curve of dry sand and flooded sand do not indicate collapse. The decrease in N_s on submergence is due to reduced surcharge (P_o') for same relative density and not because subsoil is collapsible on wetting.

Arbitration

In all such situations where interest of contractors and clients are clashing, geotechnical engineers have a formidable task to resolve the issue. The matter can then be resolved by court or by arbitration outside the court.

Increases of such cases delay the project and escalate prices and keep community deprived of the facility. The summarized disputes regarding depth of foundation referred to arbitration are shown in Table 14. Figure 17 shows data of exploration by four different tests to confirm that sand is dense.

Limits of Conservatism

There are no limits to conservatism in practical situations. Conservatism often covers ignorance, degrades self confidence and capacity to resist pressure to obtain specific recommendations.

For the above project data, net safe bearings capacity evolved by party X, author's interpretation of plate test and DCPT are shown in Table 14. The

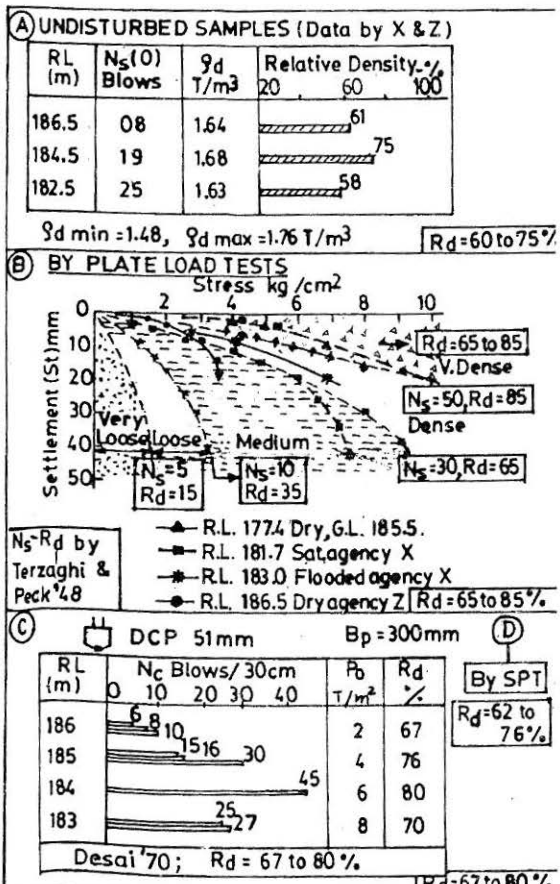


FIGURE 17 : Confirmation of Rd of Desert Sand by Sampling Plate Load, DCPT and SPT – Project Y

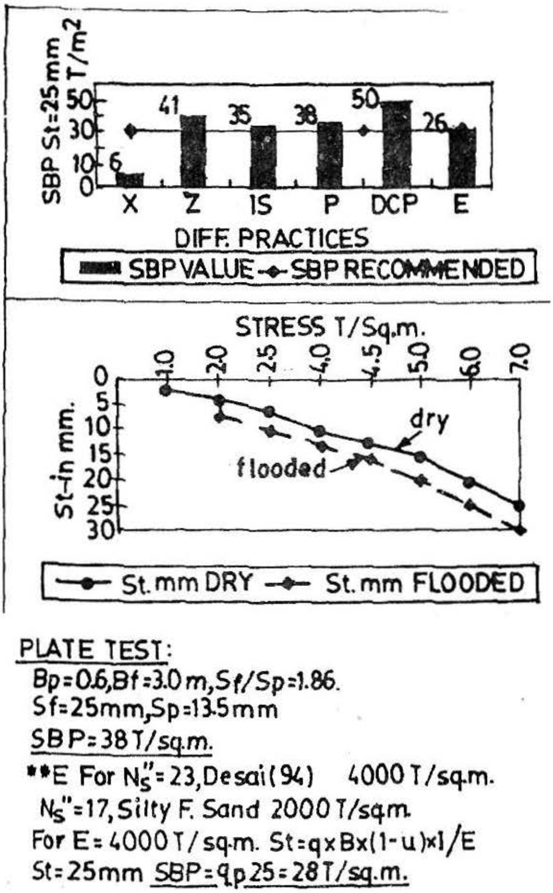


FIGURE 18 : SBP for St = 25 mm Obtained by Different Practices Project Y [Agencies of Exploration X, Z. Interpretation by I.S. Plate (P), DCP and using Modulus E]

range is 19.4 to 60 t/m^2. The SBC (lowest) was reduced to 10 t/m^2 by applying accidental flooding, in desert, by project consultant. Recommendations of party is inconsistent with safe bearing capacity by other methods and implies that shear governs design of footings on sand. Using geotechnical engineering as a tool it is common to justify poor, uneconomical and unusual designs or ground treatments. Ignorance can be corrected by education but misuse with knowledge cannot be corrected.

Use of flooding under seismic condition even with 1% probability and for minimum N_s at base (not of bulb), adopting lowest bearing capacity factors (local shear) with lowest ϕ, lowest shape factors irrespective of shape, highest factor of safety have been total improper geotechnical practices.

Table.15
Net Allowable Bearing Capacity : Project Y

SBC from Table 14, 60 t/m^2		As per tender Max. 20 t/m^2		SBP for St = 25 mm
SBP for 25 mm settlement by different practice				Remarks
Parameters	Party X and consultant	Party Z and M D Desai	Party X & Z IS code	
Design Ns blows/30cm N_s'' corrected for P_o'	Min. at F.L. 15 15	Avg. of min. 23 *45	Avg. 23 23	$*Cn = \frac{35}{P_o'}$ P_o' in t/m^2
Settlement for stress 20 t/m^2 (St_1)	40	12	24	
St_2	40	12	14.4	Settlement corrected for depth and rigidity
St_3	80	12	14.4	Settlement corrected for accidental flooding and structure of sand
SBP t/m^2	06	41	34.7	From plate test = 38 t/m^2 (Fig. 18) from DCPT: 50 t/m^2

The safe bearing pressure (SBP) is normally derived for 40 mm settlement. For more rigid, 25 mm settlement criteria for assessment is adopted. The SBP thus deduced ranges from 6 to 50 t/m^2. The data is shown in Fig. 18 and Table 15.

Can a soil engineer with common sense ever accept SBP of 6 t/m^2 in medium to dense sand at depth of 6 to 8 m below GL? Can reduced SBP take care of collapsible subsoil? Such practices and inhibition's of not taking design bearing capacity more than 20 t/m^2 even if soil exploration justifies, are trends, if not arrested, will prove Pre-Terzaghi art of foundation is more economical. The design over and above stipulates concrete apron on either side of the plant to prevent direct flooding. The rejection of the detailed exportation and tender condition in favor of preliminary data cannot be explained. Degree of conservatism of such practice will endanger existence of profession.

Economical Aspects of Foundation System

Present Trends : The economy has a basic requirement of capital for development. Civil engineers have a social responsibility of management of human and financial resources for the benefit of mankind. Thus optimum use of financial resources and building materials such as cement and steel have been emphasized. Geotechnical engineering has an important role to play in this endeavor.

Validity of practices based on cost analysis : Economic feasibility is an ever changing phenomenon. Hence foundation practices logically based on economics, are not valid for eternity. The statement that raft is economical if sum of the areas of individual footings exceeds 50% of plinth area (Terzaghi and Peck, 1967) is widely adopted even today. For housing complex at Hazira, for allowable bearing capacity of 10 $t/m^{2,}$ the footing area covered is 70% of plinth area. Thus raft was recommended. The economic evaluation by author showed that the cost of footings at a depth of 3.5 m was Rs. 575.00 per sq.m. The RCC raft at 1.2 m depth cost Rs. 770.00 per sq.m. Recommended system of Geogrid reinforced sand pad and footings was estimated as Rs. 460.00 per sq.m. Thus such generalized statement cannot be true in all cases at all times, particularly with availability of several alternatives.

Concept of Safety at All Costs : For foundations, unlike structures, there is an increasing trend to provide safety at any cost. The fear psychosis has been spread by an exploration that non-performing foundation, leading to shut down for even few days, could be more costly than additional investment in foundation. Thus expensive, so called safest system can be justified. The professional crisis, by experts, exploration agencies and consultants, has been availed to increase conservatism. Thus economical considerations always do not play any dominant role.

Foundation Design as Art : The degree of conservatism touched intolerable limits in the last decade. The art of foundation existing in pre-Terzaghi era is revived by many consulting engineers. The topo culture, adopting successful foundations for similar structures in the region, is growing. Such repeat successful designs need not be always safe. The designer is ignorant of risks involved. The practice now emerging is to arbitrarily adopt a foundation system. The soil exploration is then planned such that final report, by hook or crook, confirms the arbitrary design assumptions. Thus exploration is used as tool for a negative role.

This is non-engineering and unscientific retrogression. Even with the limitations of exploration, profession cannot afford to revert back to 1948 era. The consulting engineers must be educated about (a) waste of scarce resources of money and materials (b) unknown risk (c) more construction time and delayed benefits (d) non-scientific approach (e) loss of benefits of modern technology of ground improvement.

To illustrate the need for strict economic assessment, etc. typical case studies are discussed. They are few of many such cases.

Foundation for Light Towers

The rapid expansion of microwave telecom and electrification of railways brought out the need for type designs for towers of different heights and weights.- light, heavy, and very heavy. The R&D group at Roorkee, RDSO Lucknow and Mast Fabricators and others have developed ready design tables.

Design of 40 m high telecom tower at Olpad was referred to author as site engineer suspected that the safe bearing capacity to be less than 5 t/m^2, adopted using design tables. Such standard tables are used widely in the last decade. All such tables are based on design bearing capacity of 5 t/m^2 for submerged and 10 t/m^2 for dry soil at foundation level. Such designs ignore type and variation of subsoil, properties of soil exhibiting swelling and collapsible character, environment, source, depth, etc. Low bearing capacity is no guarantee for safe foundations.

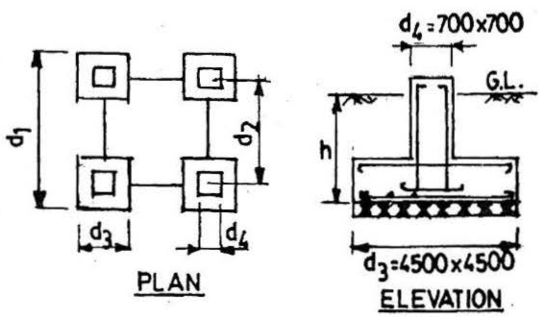
Users adopt such tables without digesting logic and rationale and add their own factor of safety. As such economics of such foundations are questionable. Some of the cases are worse than art era of foundation engineering. Cost of foundations of light towers is 20 to 40% of total cost towers (Subramanian and Vasanti, 1990). They are quickly adopted by departments, design organizations and top consultants for following reasons:

(a) Easy to adopt,
(b) Carries authentication of reputed organization,
(c) Pays more for least efforts i.e., percent cost as fees, and
(d) Ignorance of engineering is concealed.

Responsibility of exploration and analysis is shelved. The question is at what cost and unknown risk. Is reverting back by 50 years justified today?

The Olpad subsoil was explored by the author. The top 1.2 m CH expansive (DFI = 86%) soil is followed by saturated CI – CH soil up to 2.7 m (WT at 1.3 m). The soil below foundation is stiff clay, NBC = PL, Cu = 6 to 10 t/m^2, Dry density = 1.7 t/m^3, DPI = 35%. The safe bearing capacity was 20 t/m^2. Figure 19 shows tower built as per design by author (SVR design). Foundation for a 40 m tower, evolved from type design is shown in Fig. 20. Using same notations, designs of Advanced Mast by author are shown in Fig. 21. The author considered part of the moment as countered by passive pressure. The maximum stresses are 14 t/m^2 and minimum being 0.7 t/m^2 (tensile). The financial implications are analyzed and presented in Table 16

FIGURE 19 : Microwave Tower on SVR Foundations Olpad



SBP – TEN TON/SQ.METRE (DRY SOIL)

Tower Height	Light weight Tower			Heavy weight Tower		
	d_1	d_2	Wt. (M.T.)	d_1	d_2	Wt. (M.T.)
40 Mtr.	11170	6370	15.85	11742	6042	23.20

• All dimensions are in MM.

FIGURE 20 : Dimension of Foundation for Transmission Tower using Standard Table (SERC, Roorkee)

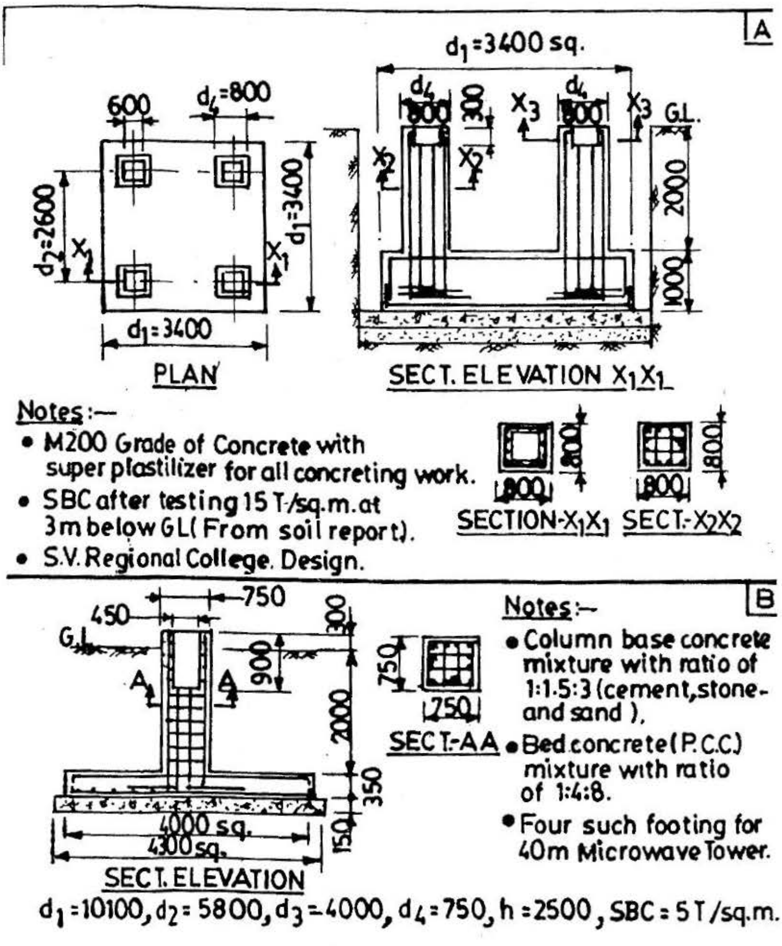


FIGURE 21 : Foundation Design of Olpad Tower based on A) SVR, B) Rodio Mast Fabricator, Hyderabad

Table 16
Comparison of Costs for Foundation of Tower

Type	d1	d2	d3	d4	h	Concrete	Steel	Risk
R&D Table	11170	6370	4500	700	3475	145	5.3	Unknown
SVR Design	4000	2000	-	800	3000	17	1.7	None
Private practice	10100	5800	4300	750	2650	27	24	Unknown Tilt

Notation as per Fig. 20 All sizes in mm, RCC is 1:1 1/2 3 mix

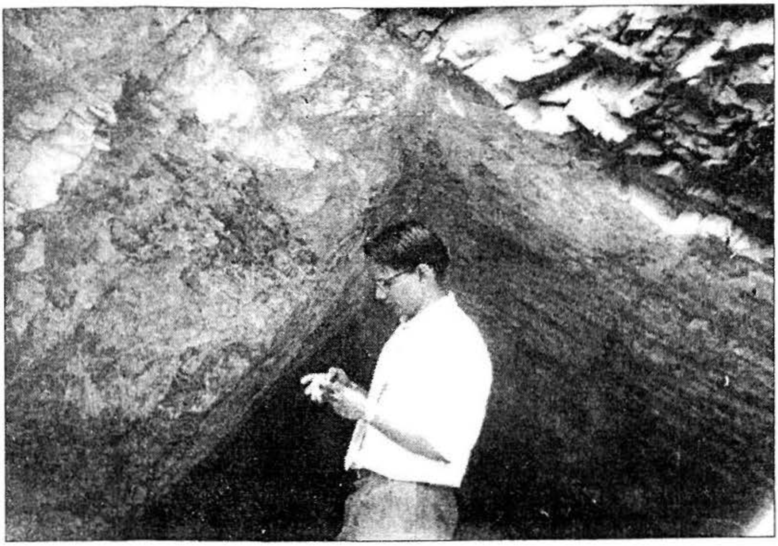


FIGURE 22 : Investigation of Difficulty Subsoil by Drift (Salal Dam)

Ultimate savings, in the present context of prices and scarcity of resources of the order of Rs. 4.7 lakhs with no unknown risk, is not by any means meager. Considering number of such projects being in thousands per annum savings in capital and materials cannot be ignored.

All safe performing foundation systems cannot be ideal models for others in the close proximity. A sound system at a given time takes into account (a) Observations of existing structures (b) Present day technology (c) Economics (d) Time factor (e) Reliable and local skill/technology available (f) Environment and other constraints. Unknown risks are large if repeat designs are adopted without any critical examination, exploration of substratum and consideration of changes in the technology over years.

Conventional sampling in practice, laboratory testing for many sites, particularly with layered deposits, gave misleading classification and hence the predicted behaviour. The parameters evolved by exploration are unsafe or over-safe and often contradictory. The jointed, disintegrated rock at Salal dam, Fig. 22, in a drift is a typical example. The zero core recovery and slow rate of drilling never indicated fractured rock until a drift was inspected. Appropriate in-situ tests unless planned knowing the local geology make analysis of laboratory and field tests unrealistic. Contradictions in field and laboratory data are not easy to reconcile.

Unless the data is scrutinized, digested, filtered by judgement based on regional experience, practical safe as well as economical design parameters

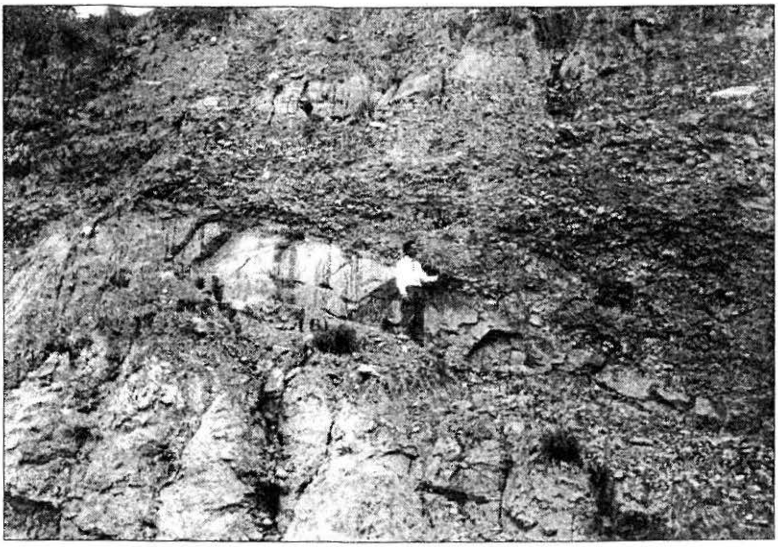


FIGURE 23 : Difficult Terrain Defying Ususal Extrapolation of Profile (Ghaggar Dam)

cannot be evolved. Knowledge of environment, geology and performance of structure assist in the above process. This is rarely done today. In many cases soil report follows design, leaving little scope to recheck design parameters by prototype tests.

Tank Site Hazira

Normally complex substrata are stratified jointed, desiccated talus and can be even sensitive deposits. Even with best sampling techniques samples do not retain structure and true grain size character of in-situ stratum. A cut in subsoil at Ghaagar Dam, with variable talus over compact silt illustrated in Fig. 23 is an example. The interpolated profile from drilling data and displayed by actual cut are different. Fig. 24 is yet another case of difficult soil for sampling at Hazira in Gujarat.

Foundation system for steel tank 2000 m^3 capacity at Hazira is illustrated. Most of the tanks in this region are founded on 1 m thick RCC cap over 50 to 80 numbers of 17 m long piles. Piles are 410 mm diameter bored, cast-insitu with a safe load capacity of 60 T. Driven 300 × 300 mm pre cast piles are also used in specific cases. This pile system was recommended for a proposed 16 m diameter tank by design consultants. Soil exploration shows clay in liquid consistency in 2.5 to 6.5 m zone in soil report. Thus a deep foundation was logically adopted. The exorbitant cost and short duration available for execution necessitated reference to author for an alternative.

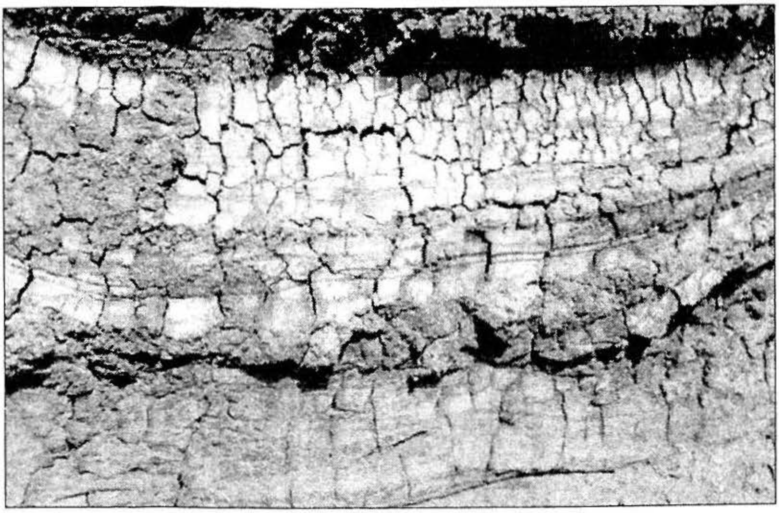


FIGURE 24 : Deep Desiccated Soil Difficult to Sample – Hazira

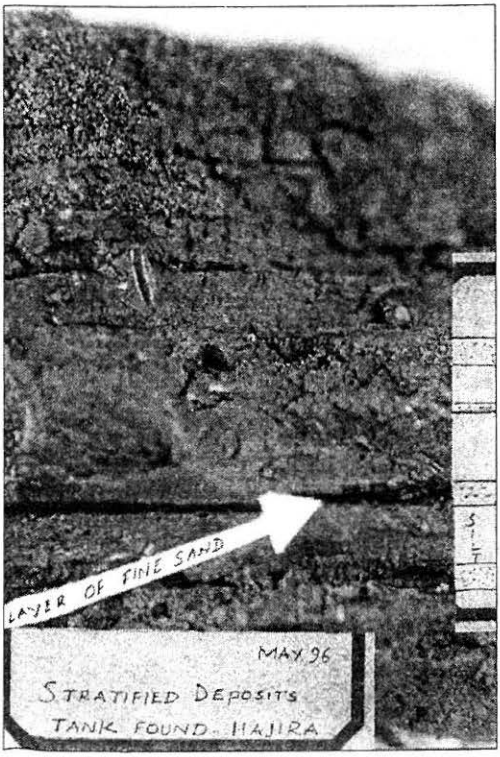


FIGURE 25 : Stratified Deposits at Site for Tank Foundation – Hazira

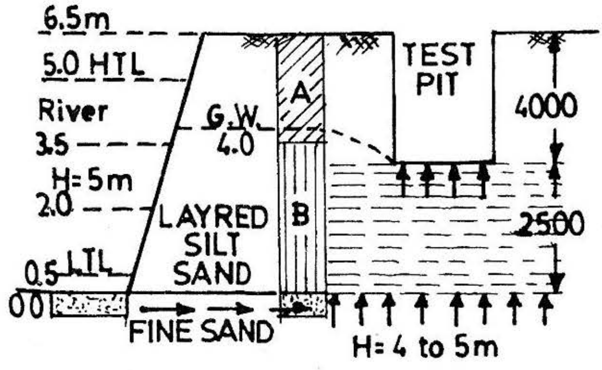


FIGURE 26 : Hydraulic State of Stress – Tank Site at Hajira

The available exploration substratum data shows N_s less than 2, low static cone resistance (q_e), clay at liquid consistency (w_L = 50%) for layer 2.5 to 7 m with clay content of 70 percent CI – CH type NMC = 53 percent, Dry density = 1.3 g/cc, SI = 22%.

The author reinvestigated the site. The properties of clay in-situ are Liquid limit 68 to 74%, Plastic limit 32%, Liquidity index 0.54 for water content 50%, C_u > 3 t/m^2 and E = 300 t/m^2 (triaxial), 760 t/m^2, (DCPT N_c = 9, P_o' = 8), 1300 t/m^2 (plate load test), 100 t/m^2 (SPT N_s = 2 to 3).

The open pit inspection shows dilatant silt in plastic to semisolid consistency, high salt content with high horizontal permeability. Uncased bore 8 m deep was stable for a week. The photo of block sample is shown in Fig. 25. The open pit for sampling Fig. 26, is subject to tidal pressure, which would damage the structure by increasing moisture content. Receding tides leache colloids. SPT in bore gave low Ns due to hydrostatic seepage pressure shown. Quick changes in layers makes CPT unreliable. Vane in such thin stratified deposits is also misleading.

According to DCP test N_c increases form 4 to 13. Dry density is 1.34 g/cc, water content 40%, e_1 = 1.1. For field drained condition C = 0, ϕ = 29°, E = 800 t/m^2. C_u = 4.5 t/m^2 (min.), 5 t/m^2 by DCPT, 9 t/m^2 by Vane, 6 t/m^2 by triaxial. The salient investigation records are shown in Fig. 27.

Design of the Foundation : The stress analysis shows stress of 8.5 t/m^2 on a 9 m diameter sand pad. Even ignoring stiffness of geogrid reinforcement average stress on compressible clay zone will be 6.7 t/m^2. This change of stress with overburden pressure of 4 t/m^2, m_v = 4 × 10^3 m^2/t, H = 4.7 m may cause settlement of 50 mm. The permissible settlement of 100 mm will not be exceeded. The time rate

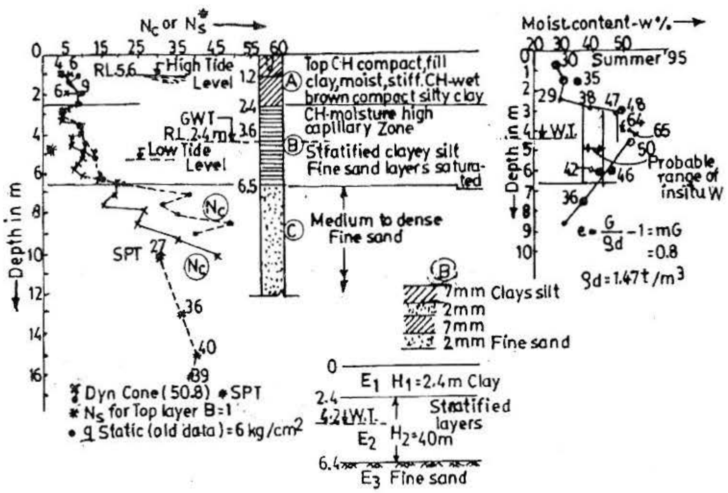


FIGURE 27 : Compilation of Soil Exploration Data – Tank Site Hajira

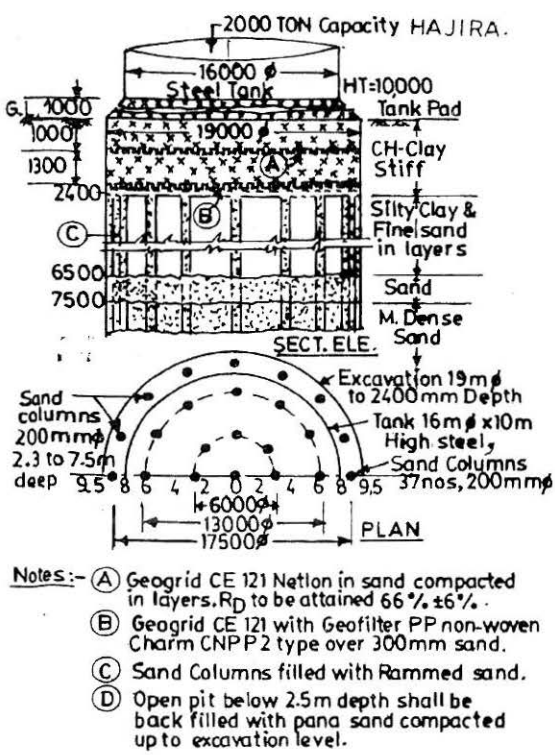


FIGURE 28 : Proposed Foundation for Tank at Hajira

settlement with C_v = 2.2 m^2/year, H = 2.35 m (two way drained) indicates 2 years time for 90% settlement. The field experience and judgement indicate settlement will be over in 60 days. Considering undrained condition, for C_u = 4 t/m^2 (min.) ultimate bearing capacity is more than 24 t/m^2. Thus the factor of safety against shear failure is 24 / 6.7 = 3.6. This foundation system with 9 m geogrid reinforced sand, as shown in Fig. 28, was recommended. Geofilter at base for control of colloidal washout and rammed sand columns, to remove even remote chances of locked up pore pressure in clay pockets, have been incorporated.

Construction: Sand columns were executed from ground and finished 2.5 m below the ground. The work was over in a week. The excavation to 2.5 m for laying of geofilter, geogrids and back fill of compacted sand was completed in the next four weeks. Thus foundation work was completed in the time stipulated as in piling contract.

Advantage : This design based on field observations and calculated risk not only saved time for construction but also reduced cost by 40% as compared to rigid pile foundation system. The settlement is limited to maximum 100 mm with time rate being reduced considerably. Number of such tanks in the country on marine deposits cannot afford to ignore cost aspects and related savings in capital and time

Foundation for Very Light Structure

As discussed earlier, overall deep exploration generally neglects top 2 to 3 m from testing. This layer is casually described in field notes as top fill, loose fill or black soil, etc. Many structures in projects like dike walls, treatment plants or tank, require foundations at very shallow depths. In the absence of proper data over safe designs are provided assuming collapsible or expansive subsoil. A typical design adopted for dike wall around Hajira is described here. The maximum load did not exceed 4 t/m length.

The following are the details of substratum generated in 1993.

0 to 4 m	CH – wet, very stiff (N_s 10 to 20). (q_c = 20 kg/cm^2)
4 m	Water table.
4.5 to 6 m	loose or soft layered silt and sand (q_c = 8 kg/cm^2), liquid consistency.
6.6 to 10 m	SW-SM non-plastic sand, N_s = 22, (q_c = 60 kg/cm^2)

For a recheck, author in 1995 preferred reexploration to give second opinion. The subsoil data revealed the following details.

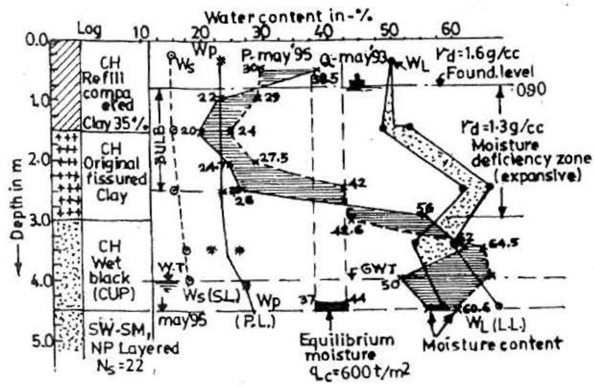


FIGURE 29 : Subsoil Data of Foundation for Dyke Wall – Hajira

0 to 1.5 m	compact CH fill (5 years old)
1.5 to 3.0 m	CH moisture deficient (w = 24%) moderately expansive soil.
3.6 to 4.5 m	CH moisture more than equilibrium moisture (37 to 40%) practically in swollen state.

The subsoil profile is shown in Fig. 29.

The subsoil is clay with clay content of 40%, w_L = 60%, I_p = 32 to 39%. Activity = 0.9. Soil is of highly swelling according to US Navy Manual (ΔV = 30%) DFS = 36 to 50% increasing with depth (ΔV = 20%), SL > 16 non critical in swelling, Heaving strain is in the range of 3 to 4% for P_o' = 6 t/m^2, Swelling zone is 1.6 m below foundation. Heave is 45 mm, Swelling pressure by IS (remoulded) test is 4 to 5 t/m^2. Plate test and field observation do not confirm liquid state of subsoil. C_u is 4 to 7 t/m^2.

Analysis: Though the subsoil exhibits swell potential, the analysis of exploration shows that it is not critical, for the environment prevalent, at this site. The soft or loose soil insitu is stratified deposits of silt and sand subjected to cyclic tidal hydraulic pressure from sand layer as shown in Fig. 29. Such a state disturbed SPT, static cone resistance and moisture content considerably. The release of stress due to exploration further aggravated it. Judgement based on observations and results of DCP tests permit assumption of insitu moisture of 40% and corresponding shear and compressibility parameters. This case permits calculated risk in foundation design.

The typical design of dyke wall with load intensity of 2 t/m^2 is shown in Fig. 30(a). Two meter wide strip with double undreamed piles extended up

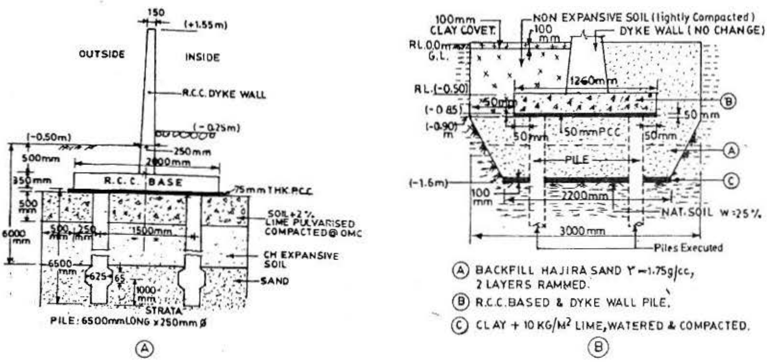


FIGURE 30 : (A) Conventional Design of Foundation; (B) Revisions by Writer during Execution for Dyke Wall Foundation – Hajira

to 6.5 m below, are used as foundation. A 500 mm CNS layer is prescribed below the RCC base strip.

This design shows 10 t/m^2 swell thrust against load of 3 t/m^2. Even the piles, having clay in a state close to its liquid limit above the bulb, will not provide effective resistance to this upthrust. The CNS. layer was made up of local CH Soil, pulverized and mixed with 2% lime. Such a process was not feasible in monsoon. This constraint led to the review of the problem by the author. As discussed, though soil has high expansive potential, the in-situ moisture at equilibrium reduces heave zone. The expected heave is 40 mm or less. The clay below 3 m is actually not in liquid state as shown by earlier exploration. Moderate C_u = 4 t/m^2 exclude the possibility of shear failure of foundation of Dyke wall. Differential heave may tilt wall, as area on one side would generally be covered or paved or planted with trees.

This revision, using locally available silty fine sand, with lime layer at clay and an impervious backfill, was recommended as shown in Fig. 30. A modified design eliminating piles, using similar system as shown in Fig. 30, was recommended for new works. For floor of the pump house, use of geogrid as separator between sand fill and metal cushion was incorporated to replace CNS layer.

This case study highlights need to consider environment, and state of soil even if the soil is highly plastic and expansive. Providing a wide strip and under-reamed piles do not always make for safe foundation for every light structure. General exploration is totally inadequate for expansive or filled up top clay. The length of dyke walls, walls for effluent treatment plant and pipe supports, etc. at the national level, will be million meters/ year. Even with a saving of Rs. 1000.00 per m as shown, yearly national saving is hundred crore. For shallow foundations for such structures, safe bearing capacity is really not a problem. Proper ground treatment offers economical foundation system with calculated risk depending on type of disaster.

It can be concluded that the over-safe designs and adoption of prevailing practices, have impact on national economy which is not negligible. The exploration of top 2 to 3 m, as in present practice, is inadequate and do not provide data for swell or collapse potential for assessment of deformations of shallow foundations. Therefore, a special exploration to evaluate behaviour has to be planned, and scrutinized and implemented.

Foundations for Culvert and Abutment Walls

For convenience quick working type designs have been evolved by IRC – SP13. It is widely adopted as it bypasses design and approval procedures of department. Though such ready designs have served the country in the past, the impact on economics today necessitates critical review. The design of abutments along Vagra Road is taken as an example.

Table 17
Design of Culverts – Abutments (Vagra Road) (Ref. IRC - SP 13)

DATA

S.No.	RL. of slab bottom (m)	Invert of Drain (m)	H (m)	Span (m)	Design available for H (m)
SC4	9.51	8.05	9.51 − 8.05 + 0.3 = 1.76	6	2.0
SC5	9.74	8.20	1.84	6	2.0
SC8	11.01	8.16	3.15	3	3.5

DESIGN FROM IRC SP 13 TABLE

S.No.	H (m)	b1 (m)	b2 (m)	b3 (m)	b4 (m)	B1 (m)	B2 (m)	D_f (m)	B3 (m)
SC4	2	0.2	0.6	0.4	—	1.5	2.7	1.5	—
SC5	2	0.2	0.6	0.4	—	1.5	2.7	2.0	3.3
SC8	3.5	0.35	1.25	0.3	0.3	2.2	4.25	1.5	—

Table 17 shows data of three culverts. The table is based on allowable bearing capacity (ABC) = 16.5t/m^2, and for lower values stepped widening is recommended by IRC. The design dimension for nearest span and height are adopted from SP13. It is surprising to understand how a national standard design bearing capacity of 16.5 t/m^2 was chosen. This design practice prohibits use of higher allowable bearing capacity. It also does not always culminate in the design safe against swelling, shrinkage, scour or collapse situations. Failures have been recorded. The safety, economy and ground improvement alternatives are ignored. At this juncture it may not be out of place to quote the statement by Terzaghi, "Soil mechanics theories are gross simplification for refinement in mathematical analysis. This often creates confusion than clarification of basic question involved. This may induce false sense of security instead of appreciation of many kinds of risk involved in foundation design". Simplified designs, in practice, in addition to risks, will waste limited financial resources. Such codes need review and revision using computer soft-wares, incorporating cost optimization and modern ground improvement technology.

For example for H = 0.81, span 6 m, Table for H = 2 is adopted in practice. The width B_2 is 2.7 m at 1.5 m depth becomes 3.3 m (B_3) as soil report recommends depth of foundation as 2.0 m. This type of design is compared with computed deign which gave B_2 = 1.9 m. Reduction of concrete of 1.8 m^3 per meter length was possible. For a two lane road (4.5 m wide) and two abutments, saving of 16 m^3 of 1:3:6 concrete is possible. On an average 4 such culverts are built every year in a square kilometer area for road-canals, etc. 100 million cubicmeter concrete could be saved by taking 50% of jobs adopting direct design procedures (3 million square meter). The prevailing prices and cheaper technological options being available do not justify use of such standards.

Vagra soil exploration report, in the absence of structural details, gave minimum net SBC of 12 t/m^2 for clay with an average C_u = 7 t/m^2 for deep CH soil, N_s = 18 (top 1.8 m expansive). Rectangular footing at 2 m depth can provide gross 20 t/m^2 bearing capacity if shape and depth factors are considered with a factor of safety of 2.5. The water table is beyond the influence zone. Soil is wet and ϕ is ignored on the safe side. Mode of analysis has not been imbibed by academia. Standard type design is unsafe or uneconomical.

From the above critical appraisal it can be inferred that SP13 and similar design codes should be replaced by modern software for safe, economical design involving bearing capacity, stepless variations of H and consideration of scour, swelling, settlement, etc. Logically, option for ground treatments could also be incorporated for cost optimization.

Legal Implications in Geotechnical Engineering

The public at large and professional civil engineers in general are of the opinion that analysis and assessment of soil properties is a formidable task and not possible easily for practical purposes. This provides a leverage for legal uses to prove or disprove the facts and exploit it for achieving goals such as victimization, exploitation and such other things. To stay construction activity, create claims of compensation for damages, decide demolition in public interest on grounds of poor performance of foundations, victimize person not falling in line are some examples of misuses. Such decisions in highly competitive environment, have to be supported by logic which can confuse or convince lawyers and the court. For the profession in crisis as explained, large variations in soil properties, interpretations and many decisions based on digested data (using judgement based on experience) provide excellent logic to prove or disprove facts.

This aspect has brought a new cadre of professionals working against its long term interest. The fast growth of cases of disputes in recent years, illustrated in case studies, are typical examples. The objective here is to educate professionals not to leave loose ends in reports, design and to prepare them for a probable defence against challenges forced by socio-economic-political rivalries and races. These cases need not be considered as reflection on any one, even if by coincidence they are similar to some cases.

The following case studies will highlight legal aspects of Geotechnical Engineering

A Project in Cochin

A 20 m high RCC framed structure, built in 1972 was found unsafe by three experts. It was therefore recommended to demolish the structure in public interest. The buildings with visible tilt are shown in Figs. 31, 32, 33. The soil profile for an adjacent site is shown in Fig. 34. Structure is 20 m high consisting of ground plus four RCC frames.

First opinion : On the basis of spot levels within and surrounding the building, a maximum of 560 mm settlement is noticed near corner B, as shown in the sketch. These settlements are 470, 432, 420 mm from surrounding ground level from B to C. Since foundation level was 300 mm above present ground level, when constructed, actual sinking is 720 to 860 mm along north side of building (No data is available for B–A, A–D and G.L). Settlement was recorded at the end of 20 years of its existence.

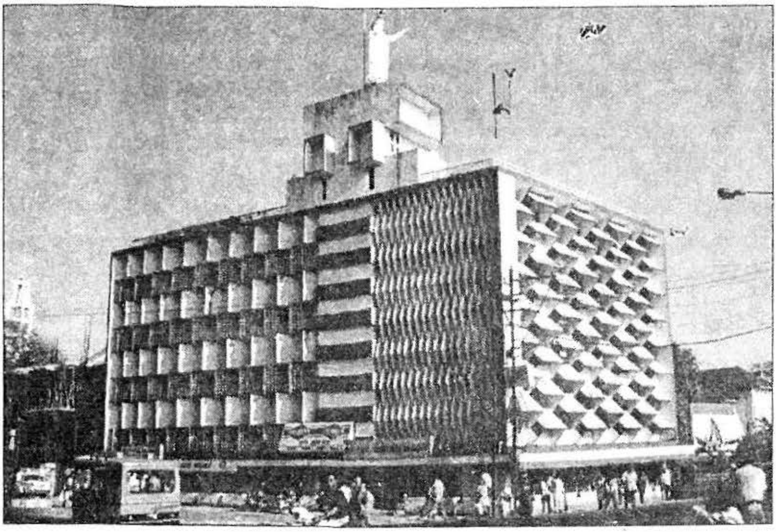


FIGURE 31 : Building showing Tilted Corner – Cochin

FIGURE 32 : North Face of Building Showing Tilt – Cochin

FIGURE 33 : Tilting of Other Building in Cochin

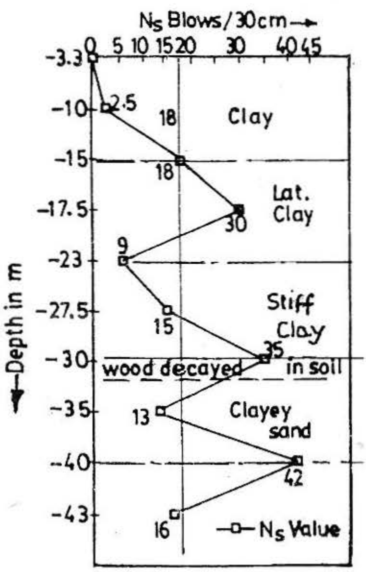


FIGURE 34 : Soil Profile for a Site near Building in Fig. 31

Following analysis was reported.

- Heavy weather protections on north-west, lift and machine at north-west corners and platform with heavy statue contributed to unequal settlement.
- Structure has no visible cracks anywhere so far (No observations of rate of tilt in 1995)
- In 1974 settlement recorded was 345 mm
- The structure in its present form is risky, unsightly and inconvenient to users.
- Even if repaired it is unimportant, unworthy structure at that location.

Second Opinion : Tilting of building is a matter of concern. It might have been caused by weight in the corner and vibrations of traffic. Some single storey buildings in Cochin have also tilted (Fig. 34) This could be due to breakdown of cementation bonding.

- Raft in such soft lightly cemented soil can cause failure of soil to some extent.
- Correcting tilt is possible only at huge cost,
- Sewage waste water lines dislocated caused disturbances to the subsoil.

Note: Even assumption of surrounding ground as bench mark and non-reporting settlements along BA/CD/AD is incorrect and biased presentation.

Third Opinion: Report is summarized as below

- Tilt of 860 mm and progressive settlement are serious problems with foundation
- Subsoil is very loose mixture of sand and clay having poor bearing capacity
- Foundation soil is neither preconsolidated nor provided with coconut piles as per local practice
- Poor structural design with sudden settlement and tilt will shift C.G. beyond stability zone thus endangering public safety.

Note: Affidavit by expert is casual, unscientific, biased and based on presumptions and inhibitions or ignorance of subsoil-structure behaviour. Supporting data collected is poor. Terms tilt – sinking, loose clay subsoil, etc. bringing out that experts are unaware of geotechnology. Experts deliberately bypassed exploration and observations essential to establish their point

Defence for the Owner : If owner does not accept advice to demolish and reconstruct, he can go in for legal protection. The points are :

- 20 m high RCC framed structure built in 1972 rests on a raft. The approximate 16 × 32 m building of this type will generate on the average 12 t/m^2 stress on subsoil.

- The subsoil in absence of data, is assumed as shown in Table 18 and Fig. 34 on the basis of data of adjoining plot.

- Raft as rigid structure has, inspite of displacement and tilt, not shown cracks even after 21 years; shows displacement is tolerable and settles on all the sides.

- The prima-facie predicted settlement of raft foundation is shown in Table 19. This computed and observed data reported by experts, are plotted in Fig. 35.

- The expected theoretical settlement is 456 mm while recorded value is 560 mm in 1993-94. In 1974 settlement was reported as 345 mm (against theoretical value of 197 mm). It is very likely that initial displacement (plastic flow) was high. It has shown equilibrium hence rate of settlement beyond 1995 has been negligible.

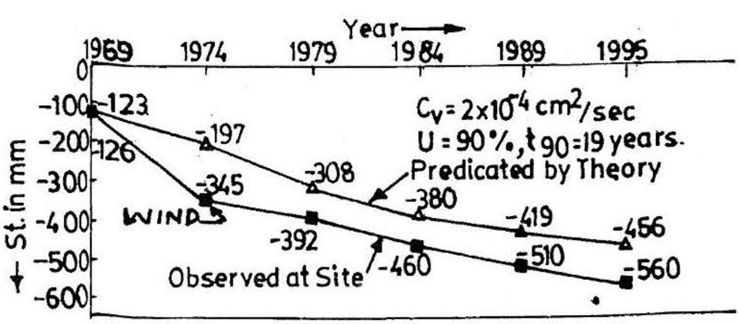


FIGURE 35 : Predicted and Observed Settlements for Building shown in Fig. 31

- Non reporting of displacement on all sides and rate, led experts to opine that building will topple very soon.

Differential settlement : The differential settlement on north face along long side (32 m) is 560 mm at B and 420 mm at C. The differential movement δ, $(560-420)/32000 = 0.0043$ i.e. tilt of 0.23° from vertical is observed. The tilt theoretically becomes visible if differential settlement is 128 mm. The panel wall shows distress when it exceeds 213 mm. With differential settlement of 140 mm no cracks are seen anywhere, the tilt is well within permissible range so far. Future trend shown in Fig. 35 is non critical.

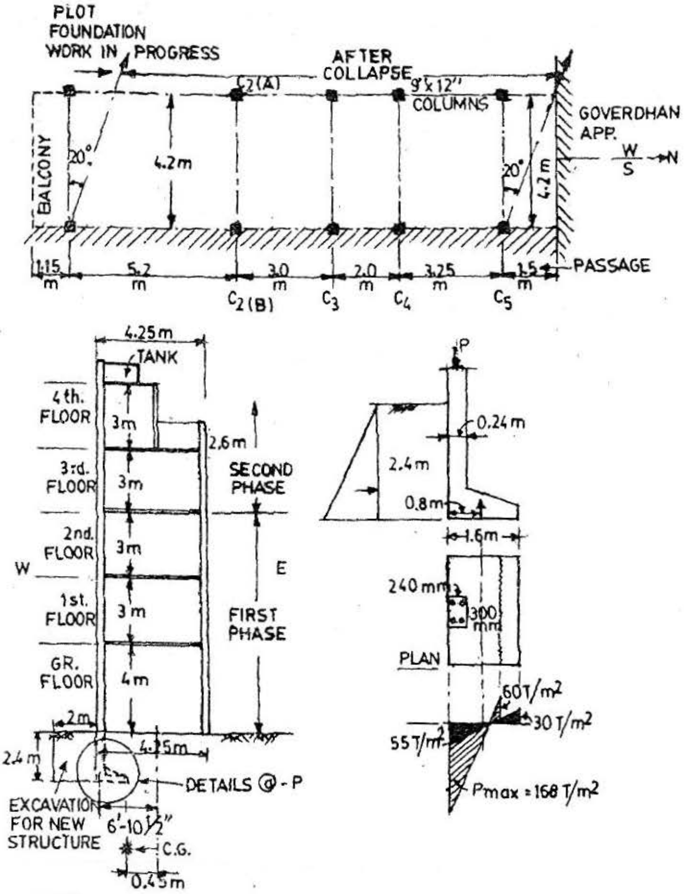
Thus considering tilt of rigid structure on raft, with no distress on structure over years, vibrations (on pre-consolidated 20 years old submerged foundations) will not add to settlement. Theoretical and actual trends of settlement indicate that experts opinions are not sustainable, unbiased and justified. This structure does not pose any threat to public and property nearby as suspected.

It can be concluded that for legal issues, which can be raised at any time, client and designer must have soil exploration data and performance record of displacement at least once in every 2 years. Even right judgement on foundations (undocumented) can be proved biased or vice versa, if foundation data is not preserved. The present practice of not keeping details of foundations, as constructed, needs drastic change. Contradictions, vague settlements, forecasting of toppling of building using terms like tilt, settlement displacement loosely, stating, heavy seepage in floor, etc., without supporting computations on records, makes defence of the case very easy.

Collapse of Building at Surat

A twenty years old four storied structure collapsed suddenly. This raw house structure shown in Fig. 36, has open plot on one side wherein foundation work had just commenced. The two storied structure was built in 1965 and latter extended up to four floors. The collapse occurred on 16.12.90 The collapsed building is shown in Figs. 37, 38 and 39. The loading of stairs and water tank are eccentric. The initial load on column $C_2(A)$ is loaded 8 t more than $C_2(B)$. This extra loading increased by 20 t when 3rd and 4th floor were added. This tilted the structure to west. Center of forces shifted up to a height of 8 m and moved laterally by 0.45 m to west.

The structural steel of 4 bars of 12 mm provided in column 9" × 15", for ultimate load bearing, was inadequate. The redistributed stress shows that factor of safety against shear was just awaiting for an excuse to fail. The excavation for footing in open plot reduced surcharge factor and lateral support of about 10 T. Thus an expected claim of damages from neighbor



NOTES:

1.0 BUILDING WAS CONSTRUCTED IN 1965 COLLAPSED ON 16.12.90 ORIGINALLY 2nd FLOOR EXTENDED LATTER BY 3rd & 4th FLOOR

2.0 INVESTIGATION ON BEHALF OF S.M. CORPORATION PUBLIC SAFETY ASPECT GENERALIZED APPROACH ATTRIBUTED TO EXCAVATION AND CONCRETING OF FOOTINGS FOR ADJOINING PROPERLY (L-FOOTINGS).

3.0 LAYOUT OF BUILDING COLUMNS AND SWAYED POSITION ARE SHOWN IN PLAN.

FIGURE 36 : Plan and Layout of Collapsed Building in Surat

could not be raised. It was proved that structure was unsafe by itself. The frame swayed around column C_2 or C_3 and failed. All west facing columns had lap joints just above the plinth level. 20° rotation and related torsion could be seen from figure.

FIGURE 37 : Collapsed Building in Surat

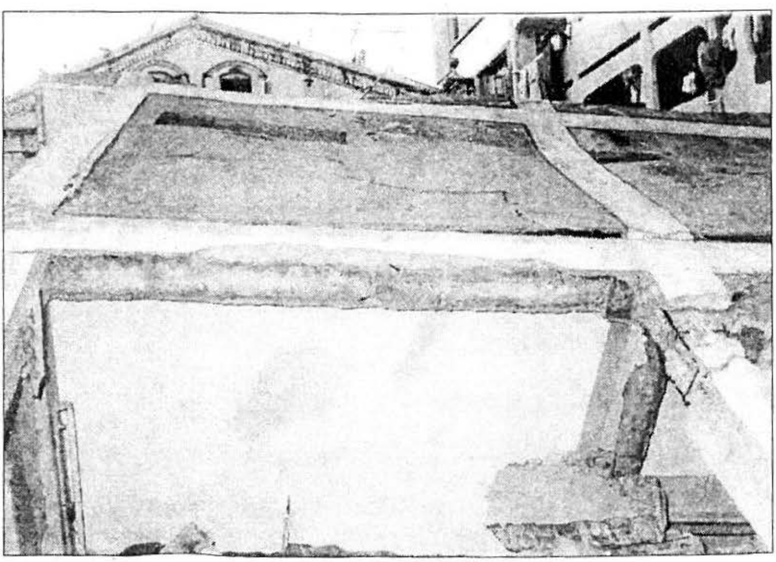


FIGURE 38 : Frame and Wall after Collapse in Surat

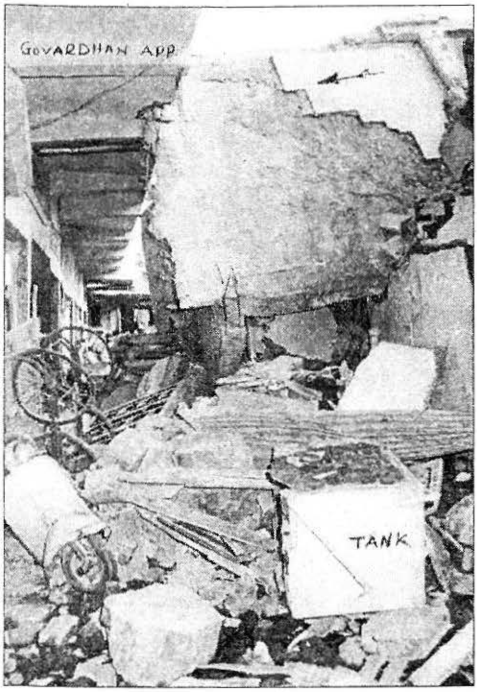


FIGURE 39 : Collapsed Building Damaging Adjacent High-rise Building in Surat

Approximate Analysis : The prima-facie structural analysis data

ITEM	INITIAL (1974)	FINAL (1990)
Structure	G + 2F	G + 4F + Tank
Column loads	50 T	98 T
Area of footing m^2	2.5	2.5
P min. t/m^2	−30	(−)60
P max. t/m^2	+35	(+)168 (see Fig. 36)
Ult. Bearing capacity	80 t/m^2	—
Final effective footing size	0.8 × 1.6 m	—
Average Stress	78/1.2 76.6 t/m^2	$F_s = 1.0$

It is very common to ignore subsoil and foundation details while raising the structure. Such adhoc raising in case of footing must be prohibited by plan approving authority. Excavation in adjoining property line, even below foundation of neighboring structure, in cohesive soil, is a common practice. Such over confidence, in majority of cases, can prove disastrous. Flooding for curing of concrete in foundations, could in swelling soil, damage neighboring foundations and cause disaster.

Collapse of Building in Ankleshwar

A building unit of housing complex collapsed suddenly. The engineer and contractor referred analysis of failure to counter probable victimization under excuse of poor workmanship or supervision by the engineer. The problem is illustrated in Fig. 40 with exploration data in Fig. 41. Figures 42, 43, 44 and 45 show existing similar structures and collapsed building. Fig. 40 shows displacement of plinth with reference to plinth of neighbouring structure.

Collapse Mechanism: Collapse shows one slab over other, with masonry walls (load bearing) thrown out as shown in Fig. 46. The settlement of 28 mm and

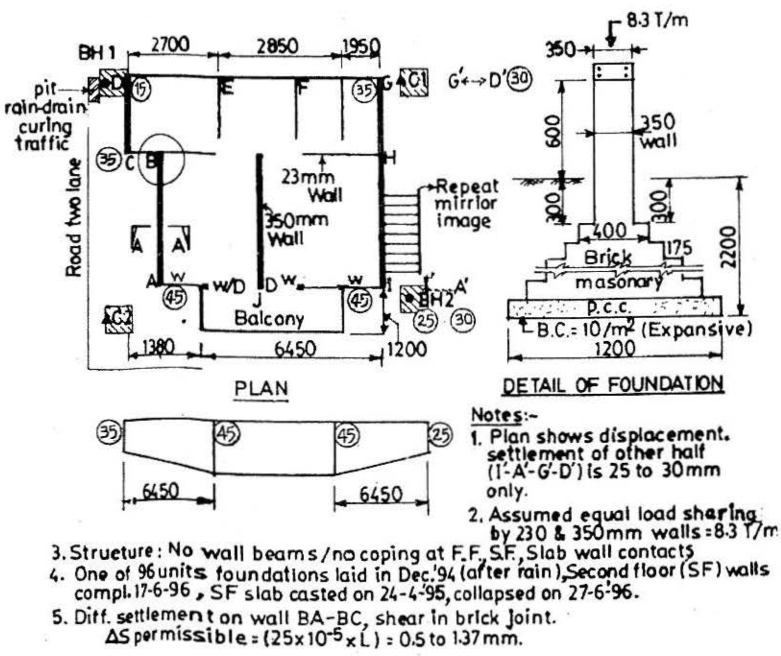


FIGURE 40 : Settlement of Plinth Recorded after Collapse of Building at Ankleshwar

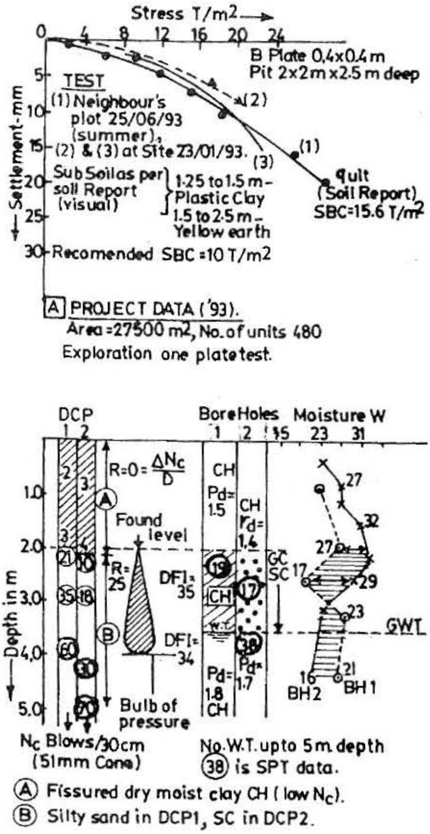


FIGURE 41 : Data of Old and Re-exploration by Writer after Collapse at Ankleshwar

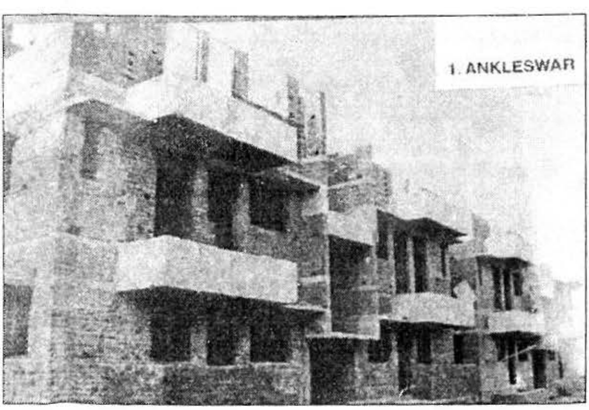


FIGURE 42 : Completed Structure

FIGURE 43 : After Collapse of Strcture

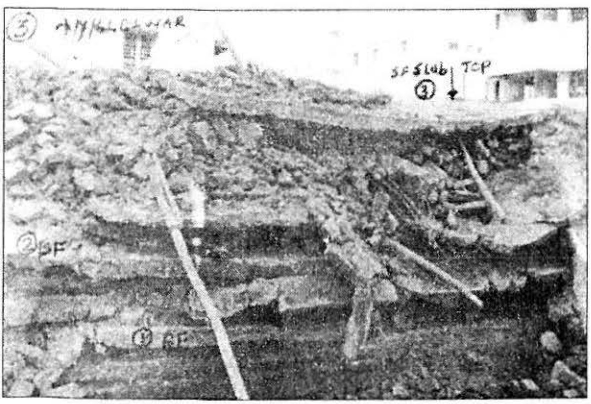


FIGURE 44 : Close-up of Collapsed Structure

FIGURE 45 : Wall below Plinth after Collapse

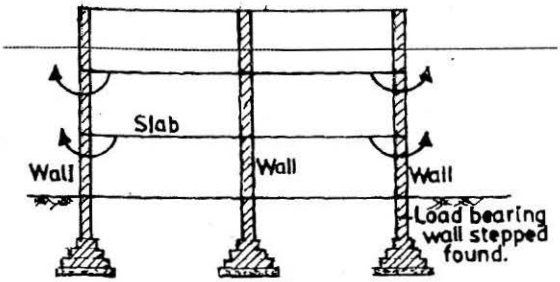


FIGURE 46 : Probable Failure Mechanism – Ankleshwar

a differential settlement of 18 mm was estimated. The permissible differential settlement for span of 3 to 5.5 m is 0.5 to 1.4 mm The observed displacement after collapse (Fig. 40) shows much higher values.

Causes of failure : The officials felt that the collapse of one of many units, was due to poor supervision and bad workmanship. The geotechnical exploration brought out real causes of failure. The failure could be attributed to poor exploration, wrong interpretation, differential settlement due to soil and moisture variation, etc. The exploration ignored existence of water table at shallow depth of 3.5 m. The actual soil below foundation was 'SC-SG' in BH2, and CH in BH1 The moisture contents 17 - 21% in BH1 was 33 to 16% in BH2 at 4 m depth.

Though total settlement was within permissible limit, the differential part turned out to be critical. Poor structural rigidity, moment on balcony producing direct and bending stresses and large opening below the balcony added to subsoil problems. The stress concentration at joints of 350 mm and 230 mm walls due to differential settlement, was critical. Combined soil heterogeneity, poor design of load bearing structure, uncovered pit collecting rain and curing water and vibrations of roller caused tilt. Critical walls moved out stacking slabs - one over other. All this which happened within the design bearing capacity of only 10 t/m^2 proves that adoption of low SBC does not guarantee success.

The single plate load test alone, as exploration for 100 Flats colony is just inadequate, inappropriate and worthless. The IS 1888 was cited without application of mind. Acceptance of poor exploration report shows ignorance of geotechnical engineering or consent to fraud. Such agencies must be checked during exploration, design and execution stage itself for healthy growth of geotechnical engineering. A reported long crack on the wall of the same building, few months before, was ignored by ordering replacement by new brick-work. Such warning of tilt, if analyzed by an expert, could have averted collapse later on.

Significance of Second Opinion and Prototype Studies

As behaviour of soil is difficult to predict, oversafe designers may find project economically nonviable. The same project when reviewed, by experts, willing to take calculated risk using experience based judgement, may make it viable and feasible. What was considered as uneconomical may now be ordinary with public awareness of pollution. In such cases use of well instrumented large scale prototype testing is justified. Following studies will substantiate the view.

Case studies

Rail link Jund – Kandla Problem: A link rail for transport of goods to Kandla Port from northern states was planned considering shortest haulage distance. A 6 m earth fill on 4 to 12 m thick marine deposits at in-situ moisture content equal to liquid limit, was found technically not feasible. The stability of embankment was unsafe (F.S. = 0.9) The continuous settlement of 300 mm was anticipated over 70 to 80 years. A longer route with permanent high recurring cost was the only alternative.

The following facts emerged as a result of the detailed review. The field behaviour of clay and vane shear strength in drilled holes appeared contradictory. The author suggested review of critical parameter C_u by Norwegian vane borer (no drilling) to reduce disturbance otherwise caused by release of stress, drilling, etc. (Desai, 1967). The compiled test results, profile of C_u along the depth are shown in Fig. 47. The C_u by borer was almost double that of vane in a bore hole. Thus, the corresponding factor safety of embankment became safe. The observations show the top 2.3 m crust of CI – MI soil as partly saturated with ground water at 2.3 below the surface. This desiccated stiff crust overlying plastic clay will also influence stress distribution and settlement. Considering the size of the project and need for an acceptable solution to railways and consultants, a test embankment with a sand filter on ground was recommended. The instruments were installed and observations have been also published. The writer recorded improvement of C_u from 0.15 to 0.28 kg/cm^2 (Fig. 47). Piezometers and settlement recorded by RDSO also indicated settlement was small after one year. The prototype test boosted courage of the field engineers to complete a project initially considered unfeasible.

Liquefaction of Ukai Sand Platform and Obra Sand: Use of conventional interpretation of low N_s at dredged sand platform in foundation of Ukai dam and sand beds of Obra, indicate loose state (Desai, 1968). For the seismicity, adopted in design, such a loose sand in foundation could liquefy. This problem was therefore, logically analyzed considering available ground improvement technologies like vibrofloat, etc.

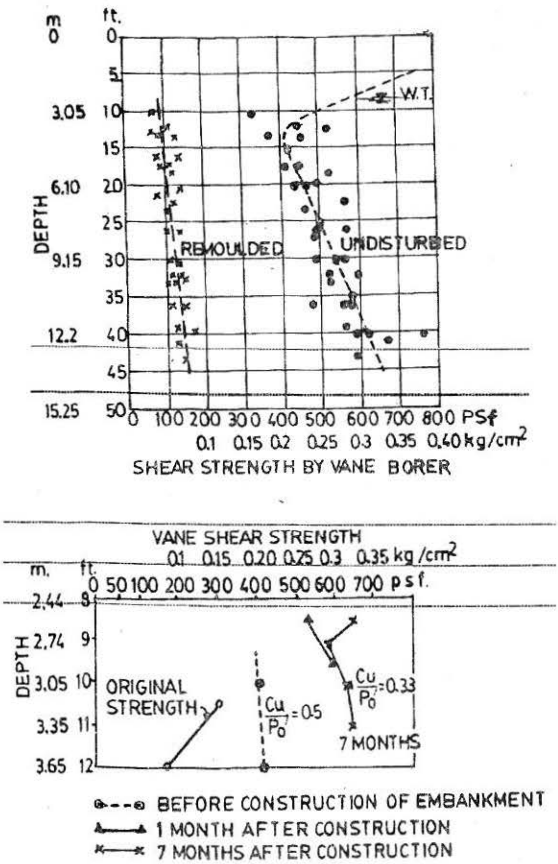


FIGURE 47 : Exploration Data of Vane Borer Showing Improvement of Cu below Test Embankment

Author, considering surcharge pressure and studies for rounded gravely sand, felt the need to check the estimated relative density. The result of check tests and in-situ measurements (Desai, 1968, 1970) brought out that sand was medium to dense with relative density more than 60%. Detailed recheck by static cone test also confirmed density. Such dense sand will not liquefy. Conventional vibration table studies based on loose sand parameters were considered uncertain and hence not considered. Hence a prototype blasting test was planned at Ukai. In-situ blasting test with simultaneous recording of intensity of accelerations, generated pore pressures, settlements with different charges also confirmed that the liquefaction did not occur. DCPT test was used to cover vast area quickly. Such large scale studies not only provided appropriate economical solution but also provided experience in use of blasting techniques for ground improvement.

Kandla Berth : The Kandla jetty (Pais & Desai, 1968) is another example. The second opinion on unsafe jetty provided probable cause and diaphragm wall as a remedial measure to stop sand flow with dredging. Though expensive at first look, if one considered land, location and planning for new jetty taking 4 to 5 years for construction, it was very economical. Geotechnical experts have to face such problems, and boost up confidence and guide construction agencies for execution using unconventional technological options.

Prediction versus Performance

There are many design models to predict soil behaviour analytically. These predicted behaviour and actual performance can have very wide gap. This gap is attributed to heterogeneity, anisotropy, wide range of grains under variable microstructural characteristics, moisture, environment and number of simplified assumptions to bring complex problems within mathematical domain. Soils below foundation and used in earthwork are massive and will not follow simplified laws. Mass can not be expected to be characterized by predicted single parameter for shear strength and compressibility characteristics.

In such cases monitoring of instrumented massive structure like walls, earth dams provide factual records of behaviour of heterogeneous soil mass. The feed back enables back analysis for close scrutiny. This enables to validate better the modeling of the behaviour of structure. Such models form the best tool for rational approach with specific factor of safety including factors of ignorance and confidence. Typical case studies presented will prove these points.

Construction Pore Pressure in Dams

Design of earth dams involves stability analysis using construction pore pressures based on Hilf's method. The construction of core of Beas dam consists of wide range of soils, different modes and degrees of compaction (Desai, 1967). The pore pressure predicted, used in the design (Desai, 1968, 1993) and observed during construction, are shown in Fig. 48. The theoretical predictions are very conservative.

Earth dams at Ukai and Damanganga in Gujarat State, in the first phase of execution, confirmed actual construction pore pressures as 60 to 70% of theoretical values. The "design as you construct" flexibility permitted economical revision of design in subsequent phases. Study of trends of field observations for number of dams, if analyzed considering variables like soil, plant for compaction, moisture range of OMC, indicate excellent potential for more appropriate model for predicting construction pore pressure for design.

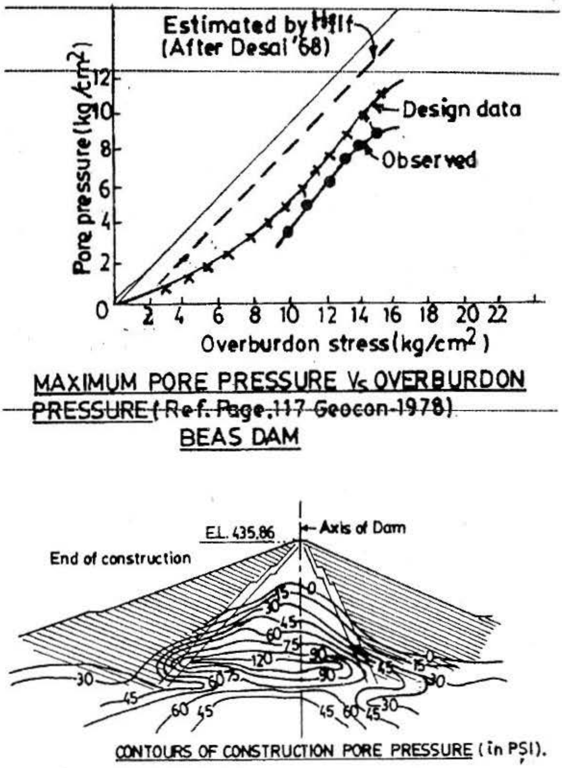


FIGURE 48 : Estimated and Observed Construction Pore Pressures at Beas Dam

Extent of Field Control

A poor quality control on massive earthwork jobs is so loosely expressed by every designer that an additional factor of safety is justified. A typical section for Shalandi Dam is shown in Fig. 49. Particle size and Atterberg's limits of the soil used as a construction material for the core, are also shown. Using data of bores on axis of completed dam, a range of in-situ density and moisture contents along with the range of MDD and OMC is plotted. The heterogeneity and wide range of Proctor and in situ densities do not justify a single value soil parameter used by designer for stability analysis.

The field engineers during execution felt that actual foundation subsoils are not as anticipated by the designer. The exploration by a constructing firm indicated lower shear parameters, high permeability and high SPT values.

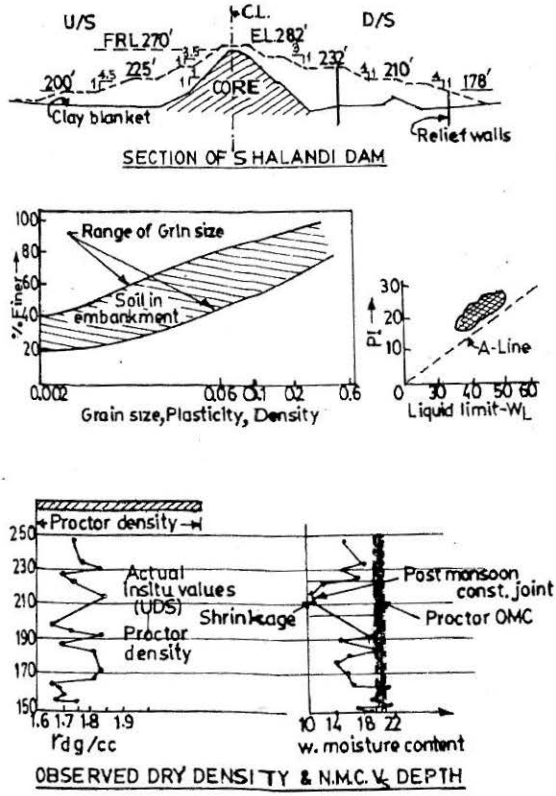
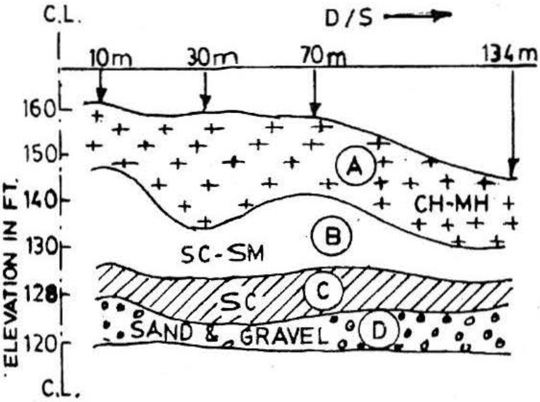


FIGURE 49 : Data of Earthfill based on Bores through Shalandi Dam

They advised flattening of slopes and ground treatment by grouting, etc. Ministry of Water Resources advised three confirmatory bores from dam constructed to 75% of full height. The objective was to evolve appropriate design parameters for foundation subsoil. CSMRS used dry drilling technique for such bores. Though only foundation data was required, data of embankment was also collected (Fig. 49)

The analysis of foundation subsoil is as shown in Fig. 50. The analysis of slope stability with these parameters gave higher factor of safety. Thus the revision considered essential by first investigation was found unwarranted by the second.

By observational approach many decisions can be taken as the work progresses. The importance of it can be seen from this case study. Field observations of embankment prevented a disaster. If only assigned work could be executed this could have been missed. The dry drilling was difficult at



FOUND. SUBSOIL

Layer	Soil	γ_d g/cc	W %	c' kg/cm^2	ϕ'
A	CH	1.55	22-31	0.07	19.5°
B	SCSM	1.52 1.92	14-27	0.3	27°
C	SC	1.5 — 1.67	20-25	0.07	32°
D	SW	1.8	—	0.07	40°

FIGURE 50 : Foundation Profile and Engineering Properties for Shalandi Dam

some depths. Use of limited jetting to puncture such layers was resorted. The heavy loss of water in stiff clay core at certain depths, recorded in field notes, unveiled a different dimension. The permeability coefficient was higher than 10^{-1} cm/sec by approximate computation.

This data and Fig. 51 highlighted a construction joint around RL 210′ at centre of dam. The earth work was at different elevations upstream and down stream. The monsoon rain water traversed zigzag on both upstream and down stream to a depth of one meter or so. Post monsoon shrinkage cracks formed dry clods, filled up by deep rain cuts during stripping. Such channels could be from upstream to downstream and are prone to internal piping on filling of reservoir. Relief wells on downstream with controlled filling of reservoir above RL 205 FT was suggested. The seepage patches were observed on downstream in 1969. Low head saturation, over time, allowed clods to swell by 40% thus obstructing free flow of water (self healing). The downstream wet patches dried out with time, confirming self healing works.

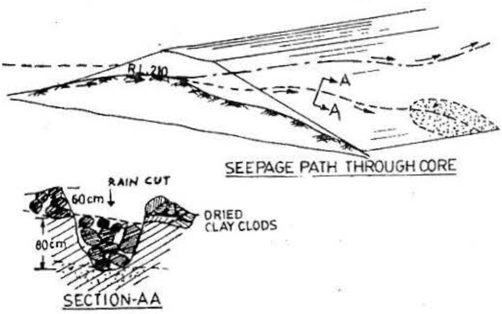


FIGURE 51 : Analysis of Exploration showing Probable Seepage Channels in Core of the Dam

Shear Parameters for Compacted Fill

The design of earth dam is based on critical shear parameters, evaluated on critical samples of proposed borrow areas. The samples are compacted to evolve design MDD and OMC (safer and feasible in field). The shear parameters are then analyzed for critical samples compacted to MDD and OMC and saturated by soaking or back pressure. Every professional associated with design and construction is apprehensive of bad workmanship in massive work. This factor is indirectly reflected in selecting design parameters from a range of values and factor of safety

Author compared shear parameters of number of undisturbed samples (UDS) from compacted embankment with design parameters for Shalandi and Beas dams. The parameters of UDS from constructed roll-fill dam and design assumption based on laboratory compacted samples for Shalandi and Beas dams are presented in Fig. 52 (Desai, 1975). They represented large numbers of samples. Inspite of wide range of soil characteristics, and compacted density and moisture (Fig. 49), the shear parameters of field core samples provided consistently higher shear resistance. Systematic scientific study of this aspect could provide an approved method of making specimens of borrow area samples such that gap between predicted and actual parameters is reduced.

Vertical Displacement

To design instrumentation for dams, author carried out analysis of well instrumented dams from literature. Theoretical methods of predicting displacements in earth dams appears impossible. The vertical displacement model evolved is shown in Fig. 53 (Desai and Desai, 1982). The trend shows

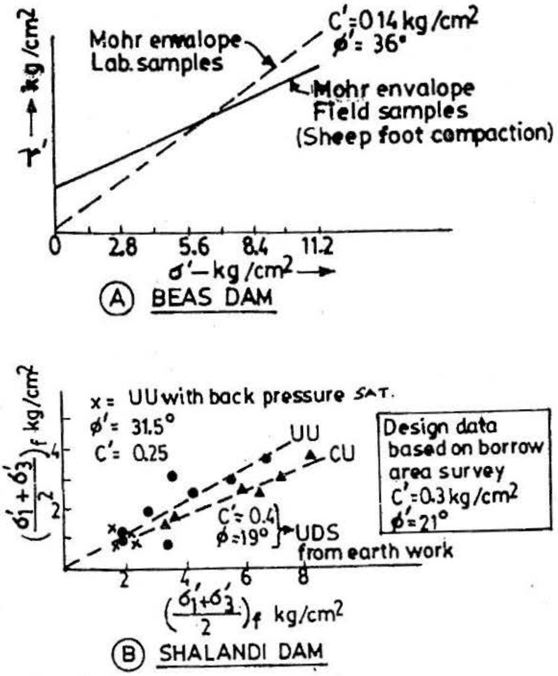
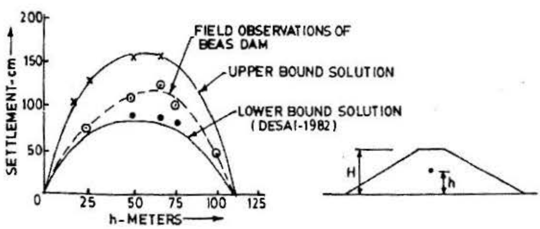


FIGURE 52 : Shear Parameters of Field Compacted Soils Compared with Design Data based on Laboratory Compacted Samples



DAM	HEIGHT (m)	SETTLEMENT	
		Predicated	Observed
USSR	150	900 cm	590 cm
BEAS	110	600 "	458 "
RAM GANGA	67	400 "	386 "

FIGURE 53 : Model to Predict Displacement at Different Height of Embankment based on Prototype Performance

maximum vertical displacement around mid height. The actual displacement reported for Beas dam is superimposed. Inspite of wide range of soil and physical characteristic, model evolved is fairly reliable. Similar observations for other dams are also summarized. Such prototype model is very useful for sensitivity analysis of theoretical models predicting displacements.

Concluding Remarks

Geotechnical Engineering has attained maturity and excellence in its 50 years of existence. The profession, in this process, cannot afford to ignore the forces generating crisis in the last decade. The cases discussed have identified those forces as novice professional experts, exploration agencies, instrument industry, consultants and legal advisors.

The backbone of Geotechnical Engineering, i.e exploration, by acts of omission, errors and commercial misuse, during the last decade, has lost its credibility. Exploration, to majority of users is time consuming and a very expensive process with poor reliability. The interpretations are subjective and could easily be contradicted, hence challenged. Some consultants have reverted to design by art or adopt successful design for worst subsoil condition. The exploration is then manipulated to establish preconceived concepts or used to prove that site has better soil than assumed in design. This has serious chain reactions which, if not reversed, could make survival of the profession to face challenges of the 21st century impossible.

Using DCPT as pilot test over the plot, preliminary design of foundations-type, depth, dimensioning can be completed in 3 days. The required correlation's developed over 30 years by the writer are recommended as first approximation. The detailed exploration of specific stress zone, for critical parameters evolved, is carried out by well planned specific program. Limited bores, critical samples, scientifically selected and personally supervised exploration, eliminates unwanted data.

Special confirmatory exploration and even direct inspection are possible, if the contradictions in design parameters still persist. This approach using low capital kit saves time, labour, minimize contradictions, improves reliability and removes largely subjective elements in interpretations.

The trends to ignore cost aspects of foundations by consultants and clients, ignoring modern technologies, (Ground Improvement Techniques) have been brought out by the cases discussed. Standard designs ignoring geotechnical engineering and adopting design bearing capacities of 5 t/m^2 have, at national level of economy is waste of scare resources of men, money and material worth hundreds of cores/year. The profession must undertake massive program of providing soft wares for type designs (for repetitive structures like towers, abutments – culverts, dyke walls, etc.) incorporating stepless variables of span, heights, evaluation of the design bearing capacity, and cost optimizations for alternative foundation systems. The program can also consider ground improvements in the case of cost of foundation exceeding the normal range. Such program replacing outdated ready design tables could save hundreds of crores of rupees every year.

The increasing trend showing misuse of profession to prove right as wrong or vice versa legally, has been observed in past few years. To defend the decisions, consultants have to evolve different modes of maintaining records. A designer could be penalized by legalities, if defense is not armed with data of subsoil, structure and foundations as built.

Problems, which reached dead end in the first instance, have been solved by second opinion. The writer feels that second opinion and large scale tests should be adopted as a way out in effective geotechnical engineering practice. Such a practice will impart transparency and add to the overall confidence.

Lastly a wide gap is observed between performance and predictions in geotechnical engineering. Use of instrumentation to observe critical parameters for all structures on difficult soils can provide feedback for analysis. Massive earth structures, inspite of wide variations of soil, structure and placement moisture have provided specific trends which can be used to make soil models. Such a model based on performance can make predictions more realistic and provide base for improving theoretical model. The better predictions will reduce crisis of confidence and unlimited factor of safety adopted at present. Despite geotechnical engineering being a science its practice is an art.

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Notations

Rd	=	Relative Density
Ip	=	Plasticity Index
g	=	Acceleration Due to Gravity
q_s - SBC	=	Net Safe Bearing Capacity
qp_{40} - SBP	=	Safe bearing Pressure for 40mm Settlement
q_a - ABC	=	Allowable Bearing Capacity
w	=	% Moisture Content (NMC)
ε_{sw}	=	Strain due to Swelling

DFI	=	Differential Free Swell Index
ws - SL	=	Shrinkage Limit
Hs	=	Layer Thickness Prone to swelling
hsw	=	Heave of Foundations
df	=	Depth of Foundation
$c - \phi$	=	Shear Parameters
B	=	Breadth of Foundation in meters
E	=	Elastic Modulus
R	=	Chainge in Nc per meter depth (DCPT).
N_c	=	No. of Blows per 30 cm, DCP Test
N_s	=	No. of Blows per 30 cm, SP Test, N_s' and N_s'' are corrected N_s for Surcharge and Dilatancy
q_c	=	Point Resistance of CPT (Static Cone)
q_d	=	Resistance to DCPT in MPa (TC -16)
p_o'	=	Average Effective Stress due to Overburden in t/m^2
W.T.	=	Water Table