01 May 2013, 2:00 pm - 4:00 pm

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Commemorate the Legacy of Ralph B. Peck
Seventh International Conference on Case Histories in Geotechnical Engineering
April 29 - May 4, 2013 CHICAGO
and Symposium in Honor of Clyde Baker

APPLICATION OF CASE STUDIES TO PRACTICE IN FOUNDATION ENGINEERING IN INDIA

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ABSTRACT

India has massive developments, urbanization, housing, communication in last decade. The optimization of cost and saving construction time to complete, are now new aspects which geotechnical engineers are facing.

Till today the typical design of shallow foundations of structure-buildings, fly over and dams on non-plastic silty fine sand subsoils found in alluvial deposits of state like Uttar Pradesh, Punjab, Gujarat, Bengal and long coastal belt, were designed by age old practice based on soil mechanics of 1948. Such proven practice became BIS codes for design and construction of structural foundations in 1976-81. The common sense and observational approach of Terzaghi (1959) did not confirm such interpretation of Standard Penetration (SP) test. SP test on non – plastic silty sand at 2 to 3 m below ground surface, being loose (R_d < 15%), had permissible bearing capacity for 25 mm settlement (q_{a25}) less than 100 kPa. This required almost double concrete in footings. Vast country with fast growth had more than million structures built/year, saving of RCC would be around 900 million cubic meter/year. The time reduced will be added advantage. Even up to 10m depth, at number of sites N recorded as 5 to 10 blows/30cm, was considered as “loose” to “medium” by the code indicating prima-facie high liquefaction potential under low seismic activity. This phobia did not spare proposed, under construction over years and existing structures from a long process of reinvestigations, consultants opinions and cost. High rise housing at Chennai, Delhi, Surat, monumental structures at Delhi, Agra, Ahmedabad, Kollkata, Panipat, Rajasthan suffered setback and perpetual suspense due to lack of proper interpretation.

Some dams under construction like Ukai, Tenughat, barrages in West Bengal, Delhi, unique projects like Akshardham (Delhi) had to be stopped or delayed by suspected liquefaction. Long chain of opinions and additional tests like evaluation of R_d by alternative methods, rechecking of SPT values, blasting test as well as cross bore holes shear wave velocity tests had to be planned to remove notional interpretation. Study proposes to eliminate such decays & cost escalation by providing alternatives. Typical case studies, showing methodology are also illustrated. Authors with professionals (30 numbers) in geotechnical engineering practicing in India formed a TC-16 technical group (Year 2000-2005) to prepare a report on ground characterization by in-situ testing.

The final recommendations for interpretation of SP test (N) and DCP test (N_C) for non-plastic alluvial deposit, investigated as per IS code are presented in the form of a chart. It gives for observed N or N_C at P_0’ (effective overburden pressure) the relative density (R_d), ∅’ (angle of shear resistance), E (deformation modulus) and permissible bearing capacity for 40 mm permissible settlement. The chart also indicate likely liquefaction potential at depth for a = 0.1g for preliminary analysis. Typical case studies have been illustrated. The authors advocated bore holes to be supplemented by uncased DCPT adequate in number, to provide recommendation for an area (not point). If results are not satisfying commonsense, check by in-situ tests for R_d, plate load, even prototype test shall be used before resorting to rejection of site or adopting ground improvement. Any recommendation, for probable liquefaction for existing or under construction project, must be checked by proper reinvestigations and interpretation.

INTRODUCTION

India had unprecedented fast growth in housing, infrastructures, irrigation and power sectors after 1980. Till then geotechnical exploration was done mostly by State or Central government research stations. Private qualified and academic sectors involved in exploration were limited to cities like Bombay, Delhi, Kolkata etc. Those days, such job was principally undertaken by piling or water bore drilling contractor whose involvement in geotechnical engineering was limited to site exploration.

The soil report with specific recommendation on liquefaction potential in seismic zones is now compulsory for all projects. This has been emphasized by National Building Code CED46 (2005). Unfortunately related codes and practices are yet to be updated taking R&D into account. The foundation system or ground improvement as per code, are still based on soil mechanics 1948-60.

The increased jobs of soil exploration and shortage of agencies that can do the job scientifically, led to crisis. The fieldworks at remote sites are rarely supervised by technical staff. The laboratory results presented contradictions, inconsistency of parameters. The final reporting never discarded/digested results which are not acceptable to common sense (Terzaghi, 1959). Thus an extreme conservative practice of adopting high safety factor, for ignorance, guided the designers. The designer of the foundation is a structural engineer who, unaware of subject, added to the safety factor.

Only problems where failures are reported, few case studies are available but never published. Now with increasing heights, stresses on soil, cost and time to execute, particularly with competitive biddings, review of the practice became inevitable.

The majority of problematic sites relate to the deposits of non-plastic silty fine sand in alluvial plains and coastal belts. The study is limited to such soil, spread widely in country.

The case studies intentionally are not named but to make R&D more effective citation became obligatory. There is no other intension except to justify need for relook at practice for betterment.

To make impact on reader cases without names have been listed. The intension being overall refinement of practice, no ulterior motives of any kind shall be presumed.

PARAMETERS BASED ON PRACTICE

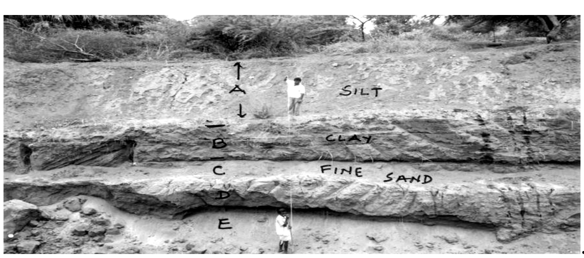

Fig. 1. Coastal zone soil stratification: - silt, fine sand & clay at Surat (Desai, 1992).

Design parameters for the silty fine sand deposits with high water table, namely relative density(R_d), angle of internal friction ($\varnothing$'), modulus of deformation (E) and liquefaction potential, are not determinate as undisturbed soil samples are not feasible with the available setups. The soil is dilatant and relatively permeable. Drilling technique and stratified layered deposits of thin layers of silt and sand do not give even representative grain size distribution. Fig. 1 shows a typical soil profile in Surat alluvium.

Thus only option was to link all the properties to determine insitu parameter Standard penetration test resistance (N blows/30 cm). Though the test is standardized by code, the drilling method adopted and rare supervision by technical staff, particularly observation of layers, drop of hammer and maintaining water level inside bore above water table, has made it nonstandard. The test by two agencies at same location differs (Fig. 10 - 11). The N was interpreted by Terzaghi Peck (1948) and IS:6403 (1981). This practice is summarized in Table 1.

Table 1. Conventional R_d-N Correlation in Practice for Sand (Terzaghi Peck, 1948; IS 6403, 1981)

State Of Strata	N observed (blows/ 30 cm)	R_d: Relative Density (%)	Angle of shearing Resistance $\varnothing$ (Deg)	Foundation Failure	Liquefaction Potential a=0.1g
Very loose	<5 (4*)	10 to 20	<30	L.S/P.S	Very high
Loose	5-10 (4-10*)	35	30	L.S/P.S	High – Medium
Medium	10-30	70	36	G.S/St	Medium
Dense	30-55 (50*)	70-85	41	G.S/St	Low
Very Dense	>55 (50*)	>85	>41	G.S/St	Unlikely

*(Terzaghi Peck Mesri (2010))

G.S- General shear, L.S- local shear, P.S- punching shear, St- Settlement

Notes:

1) Information of Table 1 to be checked by sounding test – DCPT, SCPT or load test.
2) Different geological formation, cemented sand deposits requires calibration.
3) Even the plate load (300 x 300 mm) could mislead, if finer silt fraction is very high and moisture is appreciable in sand (capillary force).
4) The test in top 0-2 m of strata subjected to climate changes and low passive pressure on the SP sampler should not be normally interpreted by using table.(Ravin et al., 2010; Desai, 2006)

Thus safest approach designated all silty sands N<5 as very loose, and N<10 as loose with R_d<35%. The $\varnothing$'<29, shear failure is by local shear for foundation. Also saturated sand with R_d<50% is liquefiable if area is seismic zone of any intensity.

Investigation of N by SPT in practice indicate low R_d whereas for same N interpreted with surcharge factor (P_0') gives completely different state of soil as shown in Table 2.

Table 2. Comparison of R_d by Practice and Fig. 6

Site	N (blows/ 30 cm)	P_0' = Effective Surcharge, (kPa)	Relative density R_d (%)		
			Practice	Insitu density test	Fig. 6
Delhi	3-5	0-20	10-15	40+5	54±8
Gorakhpur	10-20	20-90	35-55	60-80	55-75
Surat	15-20	70-100	40-65	60-85	80-85

Designer, mostly a structural engineer, has little option left than to adopt reported recommendations. He assumed investigator has all expertise in geotechnical engineering and ground improvement techniques. The designs are thus over safe, uneconomical and taking considerable time for construction of foundation systems.

After 2001 Bhuj earthquake, many projects executed, under execution or proposed are suspected for liquefaction if foundations of structures are on saturated sand. Search for remedial measures is undertaken. Some major projects on sand suffered delays, of years and imposed considerable stress on professionals and management.

The guide lines of 1948 practiced, without application of common sense and judgment based on experience acquired over years, ruled as a safe practice. It became part of BIS code IS 6403 (1981). There are hardly any researches, publications of case studies. Rare failures investigated by academic institution and research stations are confidential documents. This blind faith in practice was common up to 1990. It is still practiced by some exploration agencies and rigid followers of code.

NEED FOR RELOOK

The adoption of practice led to large footings. If sum of areas of footings exceeds 50% of plinth area, raft or piles or ground improvement are prescribed. This and the cost, time to execute, expensive ground treatments are resisted by investors in private projects. Some sites, not feasible by cost, bought pressure for better approach. Some illegal constructions of 4 floors over the designed foundations of 2 floors, in these zones gave proof of over safe practice. Tests of common sense, no impression of shoes on foundation soil with 70 kg self weight, indicated design can adopt at least twice design bearing capacity adopted by the practice.

Vast areas in around Delhi, Gorakhpur, Roorkee, Agra, Kolkata (WB), Jodhpur (Rajasthan), Tapti-Narmada alluvium and deep coastal deposits in West and East long coastal belts have similar deposits of nonplastic silty fine sand or fine sand or layered formations. It is sometimes covered by 1 to 3 m of cohesive deposits. The ground water level varies with seasons but normally it is struck at 3 to 5 m below ground level. Exception is a site at Agra where level dropped from 8 m (1988) to 20 m or more in 2012 due to massive water consumption. The rise in the land cost, rise of height of structures increasing stresses, increased cost of building materials, and labour, forced a relook at the practice. The trend of CE infrastructure investments of 11th plan saving 5% of overall cost in foundation by better analysis is justified.

The "case studies based revised practice" took shape in 1965. To arrive at the realistic design parameters say design bearing capacity or probability of liquefaction investigation of insitu R_d, assessment of R_d by plate load test (Terzaghi Peck, 48), uncased DCP test for shallow depths (sounding test), are attempted. For satisfaction of ignoring code, prototype load tests are also demonstrated. If R_d is more than 50 to 60%, to prove no liquefaction field blasting test measuring acceleration, pore water pressure, settlement, and latest cross borehole shear wave velocity tests are adopted whenever economically justified.

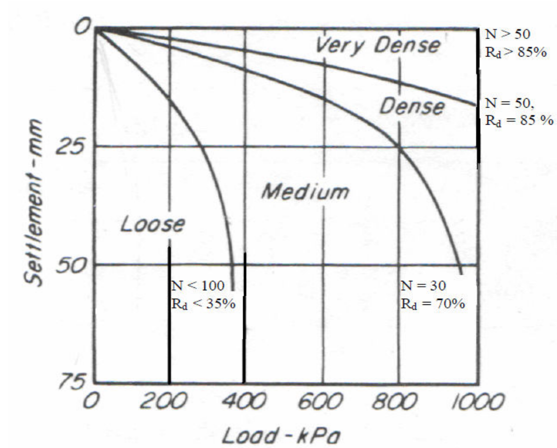

Fig. 2. Chart for estimating relative density of sand on basis of results of standard load test on bearing plate 300 mm square (Terzaghi and Peck, 1948).

PROFILE AND SOIL TYPE

The alluvial planes, desert and coastal zones studied are predominantly silty fine sand or thin layers of silt, fine sand (Fig. 1). Disturbed sample analyzed is actually a mixed soil mass of layered soils. The typical profiles of top 1 to 8 m of such deposits are illustrated in Fig. 3. The typical grading in Fig. 4 shows 20-35% silt and rest fine sand with some medium sand. All samples are non plastic. The ground water except Agra, is normally 5 m below ground level. Agra is exception where ground water level reported pre 70 as 8 m, has been 18 m in 1988 and is now below 20 m (2011). Though spread hundreds of kilometer apart, range of soil profile and grain size distribution shown in Fig. 3 & Fig. 4, modeling it as uniform saturated silty fine sand (non plastic) is justified.

The design of foundation of dams over such deposits requires shear parameters for the stability analysis and deformation modulus for settlement analysis. Structural foundations on such soils are designed for allowable B.C. which is lower of safe bearing capacity (SBC) and (PBC) permissible bearing capacity (IS 6403, 1981) for 25 or 40 mm settlement. The practice and code based on N blows/30 cm are the only available source.

INVESTIGATION AND INTERPRETATION

Normal investigation in above deposits is drilling 100-150 mm hole with alternate Undisturbed Shelby tube samples and SPT at 2 m intervals. (Terzaghi et al., 2010). The codes, equipment test, limitations of rope pulley are discussed in detail in TC-16 report: Ground property characterization from insitu testing (Desai, 2005). Undisturbed sampling by Shelby tube is not feasible as density and moisture will not be insitu values. SPT test is conducted and up to 10 m shows N=6 to 10 for Delhi (Desai, 1970), N = 10 to 12 Roorkee(Prakash, 1967), N=7 to 10 for Ukai, strata RL 195 m to 190 m Yamuna barrage Delhi(Handa, 1965), N=5 to 10; for depths up to 7 m to 10 m, N=6 to 10 at Chennai and Hajira (Surat) etc. Fig. 3 categorized this stratum as very loose to loose with R_d 20 to 35%. The shear failure will be local shear as ∅ <30 for bearing capacity (Table 1). The sand with R_d < 50% is liquefiable under seismic condition; this is the interpretation by geotechnical reports. The obvious recommendations are deep or raft foundation and ground treatments for control of settlement and liquefaction. Table 3 shows variations of predicted R_d at different sites by different methods.

Table 3. Data of Sites Indicating State of Compactness Based on N(practice) and N' (surcharge corrected) N-P_0'-R_d for Shallow Foundations (Desai, 1970; Desai, 1972)

Site (Agency)	N Blows / 30 cm	N' for P_0'= 90 kPa	R_d % based on		
			Terzaghi (N)	Based on N'	Plate load Test (Terzaghi)
Basaidhara, S.P road, Sabzimandi, Lady Hardling Medical College, Delhi. (CSMRS)	5-10	25-30	Loose	Dense	Medium Dense
PWD building of Ahmedabad (OZA et al) Df = 2 m	24	72	Medium	Very Dense	Very Dense
Surat Textile Market (SVR)	20	50	Medium	Dense	Very Dense
Dhuvaran Df = 2 m (GERI)	40	120	Dense	Very Dense	Very Dense
Delhi (CSMRS)	5-8	48	Loose (<30)	Medium Dense (50)	Medium Dense
ISBT Old Delhi (Desai, 1972) (Df = 3m, Dw = 4 m)	7-11	25-30	Loose (<30)	Very Dense (65)	Very Dense

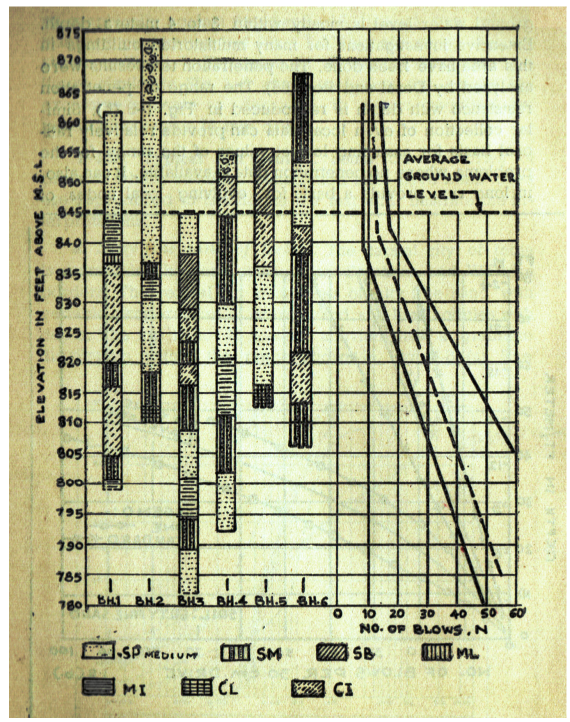

Fig. 3(a). Variation of soil and penetration resistance of subsoil surrounding Rorkee (Prakash and Singh, 1967).

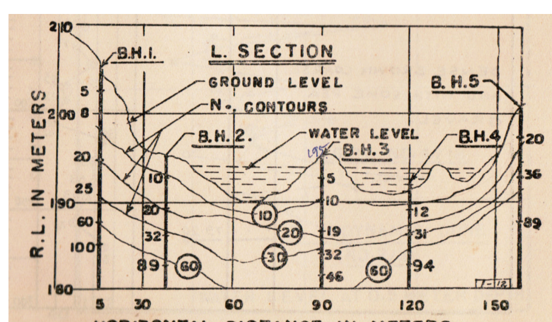

Fig. 3(b). L-section showing the contours of N with depth for a Barrage site (Handa and Chetty, 1965).

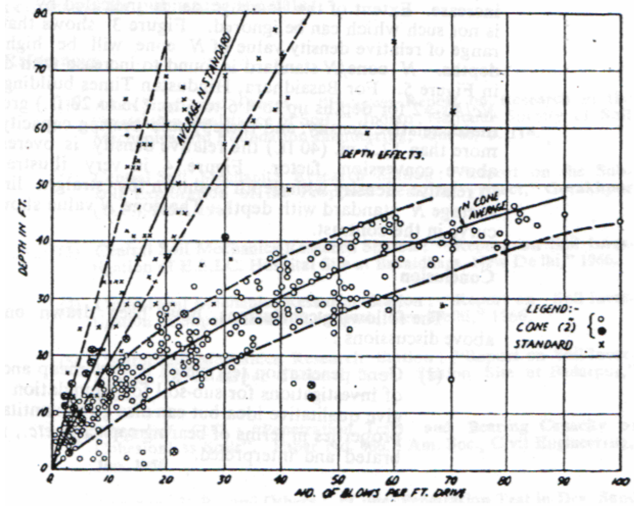

Fig. 3(c). Variation of N and N_C along depth (Desai, 1970).

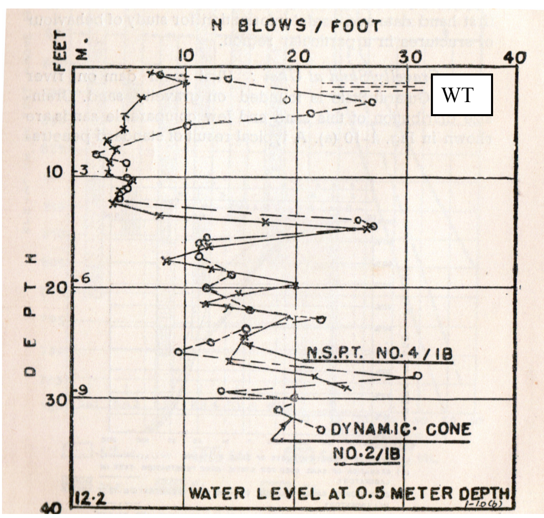

Fig. 3(d). Typical depth penetration curves for saturated gravelly sand at Ukai (Desai, 1970).

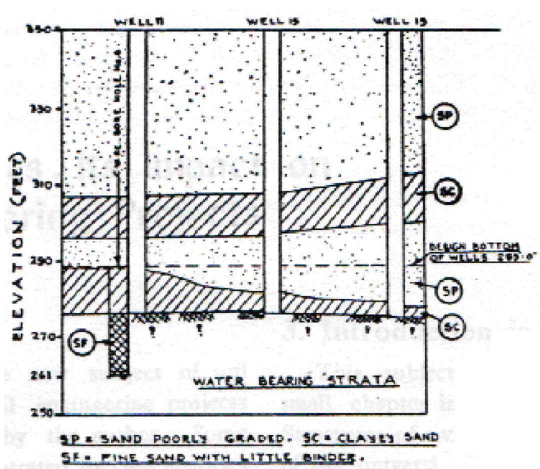

Fig. 3(e). Soil profile at Basaidhara, Delhi.

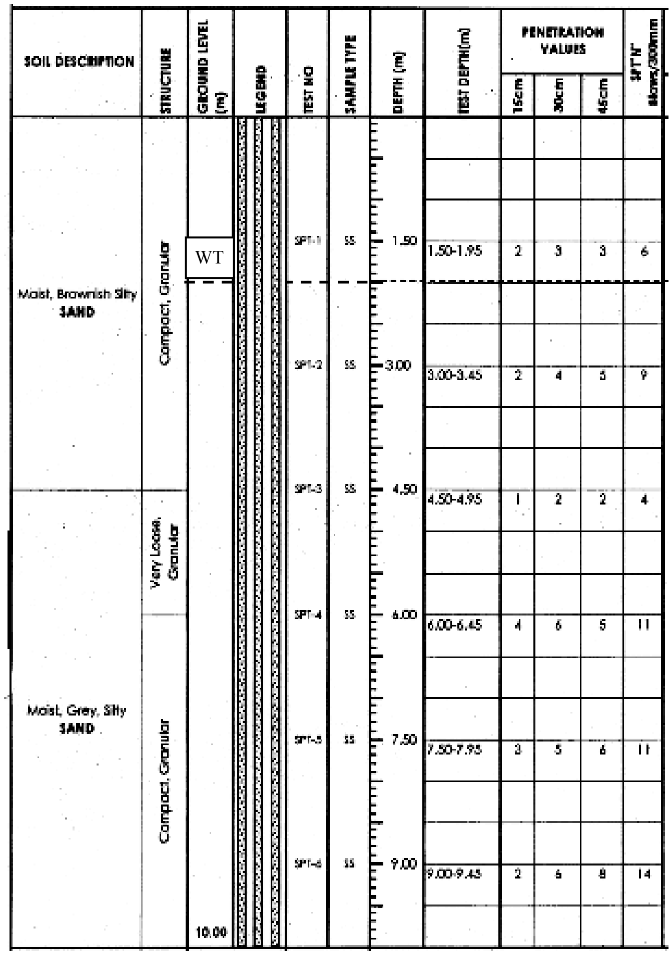

Fig. 3(f). Soil profile at coastal belt, Chennai (Geo foundation and Structures Pvt. Ltd., 2012).

The settlement of footings are computed as per IS 2132 (1981) (Desai 2005). The drilling, augar, bukey (Bailer), wash boring without casing upto 8-10 m are common. The free fall drop of hammer, pulley, coir rope, and manual operation varies depending on supervision and agency. The test, punching through clayey top, overlying saturated sand gives low N because of boiling in bore. The difference of water level in bore and outside, caused by drilling method, cause internal piping/boiling giving N of loosened strata. Many reports do not record ground water level. IS code permits use of liner but it is optional.

The N blows/30 cm is corrected to N` for effective surcharge pressure by factor C_N (C_N = 2 for P_0' = 0 kPa to C_N = 0.8 for P_0'=200kPa). For the dilatant saturated silty fine sand alone, it is further reduced to N"=15 + ½(N'-15). Using Fig. 6, N- P_0'-R_d-q_{p40}, N – P_0' line is projected vertically to P_0' curve of 210 kPa instead of 280 kPa, as reference, to read N' corrected for effective surcharge pressure for all zone explored first time. After enough observations reference curve P_0'=280 kPa is advised.

This being safer it is applied to all sands and non saturated sands by many. Some reports adopted ASTM corrections for the tests as per IS code. For all practical purposes for shallow depths up to 6 m, N_C by DCPT cone = N. This reduction and ignoring boiling and disturbance by boring technique, the forecasted parameters are very conservative.

The code for N=10, stress, P_0' = 100 kPa and width B=2 m shows St of 30 mm, for W.T. beyond 2 m depth below footing. For W.T. at 1.0 m below footing settlement will be 60 mm.

Thus for saturated silty fine sand with WT within B width of footing, design B.C. 100 kPa became a standard practice. The hydraulic structures on such foundations required anti liquefaction ground treatment.

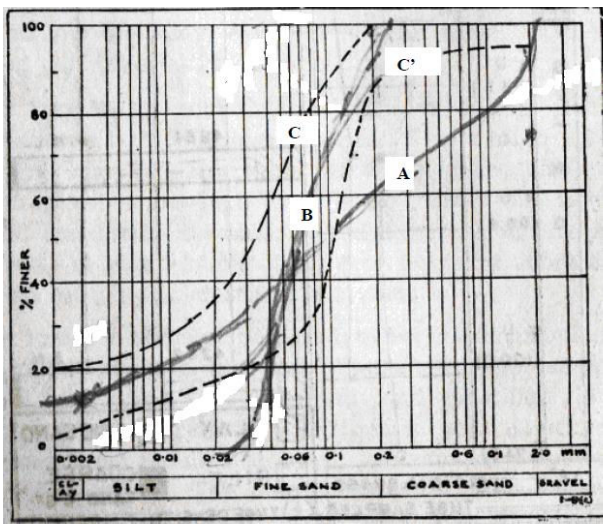

Fig. 4(a). Range of subsoil grading, A-Chennai; B-Hajira, C-C'-Delhi (Desai, 1970).

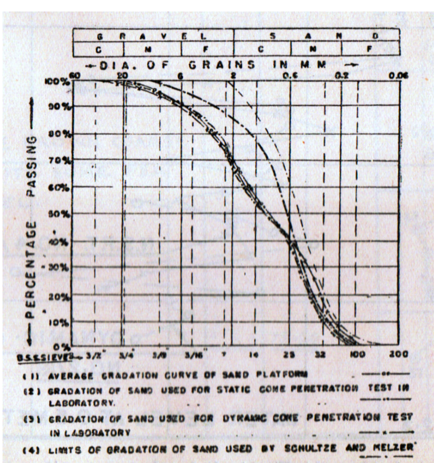

Fig. 4(b). Grading of the gravelly sand at Ukai sand compared with other sand (Desai, 1970).

IMPACT OF THE PRACTICE

Massive low cost and other urban housing in India involves million structures/year on silty fine sand subsoil areas. Each structure with average 15 columns of 100T loads requires 10m^3 of RCC footings/column with practiced 100 kPa permissible bearing capacity. If relook permits PBC of 200 kPa for same soil, RCC footings will not exceed 4m^3/column saving 6 m^3 of RCC/footing. For 15 million footings, 90 million cubic meter of concrete is saved every year. The cost, saving of materials cement, steel and time to construct, can not be ignored.

The presence of 50-100 years old structures, some in seismic zones (R= 6 to 7), many illegally raised constructions on original footing for one or two storied structures, supplemented by strong common sense that even shoe print of 70 kg self weight did not give measurable imprint on sub soil, supported relook. In past 100 years, inspite of seismic activity massive liquefaction is never reported in above areas. Hence practice and fresh investigations are subjected to forensic investigations. The impact on economy, rising prices of cement, steel and need to reduce time for construction, accelerated relook at the practice.

METHODOLOGY FOR STUDY

Data of explorations, performance, geotechnical reports of different practicing agencies, R & D report of State and Central government, practicing designers and academic institutions in above soil zones are scrutinized.

For cost sensitive industrial, housing projects exploration by bore was expanded by insitu R_d test, DCP test, Plate load model or prototype footings tests. The N-R_d relation practiced is checked by these alternative techniques, including Terzaghi, Peck & Meshri (2010) approach. Some clients agreed to bypass code, if technically higher PBC is proved by model or prototype tests. A prototype load test at ISBT Delhi is shown in Fig. 5.

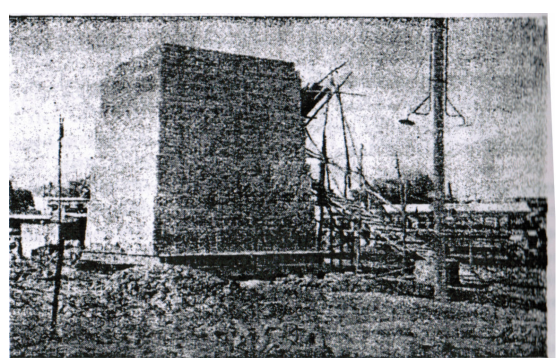

Fig. 5. Photo plate showing loading test on a prototype footing at ISBT, Delhi (Desai, 1970).

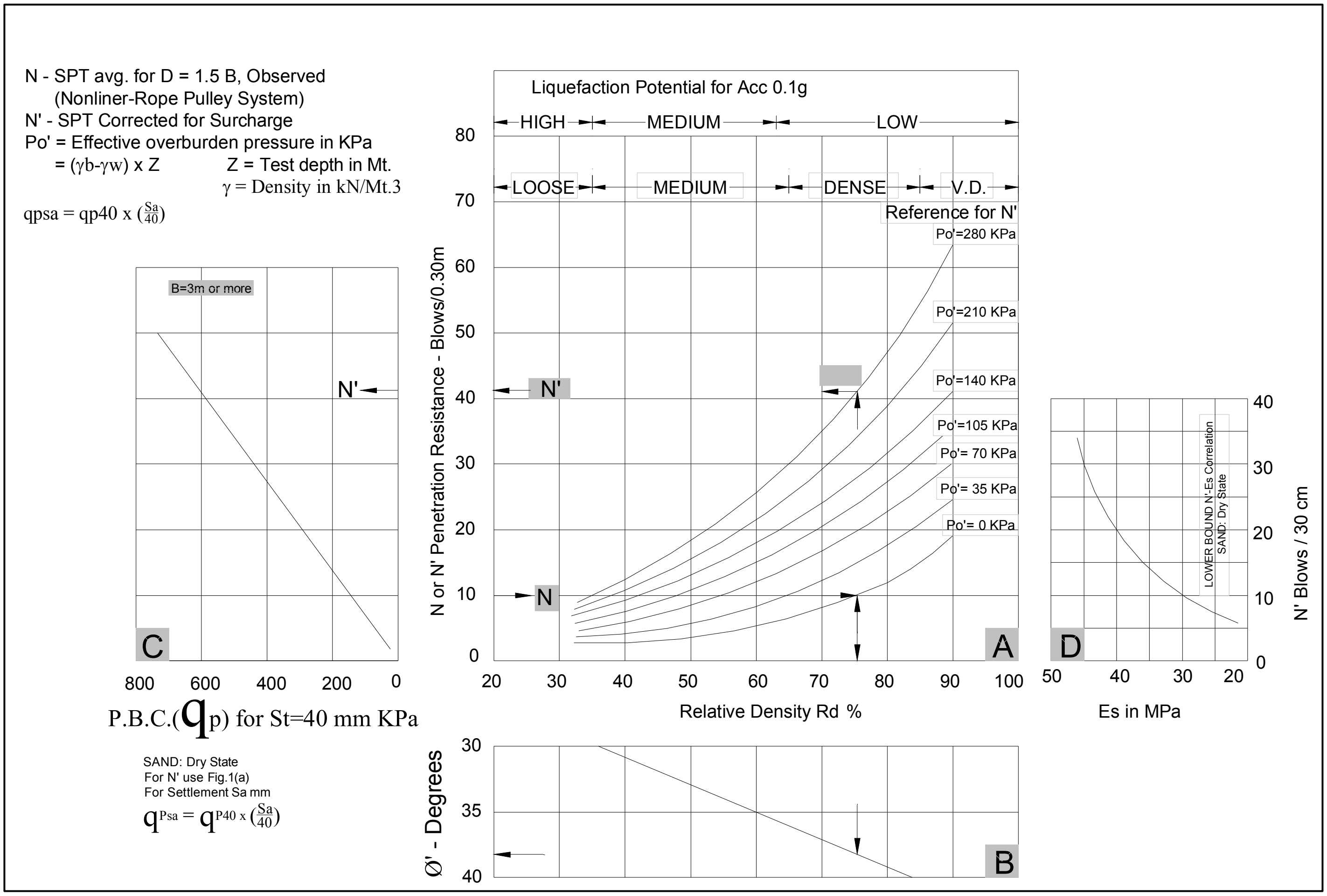

Fig. 6: (A) Correlation of N-P_0'-R_d-N', (B): N-P_0'-∅' (for dry N.C. soil), (C): Permissible bearing capacity for allowable settlement of 40 mm using N', (D) N'-Es correlation for dry sand (Desai, 2005).

Number of projects analyzed includes work of CSMRS New Delhi, published data of others and R & D publications (Desai, 1970). Indian team (1998) of TC-16 of Int. Soc. of SMFE involved geotechnical engineers, users, investigators, academicians, BIS, from all over India. The 3 years of deliberations ultimately evolved interpretation chart for SPT (N) and DCP (N_C) test 51 mm cone based on draft (Desai 70)10. This interpretation with technically supervised SPT/DCPT as per code, provides the design parameters for a given depth (P_0'-effective surcharge pressure; R_d, deformation modulus (approx.); PBC for 40 mm settlement and preliminary idea of degree of liquefaction for acceleration of 0.1g. They are given in Fig. 6 & Fig. 7 respectively. For Fig 6, N – P_0' – R_d line is projected as shown to P_0' = 210 kPa to read N' for reference P_0' = 210 kPa and liquefaction potential.

The low SPT by boiling/piping during drilling is checked by sounding test DCPT, interpreted by Fig. 7 for R_d.

The charts for first time use for a geotechnical formation, a conservative approach of adopting P_0'=210 kPa as reference is advised. The rod frictions do not permit use of DCPT beyond 8 m in saturated silty fine sands. A comprehensive book ground property characterization from insitu testing was published by Surat chapter Indian geotechnical society giving code, proposed revision and interpretation (Desai 2005).

TYPICAL CASE STUDIES

Some selected case studies for predicting R_d, ∅', E for projects spread over country are covered to establish validation of revised approach.

Estimating Relative density (R_d %)

Projects at number of cities sited elsewhere and Table 3, investigated by different agencies, have been cross checked with insitu density test, interpretation of DCP test nearby and occasionally plate load test.

Project in Rajasthan.

A typical case study of desert sand in Rajasthan project consultant treating sand as loose, collapsible by referring to geology, desert sands (Singh, 1986) advised SBC of 100 kPa.

This sand N = 9 blows/30 cm, at depth of 3 m (P_0'= 50 kPa) as per practice (Table 1) is loose. The verification at site by undisturbed samples, N_{SPT}-P_0'-R_d, DCP test, N_C= 10 blows/30cm, 300x300 mm plate load test is illustrated in Fig. 8. R_d = 60-75 %, 62-76 %, 67-80 %,65-85 % respectively shows sand is dense to very dense and not in loose state. Depending on cost sensitivity and structure, one or all techniques are selected to verify state of denseness. This index changes all design parameters and hence cost.

Ahmedabad (Gujarat):

At Ahmadabad a major housing complex explored to 8 m showed silty fine sand with water table at 1.5 m below ground level. N=12 at depth of 3.0 m (P_0'=36 kPa). The sand is treated as loose and pile foundations are prescribed. Reinterpretation shows R_d>60 % and PBC for St = 40 mm is more than 400 kPa. Considering local variations in sand, W.T, PBC 240 kPa is recommended at 3 m below ground level, subject to confirmation of N_C>10 at F.L. in each unit during execution. Table 2 & 3 (Desai et al., 1974) shows wide difference in prediction of R_d by practice and other methods.

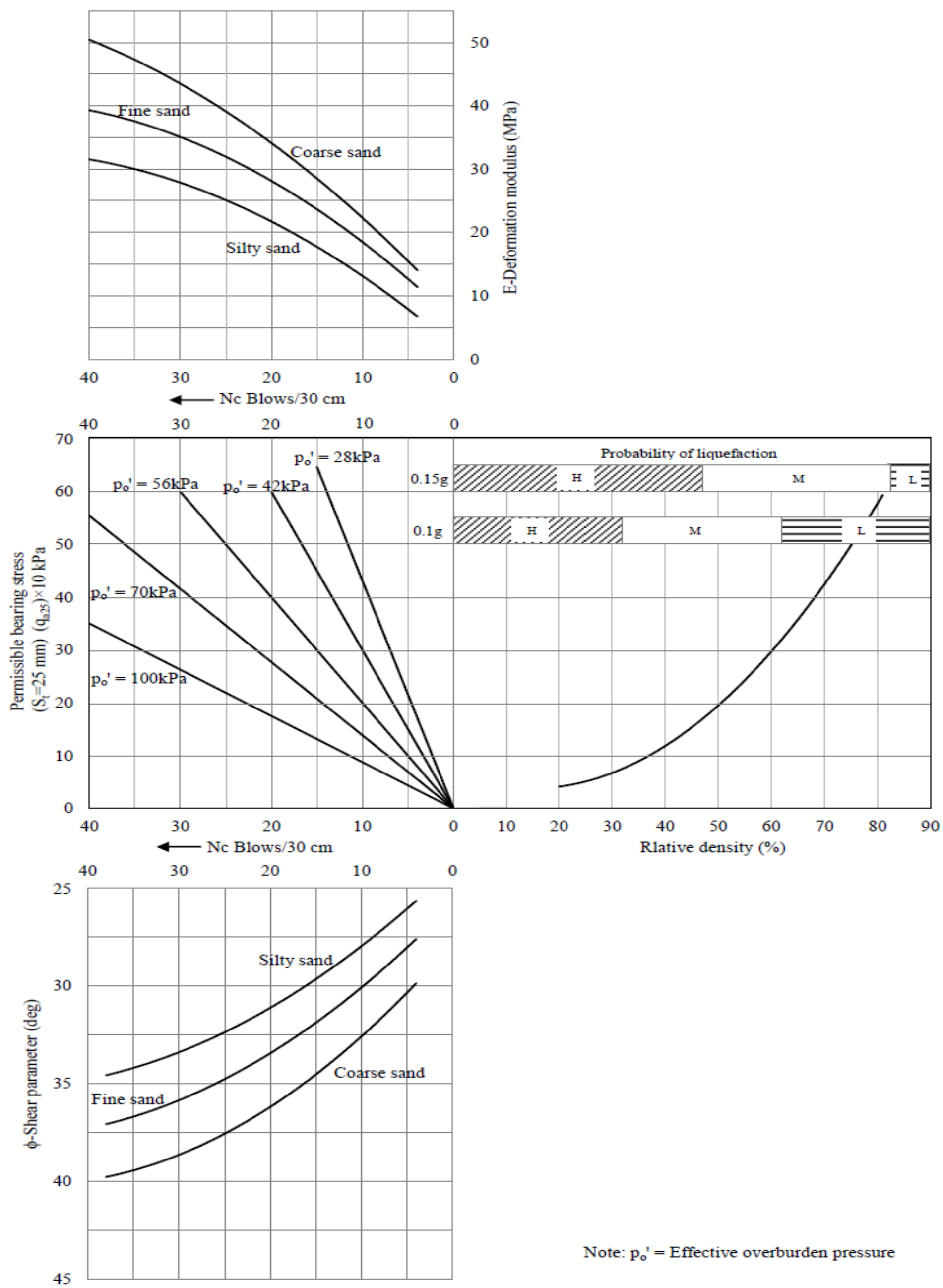

Fig. 7. Interpretation of Dynamic 51 mm cone sounding test in silty fine sand (Desai, 2005).

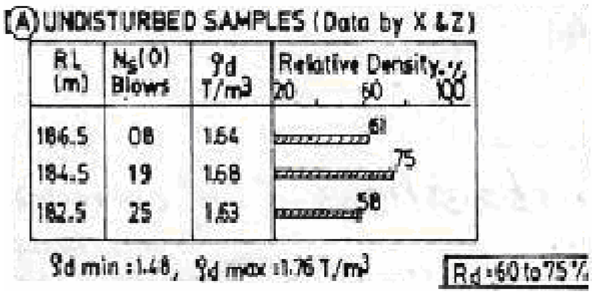

Fig. 8 (A). Undisturbed samples.

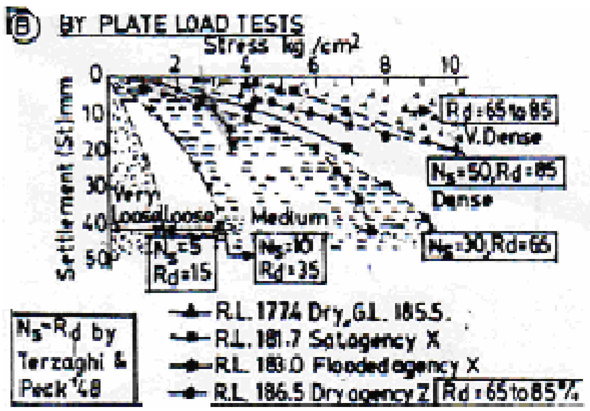

Fig. 8 (B). Plate Load Test.

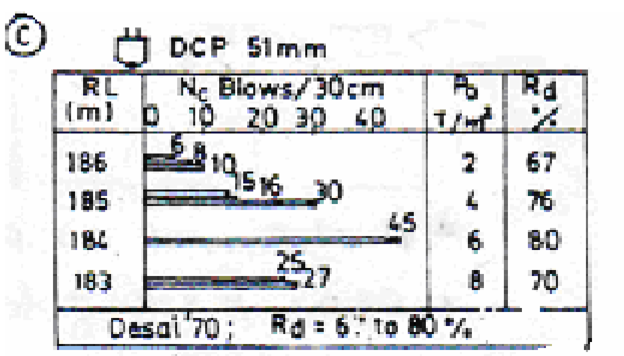

Fig. 8 (C).DCP Test.

(D) SPT DATA	
N	9 to 14
Df	2 m
P_0'	45 kPa
R_d	62% to 76%

Fig. 8 (D). SPT Data.

Fig. 8: Investigation of R_d for project in Rajasthan by different methods (Desai M D, 1999).

High Rise Structures Surat Bhatha:

Soil exploration shows top 0-4 m of MI soil overlying silty fine sand with W.T. at 3m below G.L., N=10blows/30 cm, P_0' at 3.3 m = 50 kPa shows permissible bearing pressure = 240kPa for settlement = 40mm. Open excavation, suction of excavator and seepage of water from base caused heaving. The DCPT test in pit was carried out. Fig. 9 shows reduced N_C=6 blows against before excavation N_C = 30. By look, subsoil is not suitable for design bearing capacity 220 kPa. Use of geofabric with sand cushion of 300 mm for drainage of pore water pressure adopted, could permit laying of footings. Actual overall total settlement recorded was 120 mm with all dead load. The 80 mm additional settlement has been attributed to heaving of 6 m strata below and reduced modulus to 30%. Study revealed that it is possible for other buildings to be designed so that final settlement is within limits.

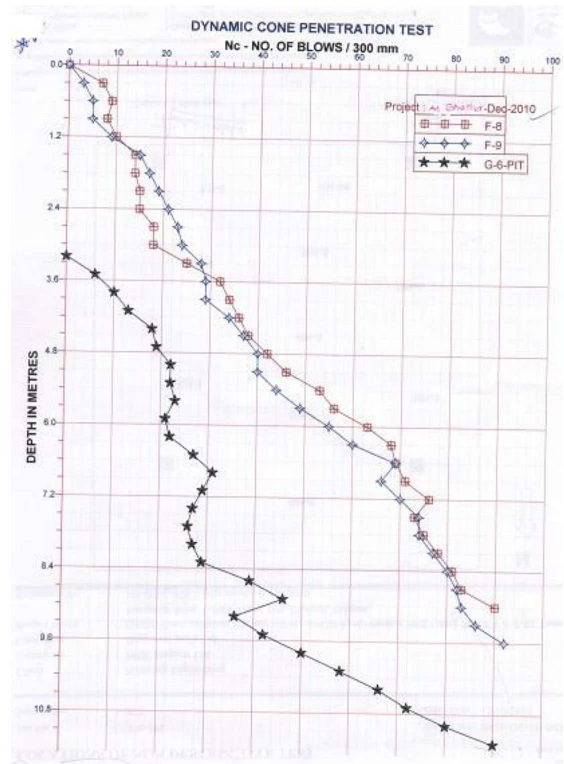

Fig. 9: Effect of excavation on depth with respect to depth at Bhatha site for high-rise building.

Structures at Ennore Chennai.

The soil exploration for shipyard at Ennore Chennai (Geo Foundation and Structures Pvt. Ltd., 2012) shows top 4.5 m silty sand with Navg = 13; 6 & 7.5 m depth shows N = 3 to 4. W.T. is at 1.6m below G.L. Before adopting N =3 to 4, loose state, check test by DCPT is advised. DCPT test is direct continuous test of shear resistance of subsoil. For practical purpose N = N_C for top 5m in saturated silty sands. As N_C recorded was 12 blows/30 cm, silty sand could not be very loose. SPT test analysis is not reliable.

Agra Project:

Fig. 10 (1988) is plot of N vs depth shows N range of 4 to 16 for 0 to 13 m depth for Agra project. This top sand N < 10 is loose liquefiable if W.T. is high. The Yamuna deep deposits have been formed under water more than thousands of years ago. The experts considered soil report to suspect inadequate bearing capacity and liquefaction. Final stage execution stopped. On advice, re-investigation was done by 8 bore for study of bearing capacity and liquifiability in 2011. Plotted in Fig. 11 shows N = 20 to 30 for 0 to 14 m depth. Obviously before advising structure is not stable if final roof is placed, check on data available, even if it may be against professional ethics, become inevitable. Similar cases are few. It includes Akshardham temple at Delhi (Desai et al., 2004). A site 100 m away explored for high rise buildings was reported to be loose, liquefiable by soil exploration. (Gupta et al., 2008) and deep piles were executed for foundation. The re-exploration at Akshardham (Desai et al., 2004) removed doubt of liquefaction and flexible raft was executed. Only time will tell reality of actual potential.

The reinvestigation removed doubts of the liquefaction and safety of foundations bearing capacity. Use of Fig 6 – 7 could help avoiding loss of years in execution.

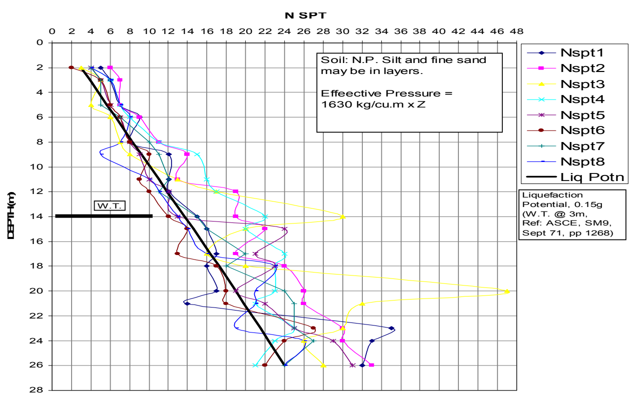

Fig. 10. Site at Agra N_{SPT} (Desai, 2012).

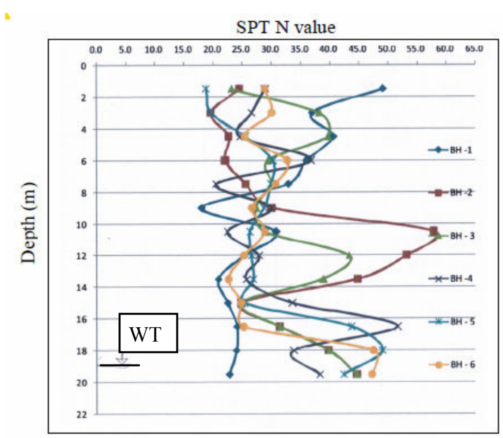

Fig. 11. Site at Agra N_{SPT} (Desai, 2012).

<u>Estimating ∅' for SBC</u>

The design parameter ∅' for saturated silty sand insitu is required for stability analysis of embankment on such deposits or computing SBC for design of foundation for structures. Both the analysis are sensitive to ∅' value which can not be determined in laboratory as insitu density and moisture are indeterminate for such deposits with available exploration techniques. Indirect empirical correlation of N with ∅' adopted in practice is given in Table 1. This is also basis for IS code 6403 (1981).

The code, for R_d = 20 to 30 % (loose state) stipulates local or punching shear failure criteria. For N<10 blows/30 cm ∅'=30° will be tan-1(0.67 tan∅') =20° is used to obtain B.C. factors Nq, Nr. The case study here will illustrate impact for 3x3 m wide (B) footings at 2 m depth, ∅'=20° gives Nq'= 6.4, Nr'= 5.39 against general shear B.C. factors, Nq =18.4, Nγ=28.4 for ∅' =30°.

Thus ultimate bearing capacity of footing will be reduced to 1/3 value for local shear. Such low SBC does not fit in common sense and indicates abnormal criteria safe bearing capacity less than safe bearing capacity for permissible settlement. The safe bearing capacity became allowable bearing capacity in practice.

Based on case studies, estimating ∅' for silty fine sand representing reliable insitu R_d using N-P_0'-R_d- ∅' chart is shown in Fig. 6. Bearing capacity parameters will be extrapolated between local and general shear as per code. For N<10 blows/30 cm shear failure will not be local and computed SBC is 340 kPa or more. To verify the correctness options of plate, model or prototype tests can be availed. A typical result of case of desert sand of Rajasthan project is tabulated in Table 4.

Table 4. The Safe Bearing Capacity and PBC for 40 mm Settlement for 3mx3m Footing at Depth of 2 to 3 m. F.S.=2.5, (Data: Fig. 8)

	Bearing capacity in kPa as per			
	Practice (IS code)	Fig. 6	Fig. 7	Plate load test
SBC	170	580	700	>500
PBC	60	410	400	380

The actual design bearing capacity min. 380 kPa is 6.5 times the value used as per practice. The Fig. 6 for N is used, check by quick sounding test with Fig. 7, is now practiced by some soil testing firms. In few cases model or prototype test are adopted to convince client / designer. Normal design bearing capacity is governed by settlement criteria for these deposits.

Case study Kattupalli Chennai (TN) (2012).
Case of housing complex at Kattupalli (Tailor et al., 2010) on coastal belt of Tamilnadu, exhibited 0-26 m non plastic silty fine sand, silt 30 to 40%. Top 10 m shows N=10 blows/30 cm, ∅'=30, categorized as loose to very loose by local practice / code by investigators (GSF Pvt. Ltd., 2012). The water table is 2 m below ground level. The site was suspected for liquefaction. Foundations for heavy structures are recommended on 500 mm ∅ cast insitu end bearing piles

resting at 30m below G.L. with a safe load capacity of 116 T. Light structures with footing at 1.5 m are proposed on layer by layer compacted sand and gravel, replacing full depth of loose strata. The allowable bearing capacity for treated soil will be 150 kPa as per soil report.

The same data referred for 2nd opinion, revised recommendations are based on correlation in Fig. 6. The same piles are planned for 18.5 m depth, subjected to confirmation of pile load test before adopting. Typical load tests conducted at site, recorded load capacity of 175 tones for 5 mm settlement.

For light structures at depth of 1.5 m, N =10 to 12 blows/30 cm at 1.5 to 3.0 m depth shows the R_d>45%, ∅'=32 and SBP for 40 mm settlement as 400 kPa for B=2 m. Even if submergence is assumed critical, without any treatment, footings can be designed for allowable bearing capacity of 200 kPa. Only compaction at foundation level and insitu density to be checked at excavated foundation level is prescribed. The revised design has overall impact on cost and time to construct the project.

Case of Delhi (2008).
Though many cases are covered around, Delhi and Roorkee etc. by (Desai, 1970) for period upto 1980, the case study presented shows old practice exists even today with some consultants. Near Dwarka (2010), soil exploration for housing complex shows 0-10 m silty fine sand, N=13 blows/30 cm, ground water level at 13 m below ground level. The report for light structures recommends footings at 1.2 m with allowable bearing capacity of 100 kPa. Footings at 2 m depth, N=15 blows/30 cm minimum, P_0'=30 kPa, Fig. 6 reads R_d>70%, ∅'=34, PBC=ABC will be 300 kPa. Liquefaction potential, became low as R_d>60%. The prediction for R_d by DCPT test for shallow depths is presented. (Desai; 1970, 1972).

Predicting deformation modulus (E)

For saturated silty fine sand, E modulus of deformation is required for computing settlement, FEM modeling of geotechnical problems etc. For shallow depths it is principally function of R_d which is related to N. The practice based N<10, loose sand has E<4000 kPa. The revised approach, Fig. 6, 7, shows N or N_C Vs E value directly. In fact for silty fine sand (SM) with similar range of relative density, layered structure (Fig.1) will have different modulus depending on stratification and their stiffness. Thus predicted settlement can vary considerably compared to measured insitu. Literature shows:

- Meyerhof (1966) reported the ratio of Spredicted/Sactual as1.5 to 3.2.

- Meigh and Nixon (1961) reported PBC based on plate load test modulus/ PBC based on SPT as 1.3 to 6.2.

- Indian Sites including (Ahmedabad, Baroda (Oza 1968)) depth 1 to 4 m, N = 6 to 20 blows/30 cm gives PBC range 40-100 kPa by practice.

- D. Appolonia (1968) based on observed 300 footings recommends reduction for water table should be dropped.

- Stress 150 to 250 kPa, Df/B=0.4 to 1, B>3 m, ratio of Spredicted/Sactual =2.0 if water table is at base and 1.25 if water table is beyond B below ground level.

- Permissible bearing capacity for 40 mm permissible settlement as per practice and code shown in Table 4.

Even if water table correction is dropped PBC for St = 40 mm by practice is very conservative. Code without water table reduction shows Spredicted/Sactual=3.6) (Desai, 1970).

The PBC$^+$ for St=40 mm practiced as per code is as under:

N blows/30 cm	5	10	20	30
q_{a40} kPa *	50	80	180	300

* q_{a40} = PBC for St = 40 mm,
$^+$ Submergence reduction is 50% for Df/B<1 and 34% for Df/B>1.

For such sites all over India, N=8 to 10 blows/30 cm, B=3m, no water table reduction, Df = 2m, Sp = 40 mm, SPB = 160 kPa by practice. Fig. 6 gave corresponding value as 280 kPa whereas plate load tests corrected for size of footing gives 270 kPa. The practice/code underestimates PBC for settlement.

The need for adoption of case study based revised parameters for Fig. 6, 7 is established to ascertain higher values, if it does not appeal to common sense, model/prototype test is resorted to.

Liquefaction potential of saturated silty sand

Since a decade, liquefaction has been a major aspect affecting foundations, at time project feasibility, all over India. The vast areas of river alluvium and coastal zones having saturated silty fine sand, discussed earlier, have to be investigated for liquefaction potential. The delicate balance of cost of projects, schedules, long term success are hinged on geotechnical engineers ability to predict, assess and deal with liquefaction susceptibility effectively. (Gupta et al., 2008).

Many projects proposed, under execution or near completion have been subjected to scrutiny of liquefaction. There is no quick sure answer to doubts in absence of data of soil, R_d, seismicity, expected intensity of earthquake, actual water table, return period of earth quake within life of structures. Many projects are reported to be delayed by years resulting in cost escalation and related chain of benefits.

Many agencies of soil exploration interpreted code IS 1893 (part 1) 2002, to indicate liquefaction likely in silty fine sand below water table if corrected N value is less than 15 blows/30cm at 5 m below ground level and less then 25 blows/30cm below 10 m below ground level. It is irrespective of expected acceleration due to earthquake at a distance. This and denseness predicted by N in practice raised doubts on the stability of dredged sand pad below foundation of major Ukai earthdam (1968) in stable southern basalt plateau. Similar doubts for Tenughat dam (1969) and Barrage in West Bangal are reported. To clear the doubt and evolve treatment during execution to convince approving authorities, is task beyond comprehension. Some projects near completion or even completed faced crisis for years in absence of data and experience of real seismic activity and related boiling and piping etc. The zones revised gives acceleration for a vast zone. No other ready maps are available for sites to evolve effects of damping, history etc. Some parts of Kolakata, Chennai, Panipat, Ahmedabad surat etc have been categorized as liquefiable by some investigators.

The Bhuj earthquake 2001 brought out need for compulsory certification of liquefaction potential for all the projects. The above practice and over safe attitude covered many structures – buildings and dams on sand foundations in seismic zones.

Earlier major earth dam at Ukai in 1967 (Desai, 1999) of 110 m height was planned on 6 – 8m of dredged sand platform. This was required to allow post monsoon seepage to downstream reservoir. Based on investigation data of N = 5 to 7 at P_0' = 10-30 kN/m^2 during construction in 1967, the strata was suspected to liquefy even by the tremors of assumed reservoir induced seismicity. The work was suspended. The search of remedial measures of ground treatment and design modifications delayed project by more than 2 years. Foreign experts required SCPT data and geological details. The vibrofloatation for compacting sand was also examined. Loading berms designs and stability analysis for seismicity 0.1g were undertaken.

Meanwhile investigation by deep large diameter well sinking to evaluate layer by layer density, dynamic cone penetration and SCPT are compiled. The N - P_0' - R_d, N_C – P_0' - R_d, Fig 6 and 7 indicated, in situ density indicated $R_d \geq 65\%$. For the major project, this R & D is not adequate. Finally a basting model test on platform at site, creating seismicity of 0.5g and observing pore pressures, surface settlements, boiling, actual acceleration at radial distances established sand as not liquefiable. Similar problems at Barrage in West Bengal and Tenughat dam have also been reported.

Most of sandy sites discussed earlier, have been suspected for probable liquefaction. If N < 10 blows/30 cm at 5m depth with high water table. Some areas investigated and mapped in Panipat, Ahmedabad, Surat, Daman, Chennai, Agra, Noida, Kolkata have reported probability of liquefaction by the above criteria. Five sites in Panipat (Jain et al., 2007) are illustrated as typical case.

The depthwise soil classification, D50, N, R_d % by practice/code as observed unit weight based on Shelby tube UDS, water table are shown in Table 5.

Table 5. Soil Profile for Site 1 at Panipat Division (Jain et al., 2007)

Sr. No.	Depth (m)	IS Classi-fication of soil	∅' (deg.)	D50 (mm)	N value (blows/ 30 cm)	R_d (%)	Unit Weight (kN/m^3)	Remark
1	0.75	ML	25	0.160	8	28	20	
2	1.50	ML	25	0.155	8	28	20	W.T. at 1.5 m
3	3.00	ML	25	0.150	8	28	20	
4	4.50	ML	31	0.090	8	28	20.20	
5	6.00	CL-ML	31	0.150	13	39.50	20.20	
6	7.50	ML	33	0.140	13	39.50	20.10	

Table 6. shows reinterpreted data of above site locations by considering surcharge effect (N- P_0'-R_d) by Fig 6, other sources

Table 6. Reinterpreted data of above site locations by considering surcharge effect

Sr. No.	N – Reference of interpretation	N corrected (blows/30cm)	R_d (%)
1	UD tube samples partly reliable	-	28
2	Jain et al (2007)	9 - 10	28 – 30
3	As per Figure 6	18 - 22	>57
4	Gibbs & Holts(57)	25	>55
5	Schultze (1961)	20+	>60

Thus minimum R_d >55% has been underestimated as 28% by practice. Former results show low probability of liquefaction whereas latter indicates high probability for same site.

The Table 7 shows analysis of liquefaction by Seed et al (1985) based on data of Jain et al. (2007). The analysis in Table 8 based on N' by Gibbs & Holtz explains how N value or index of R_d influences liquefaction from yes to no.

Table 7. Liquefaction Potential by Seed et al. (1985) with Data of Jain et al. (2007), Patel (2008)

Sr. No.	Depth (m)	σ'v (kPa)	τav * (kPa)	N	$(N_1')^+$ 60	Stress ratio	τh ** (kPa)	Lique-faction	Remark
1	0.75	15	2.3137	8	9	0.165	2.475	No	
2	1.50	30	4.5747	8	9	0.165	4.950	No	W.T. at 1.5 m
3	3.00	45	8.9388	8	7	0.145	6.525	Yes	
4	4.50	60.30	13.1359	8	7	0.145	8.7435	Yes	
5	6.00	45.60	17.1204	13	10	0.175	13.230	Yes	
6	7.50	90.75	20.8713	13	10	0.175	15.881	Yes	

* Average cyclical Shear stress
** Shear stress that cause liquefaction
$^+$ N_1', Corrected N for P_0' as per ASTM

Table 8. Liquefaction Potential by Gibbs and Holtz (1957) Interpretation, Patel (2008)

Sr. No.	Depth (m)	σ'v (kPa)	τav (kPa)	N	N' ++	Stress ratio	τh (kPa)	Lique-faction	Remark
1	0.75	15	2.3137	8	-	-	-	No	
2	1.50	30	4.5747	8	-	-	-	No	W.T. at 1.5 m
3	3.00	45	8.9388	8	20	0.360	16.200	No	
4	4.50	60.30	13.1359	8	19	0.320	19.296	No	
5	6.00	45.60	17.1204	13	26	-	-	No	
6	7.50	90.75	20.8713	13	28	-	-	No	

++ - N corrected for P_0' by Gibbs & Holtz

Foundation of ISBT Delhi, Barrage on Yamuna (Gupta et al., 2008), Akshardham Noida, site at Agra, Chennai, High rise at Bhatha Surat city of kolkatta, are suspected for liquefaction by practice. Only correct field density index, N, DCPT, CPT permitted real assessment. Games Village, Delhi considered liquefaction probability and hence planned structure on deep piles. Whereas, Akshardham temple 100 m away rests on shallow foundations with treatment for pore water dissipation (Desai, 2004). Case study for structure at Agra, suspected for liquefaction at stage of completion brings out importance of strong common sense, application of local site conditions, history formation etc. Finally to be doubly sure, of such recommendations, for ongoing projects likely to be seriously affected in terms of cost, time to complete etc., elaborate additional re-exploration for check is inevitable. Even basic data can not be taken for granted and confirmation by Nuclear Probe, cross borehole, shear wave velocity or dynamic prototype test such as blasting is must. The forecast of liquefaction probability, considering impact on project, shall take into consideration following case study.

A site in Agra (Desai, 2012) in deep silty fine sand has water table 8 to 18m deep by different report of 1988, 2008, 2011. The investigation of ground water level is taken casually in many explorations. The impact can indicate wide range of No to Yes for liquefaction. Even if highly complex analysis of liquefaction will be misleading as P_0' will change N' value adopted. The soil report (1988) at site by 8 bored indicated N = 5 to 8 blows/30 cm, allowable bearing capacity for settlement of 40 mm as 60 kPa and 235 kPa for deep well foundations at 14 m depth. This formed basic reference for expert opinions.

The structural damages during ongoing construction over decades, on relook, brought out stresses on walls are very high. Unless loads of structure are reduced, top roof laying is not feasible. Also seismic analysis of structure (Zone IV) and susceptibility of liquefaction are advised by structural engineers. The relook at project by soil consultant, primafacie based on records, analysis, geological history of Agra, Yamuna river and alluvium, behaviour of 500 year old structures around and analysis by Seed and Idriss (1985) approach (Refer Table 7 & 8) but adopting N' (not N), indicates low probability of liquefaction. The basic field data is inconsistent for site and hence fresh exploration by 8 bores and cross bore hole seismic shear tests are advised. The experienced specialist agency was insisted by consultant.

The data of 1988 and 2011explorations plotted in Figure 10 and 11, shows the difference for same site. The water table is beyond 20 m now against 13 m(1988). The site after 2 years loss of time, money, stress was cleared of doubt of liquefaction. The safe bearing pressure for N = 35 at 13.5 m depth for deep foundation was 532 kPa against 235 kPa. Hence the stress problem also did not persist.

CONCLUDING REMARKS

The country from north to south has alluvial or coastal deposits of non plastic silty fine sand. The sites having high water table 3 to 5 m for projects at Roorkee, Delhi, Gorakhpur, Rajasthan, Ahmedabad, Surat – Hajira, Chennai etc explored for shallow or deep foundation by SPT (N Vs depth) are scrutinized for low bearing capacity for design and probable liquefaction.

For most of sites practice, code interpretation of N < 10 blows/30cm at 3 to 4 m permitted permissible bearing capacity of 100 kPa for 40 mm settlement. The site was reported liquefiable. This has serious overall impact on economy, cost of foundations – raft or deep piles and upsetting schedules of completion. Major dams or projects were delayed for years to search remedial treatments or reinvestigation.

The low N recorded, is attributed to drilling operations, literally no technical supervision of manual drop operations maintaining bore water level above G.W., degree of damaged pulley rope system etc.

Technical committee of users, exploration agencies, academicians, designers under TC – 16 Indian committee of Int. Soc. Soil mechanics and foundation engineer 2005, brought out a draft to revise codes. It provided revised N' for N observed and P_0' effective overburden pressure, R_d, PBC for 40 mm settlement and modulus of elasticity as new interpretations. The projections based, Fig. 6 have been checked by insitu actual R_d, by sounding N_C (50 mm) (Fig 7), plate load test (Terzaghi Peck, 1948). Some sites are tested for prototype load on footings, piles etc. This validation was fairly reliable.

The revised approach gave q_{p40} almost 2 times or more and probability of liquefaction doubt was dispelled. Major projects reinvestigated by bores, DCPT, blasting model test to observe prediction of liquefaction and adoption of cross bore hole shear wave velocity test have proved predictions of Fig. 6. The usual Seed and Idriss (1985) analysis done by adopted surcharge corrected N (N') gave logical answers appealing to common sense, experience based judgement in foundation of dams.

The work presented shall restrict over safe assumptions at cost and loss of time. Before making specific recommendations on work completed/in progress, designer must reinvestigate soil – N – water table variations, seismic history again by a well supervised reliable agency. The practice to blindly depend soil report could lead to crisis of costing and scheduling.

Also designer must apply mind, commonsense, experience based judgement for geological history of site rather than accept soil report as final word as advised by Terzaghi and Peck.

ACKNOWLEDGEMENT

Authors are grateful to all who are associated with projects, clients, geotechnical professionals. The institute of SVNIT Surat has encouraged activity and provided infrastructural facility. But for sincere association of Ph. D scholars Jigisha Vashi, Yogendra Tandel and Ravin Tailor, the paper preparation was impossible.

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