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Liquefaction Remediation

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1. Introduction

Naturally occurring soils are heterogeneous materials due to their varying composition broadly termed as granular and cohesive in view of their grain size distribution, and or contractive and dilative according to physical behaviour. Their engineering behaviour is affected by composition, moisture conditions, stress history, boundary conditions, future loading conditions, seismic conditions, etc. The intensity and magnitude of such variable factors make the engineering behaviour of soils under static and dynamic conditions variable from one point to another and is thus regarded as nonlinear and anisotropic. Behaviour under low and high moisture conditions is significantly different under dynamic loading such as earthquakes, machine vibrations, traffic vibrations, etc.

Engineering behaviour of soils under loads is evaluated in terms of shear strength which comprises of parameters known as cohesion, angle of internal friction, effective stress, and pore pressure. Under static conditions (monotonic loading) when loads reach to limit where pore pressure equals effective stress, saturated loose granular soils loose their strength and behave like a liquid. This phenomenon is called liquefaction. While behaviour of medium to dense saturated sands under static loading is dilative in nature, under cyclic loading of certain magnitude and shaking, undergoes failure similar to liquefaction called “cyclic mobility”. Civil engineering structures built on sites of loose saturated and dense cohesionless soils are therefore at risk of liquefaction and cyclic mobility with potential of damage to the structures and need improvement to safeguard against damages by liquefaction or cyclic mobility.

In the wake of failures due to flow slides of loose saturated sand slopes and hydraulic shells of dams, the concept of liquefaction was introduced by Arthur Casagrange in between 1935 and 1938. Large scale damages due to liquefaction came in the lime light during 1964 earthquakes in Anchorage, Alaska, and Nigata in USA and Japan and in Loma Prieta earthquake of 1989 in USA. Geotechnical engineering concepts of shear strength, effective stress, and pore water pressures, etc. were introduced by that time and the geotechnical engineers were able to explain the reasons of such damages; the loss of strength of loose saturated sand soils due to cyclic stresses caused by the earthquake waves called, “liquefaction”. Since then, exhaustive research efforts have been made to understand phenomenon of liquefaction due to monotonic and dynamic loadings, procedures to evaluate liquefaction, and remediation potential hazards due to liquefaction.

2. Background

On October, 8, 2005, Kashmir area of Pakistan was struck by a 7.8 magnitude of earthquake. Keeping in view, damages of public life and infrastructure (roads, life lines, etc.), this was one of deadliest earthquakes in the history of Pakistan and the world. The destruction zone remained in the mountainous regions where soil sediments are mostly devoid of loose saturated sands and therefore damages due to liquefaction were not noticed. However, in the backdrop of damages caused by this earthquake, building codes were revised with stringent measures on evaluation of all earthquake related hazards and construction of earthquake resistant structures.

Soon after this earthquake, a firm had planned to construct a heavy workshops complex in an industrial area located some 65 km west of Islamabad. The planned RCC framed workshops had covered area of 25000 m² and individual column loads varied from 1200 kN to 2300 kN. Geotechnical site investigation revealed that the soil at construction site is composed of loose pockets of alluvial deposits of Indus River comprising silty and sandy strata with varying degree of fines in different layers. Besides potential hazard of liquefaction, the site had low bearing capacity. Allowable bearing capacity (BC) of 100 kPa was available against 150 kPa required to support the foundation. Deep Dynamic Compaction DDC technique being the cheapest as compared to other techniques was selected to remediate liquefaction hazard and to improve the BC to 160 kPa. Uncertainty both in defining subsurface conditions and soil response to a specific soil improvement technique necessitated detailed testing of the site to evaluate effectiveness of the designed program.

3. Purpose and scope

The DDC project at Attock industrial area being first soil improvement project in the country was viewed as a good opportunity to enhance practical aspects of this technique. While the project was going on, the opportunity was used to carry out research with a view to formulate liquefaction evaluation methodology, devise methods to monitor degree of improvement and effectively measure and manage ground vibrations. This chapter while highlighting literary aspects of this technique provides detailed account of planned objectives and outcomes. The author has proposed a practical procedure for undertaking soil improvement project based on his personal experience of this real time project. The scope of the chapter is limited to practical aspects of case history project and covers following:

- Approach adopted for site characterization.
- Methodology for liquefaction evaluation.
- Liquefaction Remediation.
- Ground Vibrations.
- Quality Controls.
- Conclusions and Recommendations.

4. Site characterization

Evaluation for liquefaction involves identifying nature of soils and loading conditions that will cause the liquefaction to occur. While determining liquefaction susceptibility, there could be two possibilities; firstly the soil deposits found in natural state of stresses and subjected to earthquake shaking and secondly the natural soil deposit subjected to construction of structures (static loading) and expected dynamic loading from machine or traffic vibrations. Evaluation for liquefaction must therefore encompass site characterization and well considered loading conditions (past stress history, stresses due to structures, earthquake, traffic/machine vibration, etc.).

Site characterization comprising geotechnical and geophysical techniques of the potential project site and up to a depth of influence of the planned structures should be performed to evaluate all aspects affecting engineering behaviour of the site during useful life of the planned structures. Various aspects of site characterization are described separately and shown in Figure 1.

4.1 Geology

Geological structures such as faults folds, etc., type of sediment (marine, plains, mountainous, etc.), age of sediments, degree of consolidation, past stress history, stratigraphy, etc.

4.2 Topography

Topographic details such as location of the site, general relief of the site and adjacent areas, flow of storm water towards the site, elevation, etc.

4.3 Hydrology and boundary conditions

Hydrological and boundary conditions such as depth of ground water table and its fluctuation with seasonal variation (if any) due to rains, drainage pattern of the site and adjacent areas, proximity to water bodies affecting moisture conditions of the site such as floods, ground water flow regimes, etc.

4.4 Regional tectonic setting and seismicity

Aspects related to regional tectonic setting and seismicity such as proximity to fault lines, type of fault lines, past earthquake tsunami history, local seismic codes on horizontal and vertical peak ground accelerations, duration of shaking, past history of earthquake damages such as ground ruptures, liquefaction, collapse of building, destruction of life lines, etc. A site specific seismic hazard analysis will be very useful tool in evaluation of more realistic dynamic parameters and is recommended to be performed.

4.5 Field and laboratory investigation

Field and laboratory investigation should be conducted to develop SPT/CPT/DCPT profiles (these would later be used for comparing pre to post compaction degree and depth of improvement), unit weight, moisture content, Atterberg Limits, shear strength parameters, shear wave velocity (this will be used to assessing ground vibrations), etc. Depth is an important aspect for evaluation of liquefaction hazard. Generally it should be equivalent to the depth of influence of the planned structures or 20 m depth whichever is larger. Depth of influence of structures can be calculated using stress distribution theories available in the literature.

4.6 Geotechnical profile

Finally a pre improvement geotechnical profile of the soil deposit at site should be developed consisting of type of layering, percentage fines, moisture content, and SPT/CPT/DCPT variation, shear wave velocity in tabular or graphical pattern as shown in Table 1 and Figure 2 (more columns can be added for pre to post compaction comparison). This format will be a useful ready reckoner during the execution phase of the project and for pre to post compaction comparison.

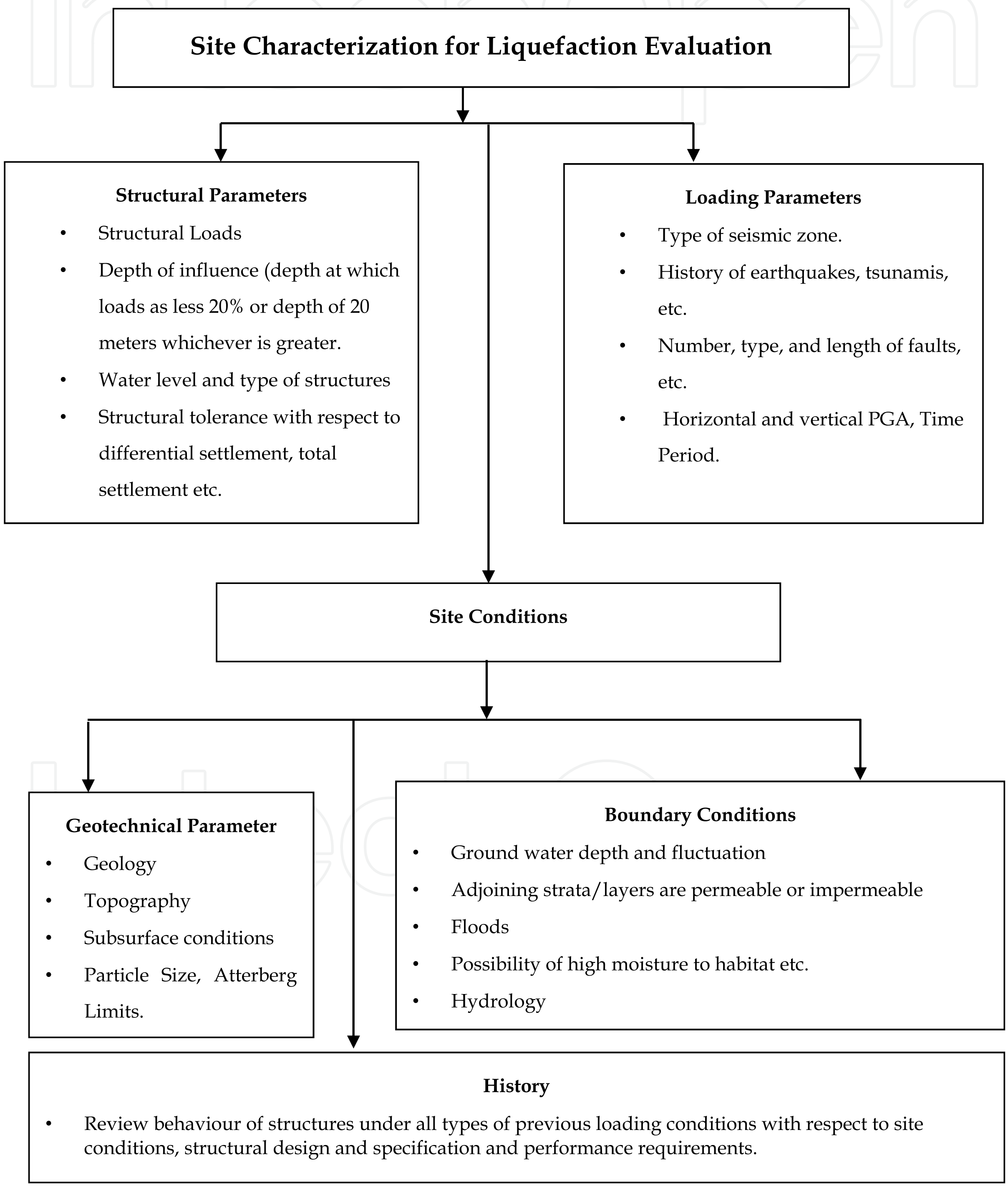

Fig. 1. Parameters fir site characterization

Depth (m)	Type of Soil	Fine Contents (%)	Moisture Content (%)	SPT N-Values		Shear Wave Velocity (m/sec)	
				Pre compaction	Post compaction	Pre compaction	Post compaction

Table 1. Geotechnical Profile

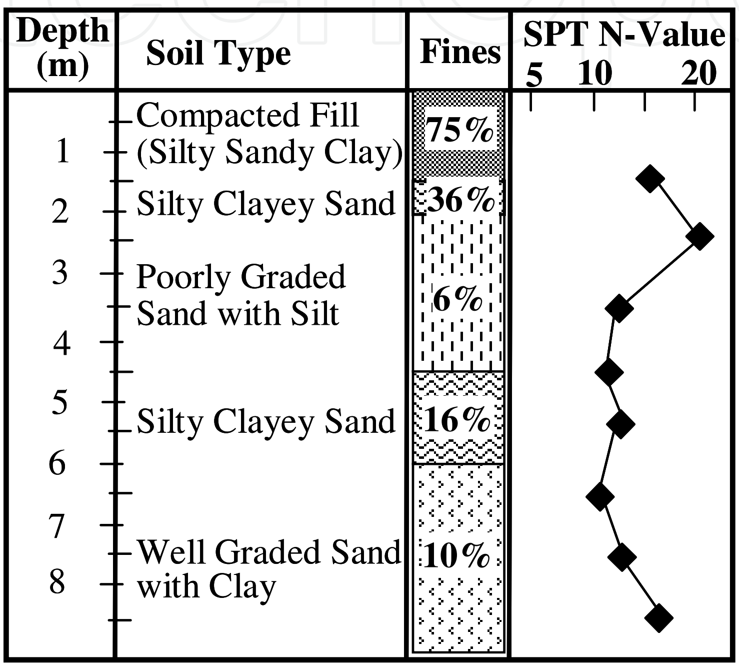

Fig. 2. Geotechnical Soil Profile

5. Methodology adopted for evaluation of liquefaction

Susceptibility of soil deposit at site to liquefaction should be determined through multiple approaches since remediation of liquefaction hazard is a cost intensive event that would raise the overall project cost. Various criterions evolved by researchers. Historical, geological, and compositional criterion suggested by Kramer, (1996) were used to evaluate liquefaction susceptibility.

5.1 Evaluation for liquefaction susceptibility

Evaluation for liquefaction susceptibility can be made according "To" "Historical" "Geologic" "Compositional" criterion proposed by Kramer (1996) and empirical correlations based on SPT N-values. Historically, the construction site of research project is located in Himalayan seismic zone having a long history of earthquakes in the past and prediction of earthquakes in the future therefore the site demanded an evaluation for seismic hazards. Geologically the construction site is composed of poorly sorted an alluvial stratum of recent age with un-corrected SPT N-value of less than 12. Compositionally, the soil layers present at depths from 3.5 to 5.5 m, containing fines as low as 6%, low SPT N-values, and rise of ground water table to a 5 m (from existing 12 m) in monsoon season renders the project site susceptible to liquefaction**.** Since al sites susceptible may not actually liquefy, therefore evaluation for initiation of liquefaction must be performed to actually establish or rule out the potential hazard of liquefaction.

5.2 Evaluation for initiation of liquefaction

The next step after evaluation of liquefaction susceptibility is the evaluation for initiation of liquefaction because it might happen that a soil at a particular site is susceptible to liquefaction but devoid of monotonic and dynamic loading conditions necessary to initiate it. Its occurrence requires a loading that is strong enough to initiate or trigger liquefaction. Most popular methods for evaluation of initiation of liquefaction are "cyclic stress approach" and "cyclic strain approach" (Kramer, 1996). In this paper, initiation of liquefaction is evaluated using cyclic stress approach. An earthquake of magnitude 7, peak ground acceleration of 0.16g, and ground motion of 0.4 seconds were used for evaluation for initiation of liquefaction for the case history project.

5.3 Characterization of site specific dynamic loading

Site specific analysis should be carried out for the expected monotonic loading and the existing soil conditions. Site specific dynamic analysis for a particular site requires characterization of expected dynamic loading (earthquake, machine, traffic, etc.) in terms of a level of uniform cyclic shear stress that is applied for an equivalent number of cycles (Kramer, 1996). Equation (1), proposed by Seed et al. (1983) can be used for calculation of equivalent cyclic shear stresses. In equation (1), τ_{cyc} = cyclic shear stress at the time of earthquake, $a_{\max}$ = peak ground surface acceleration, σ_v = total vertical stress, g = acceleration due to gravity, r_d = value of stress reduction factor for a given depth (0.98 to 0.96 for depth of 3.5 to 5.5 m depth, Seed and Idriss, 1971).

$$\tau_{cyc} = \frac{0.65 a_{\max}}{g \times \sigma_v \times r_d} \tag{1}$$

5.4 Characterization of liquefaction resistance

Cyclic shear resistance should be calculated at various depths using equation (2), (Kramer, 1996).

$$\tau_{cyc,L} = CSR_L \times \sigma'_{v^o} \tag{2}$$

Where σ'_{v^o} = initial vertical effective stress and CSR_L = cyclic stress ratio. SPT N_{Field} values were corrected to $(N_1)_{60}$ by applying appropriate corrections which include corrections for overburden, energy, borehole diameter, rod length, and sampling.

5.5 Factor of safety

Factor of safety against liquefaction is expressed as $\mathbf{FS}_L = \tau_{cyc,L} / \tau_{cyc}$ **.** Liquefaction occurs when $\mathbf{FS}_L$ is less than 1 (Kramer, 1996). Factor of safety should be checked at various depths to ascertain initiation of liquefaction or otherwise. Factor of safety of the research project was less than 1 prior to DDC, Table 2.

Depth (m)	$(N_1)_{60}$	CSR_L	σ'_{vo} (kN/m^3)	$\tau_{cyc,L}$	τ_{cyc}	FS_L
3.5	6	0.06	64.01	3.84	6.52	0.59
4.5	7	0.07	71.43	5.00	8.19	0.61
5.5	8	0.08	80.12	6.41	9.96	0.64

Table 2. Cyclic Shear Resistance of Soil and Factor of Safety (FS_L)

6. Liquefaction remediation

Various techniques are available to improve a potential project site to a desired level of site performance vis-à-vis potential hazards. Selection of site will depend on factors such as geotechnical site conditions, required degree of improvement, availability of technology, cost, time, etc.

6.1 Soil improvement techniques to remediate liquefaction

Some of the techniques suitable for remediation of liquefaction hazards are; soil replacement (if desired depth of improvement is shallow), deep dynamic compaction, vibrocompaction, grouting, deep soil mixing, etc. In order to select a most efficient and economical remediation technique requires a comprehensive review of existing geotechnical site investigation, susceptibility to liquefaction hazard, compactibility of soil at site, cost-benefit analysis of techniques vis-à-vis desired degree and depth of improvement. Since this chapter deals with remediation of liquefaction hazard using DDC, only details pertaining to this technique will be discussed at length hereafter.

6.2 Deep dynamic compaction technique

Deep dynamic compaction technique has effectively been used for remediation of liquefaction hazard in Pakistan and other countries. The technique has also been used successfully for improvement of Municipal Solid Waste sites for construction of buildings, parks, recreational facilities, etc. The technique though simple, requires careful design, construction, and quality controls as described to ensure desired results. It involves repeatedly dropping of heavy weights from a crane on each impact point. The impact energy of the falling weight densifies the soil at depth. DDC program generally consist of two or more passes followed by ironing pass to compact the top ground surface, Figures 3 and 4. The weight of tamper, height of fall, number of drops, and grid spacing is selected to achieve required depth and degree of improvement. The mechanism of densification for unsaturated soil is analogous to large scale proctor compaction. For loose saturated granular soils, the impact from heavy weights liquefies the soils and particles are rearranged to a denser configuration. An impact of heavy falling weights generates ground vibrations which limits its use in built up areas although there are techniques to minimize ground vibrations such as trenches, air bags, etc.

6.2.1 Objective of improvement

The author is of the opinion that a site that requires remediation for liquefaction hazard would also need improvement for soil bearing capacity. Considerations for a soil improvement project should therefore include type of structure, type of foundation, bearing capacity vis-à-vis depth and degree of envisaged improvement. Existing site conditions must be carefully reviewed while selecting and designing a site improvement project. It might happen that the planned site is undulating and requiring fill, or water logged, or may contain vegetation and waste material of old disused structures. While preparing a site for DDC, following must be ensured:

- All fills must be of a material suitable to act as a granular blanket.
- Old pavement and floors of the demolished buildings must be ripped and removed and back filled with granular material to facilitate transfer of impact energy to deeper layers.
- Underlying services and trenches must be removed and suitably back filled.
- Preferably, the site should be fairly levelled.

Multiple options must be considered to improve the site and finalized only after the detailed investigation has been completed for example, for lighter structures or area requiring shallower depth and degree of improvement, different option can be considered than the heavier and sensitive structures. The layout of the planned structures may even be re-adjusted to place critical structures on one segment and remaining ones on the other side. The type of foundation must be considered in post improvement scenario, it might be possible to place the foundation for lighter structures even on the surface thus reducing overall cost of the project.

Fig. 3. Dynamic Compaction Equipment (heavy crane, tamper, release mechanism) and process

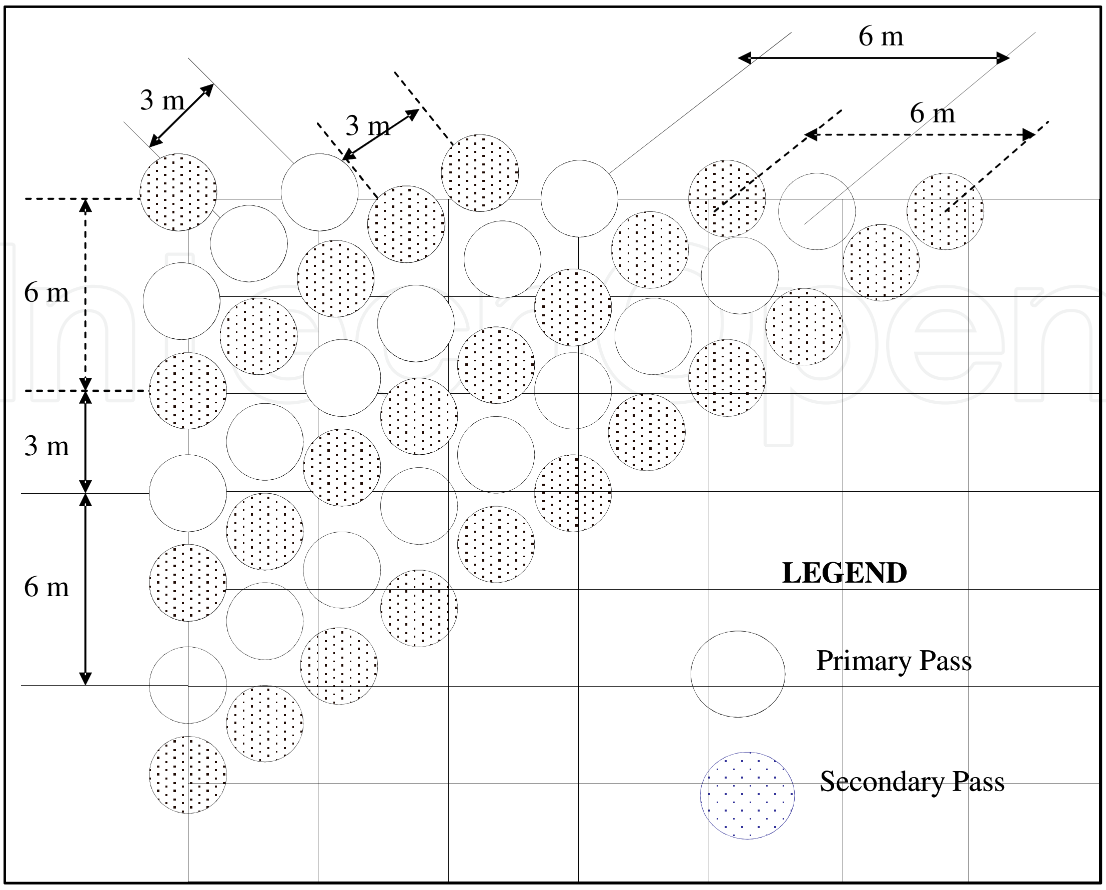

Fig. 4. Diagrammatic layout of primary and secondary pass during DDC

6.2.2 Project performance needs

Candidate civil engineering project requiring evaluation for liquefaction hazard may include low and high rise buildings, life lines (water supply lines, gas pipe lines), bridges, etc. Usually a large project comprises of number of structures with varying specifications, depth of influence and performance needs. It is important to note that all structures within a project may not need soil improvement therefore it would be important to identify structures demanding improvement vis-à-vis their criticality and user needs.

6.2.3 Availability of technology and experience

DDC technique requires heavy duty cranes with lifting and drop arrangements for a tamper from different heights. The tamper can be made of steel, concrete or concrete with steel casing. The tamper should have air vents through its body to enhance impact energy. There could be many unforeseen in such projects therefore an adequate reserve of essential crane spares should be available at site.

6.2.4 Time and cost

Since this technique requires mobilization of very equipment to the site, like all other projects a cost benefit analysis of DDC compared with other options such as avoidance, soil replacement, deep soil mixing, grouting etc. is a pre-requisite for adoption of this technique.

Apart from the cost, DDC is a simple technique but is a time consuming process. Availability of time must therefore be considered right at the outset of the project. Generally this technique is suitable for large projects that do not have very tight timelines.

6.2.5 Preliminary DDC design to evaluate improvement capacity of soil

Determination of compactibility or improvement potential of soil deposit at a site needs no emphasis and depends on factors such as particle size, liquid limit, relative density, type of layers, and depth of ground water table. Although DDC is suitable for most of the soils including municipal solid waste site, its best effectiveness has been reported in granular soils. For cohesionless soils, depth of improvement is proportional to square root of the energy per blow.

A preliminary DDC program can be designed from correlations proposed by Menard and Broise (1975), Lukas (1986) given below as Equation 3 and 4 respectively. Attention must be paid to the applied energy range over the project area for Equation 2 and "n" values for different soils when using Equation 3.

$$D_{max} = \sqrt{WH} \tag{3}$$

$$D_{max} = n\sqrt{WH} \tag{4}$$

As a thumb rule, grid spacing of primary pass of DDC should not be less than the desired depth of improvement with secondary (and tertiary if spacing of secondary pass is more than 3 m) in between the primary pass. An ironing pass with an overlap of 1/3rd of the tamper diameter is good enough to compact the upper soil layer. To finalize the number of passes, number of drops at each impact point, a comprehensive experimental design is required to record both degree and depth of improvement in horizontal and lateral extents. Improvement and vibrations should be verified after pre-selected number of drops, author suggests; firstly after half the number of drops, secondly after next 2/3rd number of drops and finally after all the drops are completed. Keeping in view large number of variables such as site and soil conditions, moisture conditions, depth of ground water table, etc., the author is of the view that best mean to achieve desired improvement is through well planned, executed, and monitored preliminary DDC program. Minimum area for of 24m Xx 24 m is recommended for preliminary DDC.

6.3 Experimental design for evaluation of improvement for field trial of preliminary DDC program

Pre to post compaction improvement can be evaluated by comparing pre to post compaction electrical resistivity profiles, SPT N-values, crater depth, quantum of backfill material, shear wave velocity, etc. As far SPT is concerned, a value should be selected as the criteria to terminate further drops. For alluvial soil deposits with 5 m desired depth of improvement, an SPT value of 15 was selected as limit for discontinuing drops for the case history project. There could be different ways to evaluate improvement during preliminary design and field trial stage of the project. To observe improvement vis-à-vis number of drops or impact energy and intensity of ground vibrations, author used a layout shown in Figure 5 which proved very effective and can be used for future projects.

Evaluation of improvement under impact points, between the craters, and laterally away from impact points is very important, as shown in Figures 6, 7, 8, respectively. With a thoughtful evaluation methodology, actual improvement is possible to assess. For the research project, the layout of test craters and location of boreholes was designed with a view to keep the distance between pre and post compaction boreholes as minimum as practically feasible. In this research, pre to post compactions SPT were performed within a

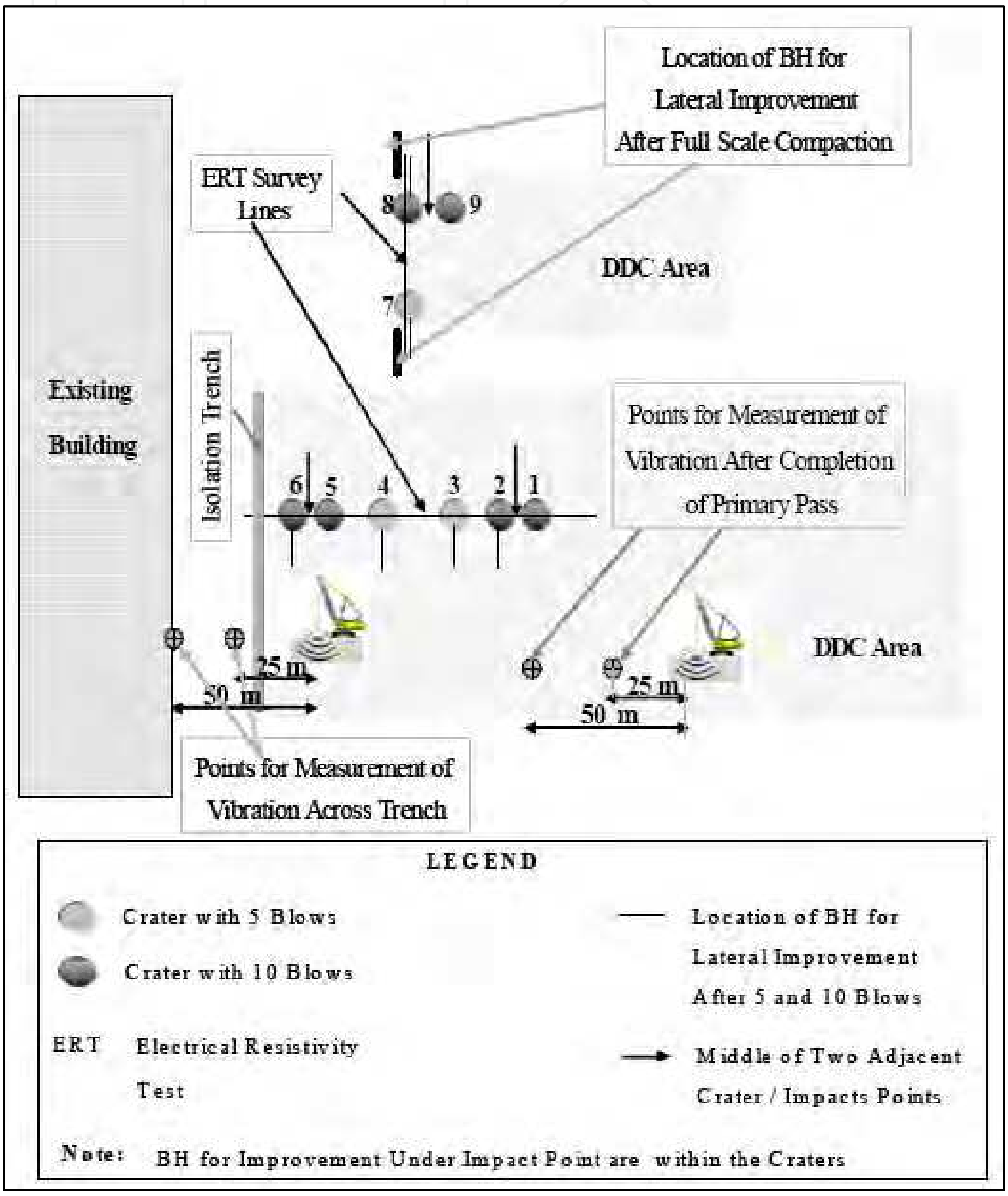

Fig. 5. Layout of Compaction Site, Test Craters, Vibration Measurement Points and Location of ERT Survey

distance of 2 m. Total of 9 test craters; crater no. 1 through crater no. 9, were used in this research. Total of 48 boreholes were drilled and 384 SPT performed to evaluate improvement at different points. Detail of experimental design is given as, refer to Figure 5:

- improvement after 5 blows was evaluated at crater no 3, 4, and 7
- improvement after 10 blows was evaluated at crater no 2, 6, and 8
- lateral improvement after 5 blows was evaluated around crater no. 1, 5 and 7
- lateral improvement after 10 blows was evaluated around crater no 2, 6, and 8
- Improvement at middle of adjacent craters was evaluated between crater no 1 & 2, crater no 5 & 6 and crater no 8 & 9 after full scale compaction i.e. after primary, secondary and ironing pass no compaction was carried out within 9 m of the test craters.
- Locations of all boreholes has been referenced to the center of impact point and are given in terms of tamper diameter "D" (D = 2.4 m)

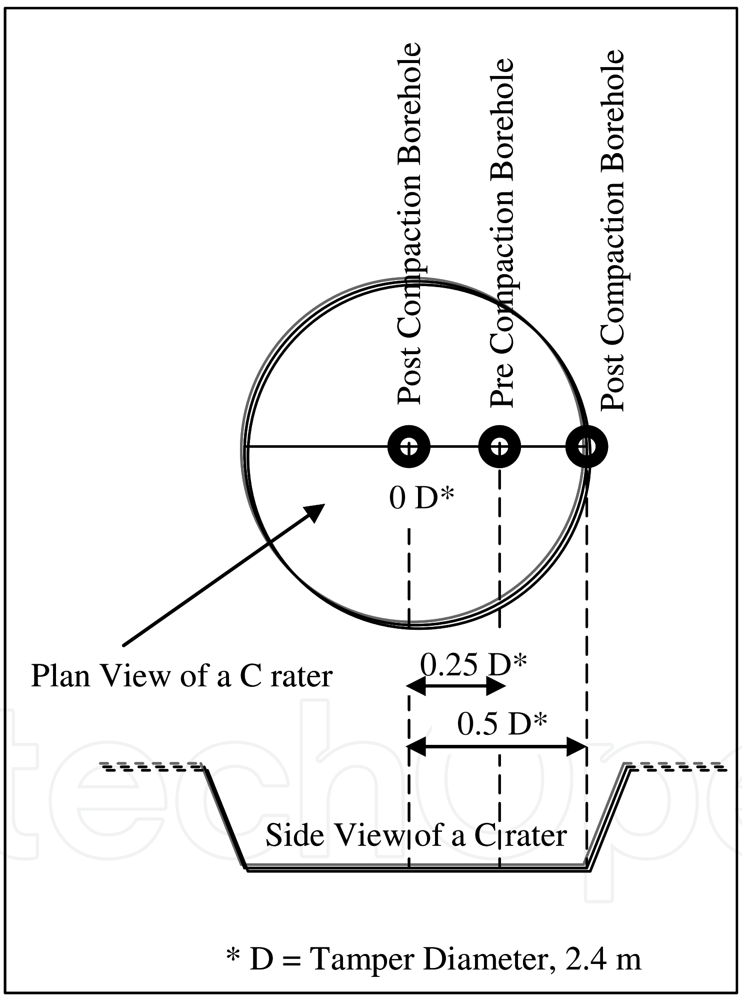

Fig. 6. Location of boreholes for evaluation of improvement under impact point

6.4 Field trial and finalization of dynamic compaction program

Once the field trial is complete, improvement in depth, between impact points and in lateral direction must be evaluated. To assess improvement in realistic manner, details of the field trial of research project is explained in the ensuing paragraphs. SPT were performed two weeks after full scale compaction. Improvement under impact point was evaluated at crater # 2, crater # 6 and crater # 8 (upper halves). Improvement was also evaluated at the middle of crater # 1 and crater # 2, crater # 5 and crater # 6, crater # 8 and crater # 9. Improvement in lateral direction was measured only at two places, i.e., towards lower side of crater # 7 and upper side of crater # 8, refer to Figure 5.

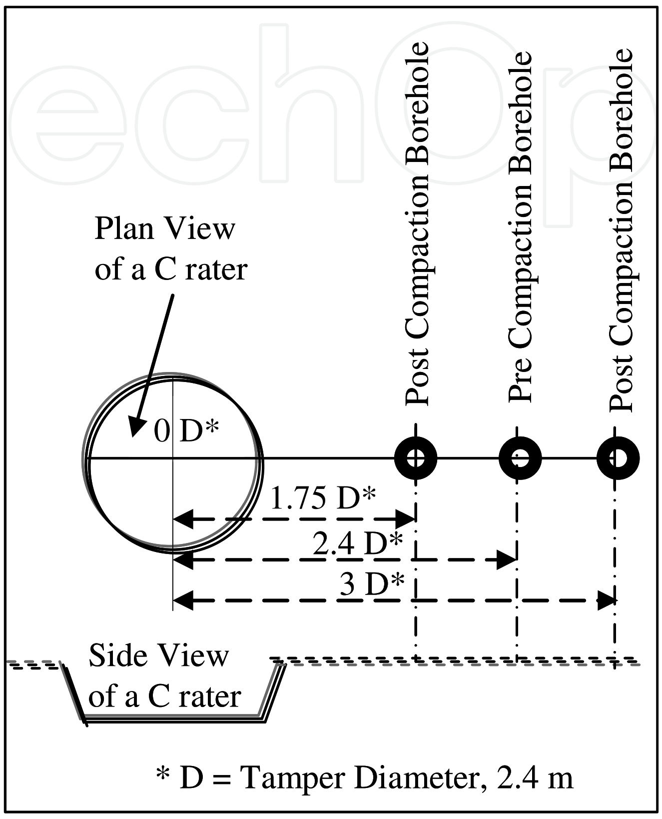

Fig. 7. Location of boreholes for evaluation of improvement in lateral direction

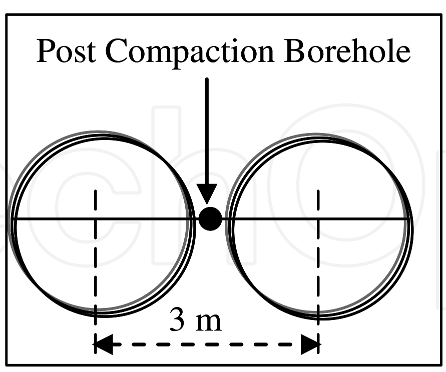

Fig. 8. Location of boreholes for evaluation of improvement at the middle of adjacent craters

6.4.1 Improvement at crater # 1 (upper half) and middle of crater # 1 and crater # 2

Layout of BH for evaluation of improvement under impact point and at the middle of two adjacent craters (crater # 1 and crater # 2) is shown in Figure 9. Pre-to-post compaction SPT profile after full scale compaction for improvement under impact point at crater # 2 and at the middle point of crater # 1 and crater # 2 is shown in Figure 10.

It is evident from Figure 10, that directly under impact point on upper side of crater # 2; significant improvement was noted in top 3.5 m strata. Moderate improvement was observed in strata between depths of 3.5 m to 5 m. Improvement was marginal in strata depth of 5 m to 7 m. Improvement below 7 m depth remained insignificant. At the middle of crater # 1 and crater # 2, improvement was slightly less than the improvement under impact point however still the improvement was significant in upper 3.5 m strata. Improvement below 3 m depth at the middle of these craters was almost same as that of improvement under impact point of crater # 2.

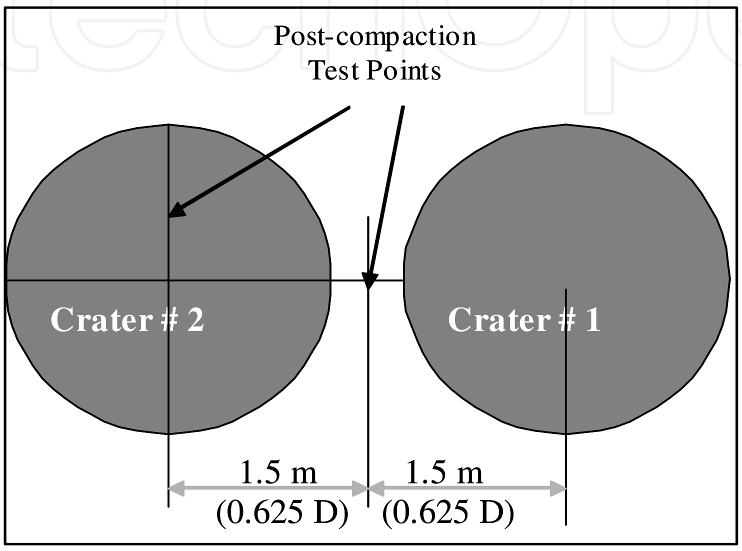

Fig. 9. Layout of BH for Evaluation of Improvement after Full scale Compaction at Crater # 1 (Upper Half) and Middle of Crater # 1 and Crater # 2

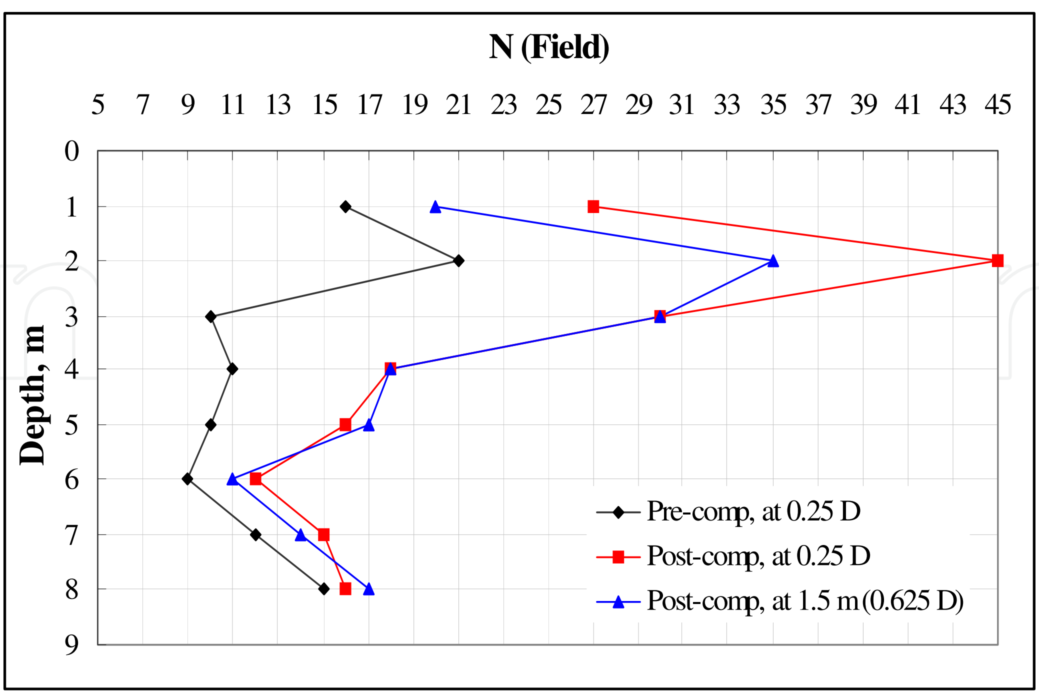

Fig. 10. Pre-to-post Compaction N (Field) Comparison of Improvement at Crater # 1(Upper Half) and Middle of Crater # 1 and Crater # 2

6.4.2 Improvement at crater # 5 (upper half) and middle of crater # 5 and crater # 6

Layout of BH for evaluation of improvement under impact point and at the middle of crater # 5 and crater # 6 is shown in Figure 11. Pre-to-post compaction SPT profile after full scale compaction for improvement under impact point at crater # 6 and at the middle point of crater # 5 and crater # 6 is shown in figure 12.

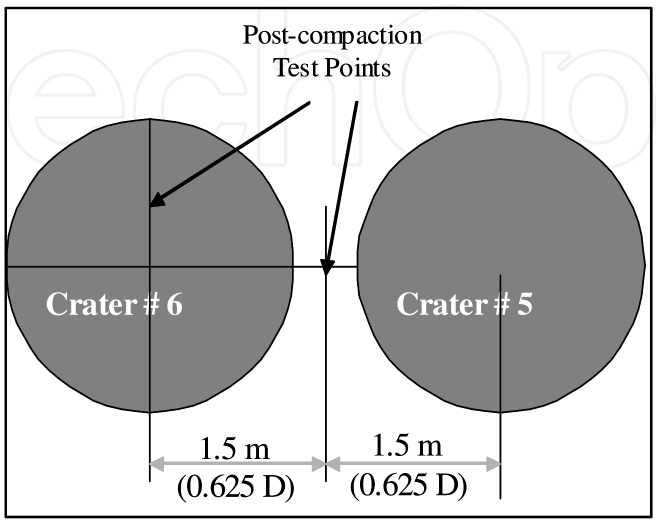

Fig. 11. Layout of BH for Evaluation of Improvement after Full scale Compaction at Crater # 5 (Upper Half) and Middle of Crater # 5 and Crater # 6

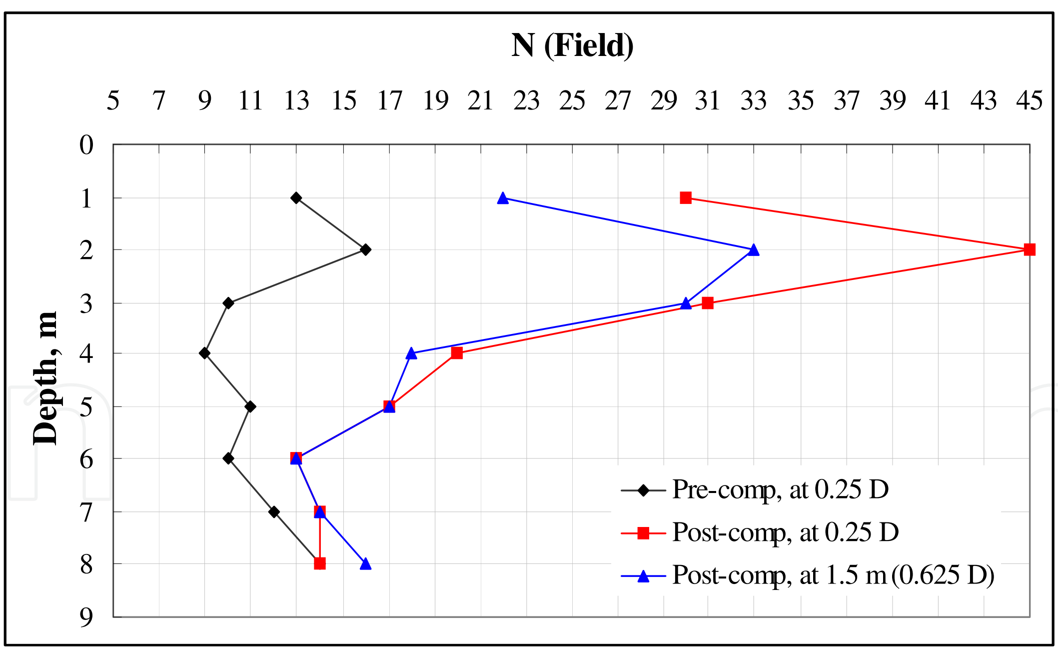

Fig. 12. Pre-to-post Compaction N (Field) Comparison of Improvement at Crater # 5 (Upper Half) and Middle of Crater # 5 and Crater # 6

Under impact point, improvement was significant in upper 4 m strata, moderate in strata depth of 4 m to 5 m and marginal from 5 m to 6 m. At the middle of crater # 5 and crater # 6, improvement was slightly less than the improvement under impact point however still the improvement was significant in upper 3 m strata. Improvement below 3 m depth at the middle of these craters was same as that of improvement under impact point of crater # 6 with slight variation at depths of 4 m and 8 m.

6.4.3 Improvement at crater # 8 (upper half) and middle of crater # 8 and crater # 9

Layout of BH for evaluation of improvement under impact point and at the middle of crater # 8 and crater # 9 is shown in Figure 13. Pre-to-post compaction SPT profile after full scale compaction for improvement under impact point at crater # 8 and at the middle point of crater # 8 and crater # 9 is shown in Figure 14.

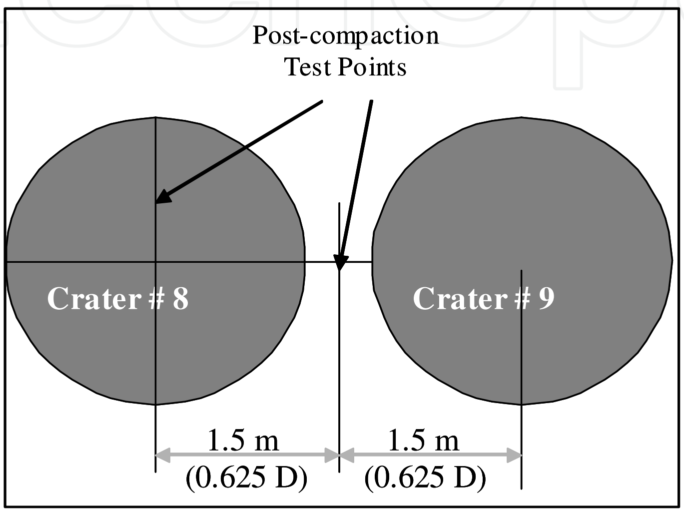

Fig. 13. Layout of BH for Evaluation of Improvement after Full scale Compaction at Crater # 8 (Upper Half) and Middle of Crater # 8 and Crater # 9

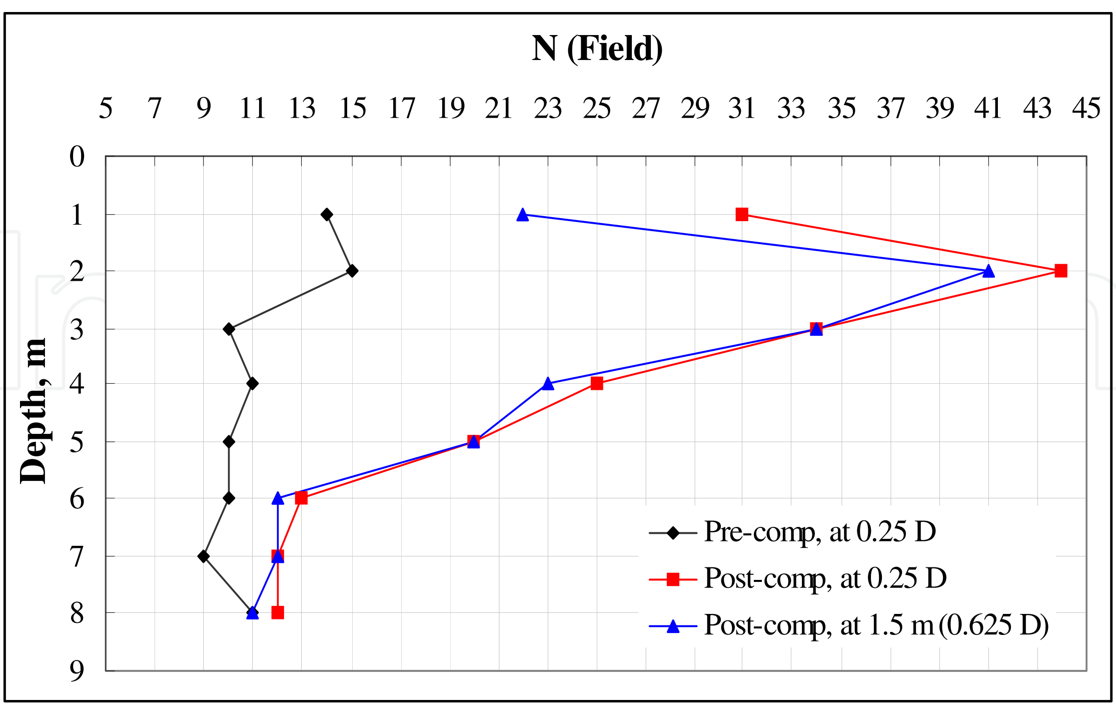

Fig. 14. Pre-to-post Compaction N (Field) Comparison of Improvement at Crater # 8 and Middle of Crater # 8 and Crater # 9 (Figure 5.17)

Directly under impact point on upper side of crater # 8; significant improvement was noted in top 4 m strata. Moderate improvement was observed in strata between depths of 4 m to 5.5 m. Improvement was marginal in strata depth of 5.5 m to 7.5 m. Improvement below 7.5 m depth was insignificant. At the middle of crater # 8 and crater # 9, improvement was slightly less than the improvement under impact point in the upper 2 m strata, still the improvement was significant. Improvement below 2 m depth at the middle of these craters was same as that of improvement under impact point of crater # 8.

6.4.4 Lateral improvement , lower side of crater # 7

Pre-to-post compaction N (Field) comparison after full scale compaction for improvement in lateral direction on lower side of crater # 7 is shown in Figure 15. In lateral direction, improvement was moderate in upper 2.5 m strata and marginal from 2.5 m to 3.5 m strata from centre of impact point to distance of 1.75 D. Improvement was marginal in the upper 1.5 m strata from 1.75 D to 3 D from centre of impact point. Improvement below this depth was insignificant.

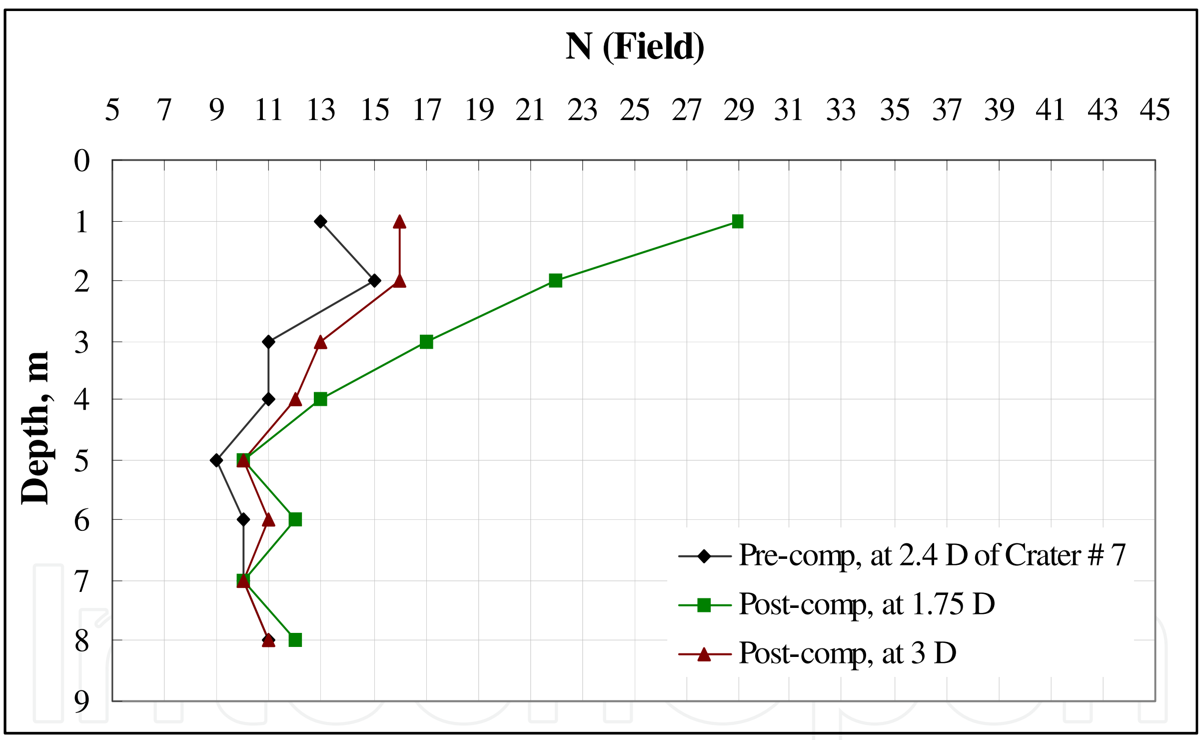

Fig. 15. Pre-to-post Compaction N (Field) Comparison of Lateral Improvement at Lower Side of Crater # 7

6.4.5 Lateral improvement, upper side of crater # 8

Pre-to-post compaction N (Field) comparison after full scale compaction for improvement in lateral direction on upper side of crater # 8 is shown in Figure 16. Improvement was moderate in upper 3 m strata and marginal from 3 m to 5 m strata from centre of impact point to distance of 1.75 D. Improvement was marginal in the upper 1.5 m strata from 1.75 D to 3 D from centre of impact point. Improvement below this depth was insignificant.

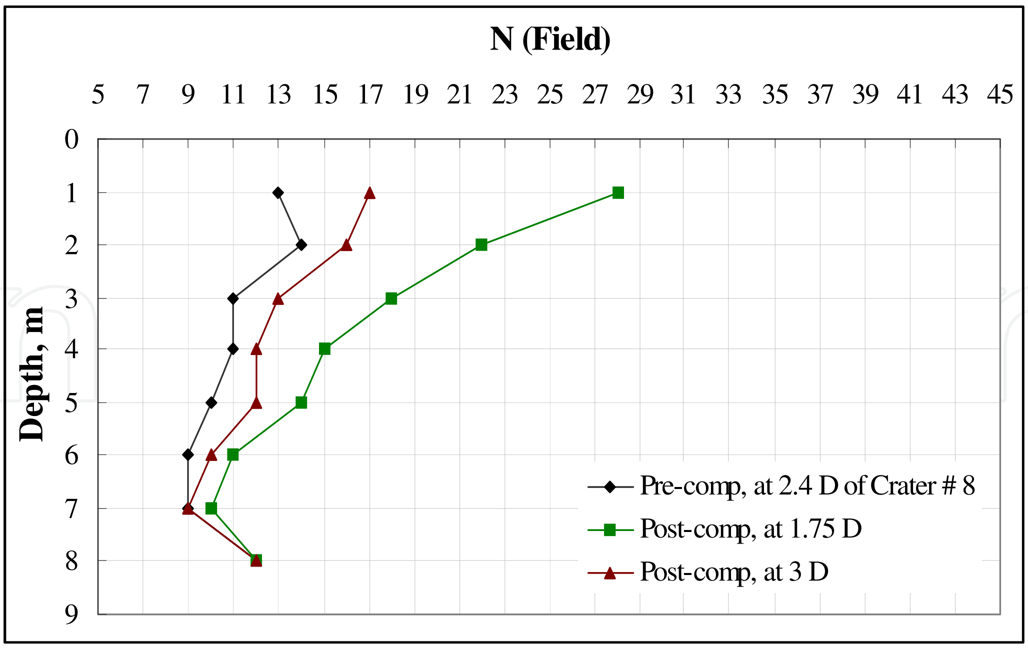

Fig. 16. Pre-to-post Compaction N (Field) Comparison of Lateral Improvement at Upper Side of Crater # 8

7. Quality controls

At the end of field trial, DDC program can be finalized with respect to number of passes, number of drops, grid spacing of all the passes (primary, secondary, tertiary, ironing etc.), and height of fall. During field trial, improvement monitoring mechanism is finalized in terms of crater depth vis-à-vis number of drops, quantum of backfill material, and increase in penetration resistance (SPT, CPT, or DCPT). Since it may not be possible to perform so many penetration tests, more convenient means would still remain crater depth, and back fill material. Author also used pre to post compaction electrical resistivity profiles as a tool to monitor depth and degree of improvement. It is however emphasized that electrical resistivity profile can be tricky in case of rains or increase in moisture conditions at the site. In such circumstances, only an experienced geophysical or geotechnical engineer will be able to interpret profiles. While pre to post compaction resistivity profiles can be compared easily in 2-dimensional perspective, monitoring of improvement through crater depth and back fill will require tabulated format which must be kept at site and filled in carefully.

Consequent to results of field trial of the designed compaction program, 5 cm settlement between any two successive blows was selected as acceptable compaction criteria for achievement of desired improvement. Final crater depths after 5 blows and 10 blows are shown in Figure 17. Observations on crater depths are:

- Almost same trend in crater depths has been observed at all craters
- Crater depths increased with increase in no. of blows
- Crater depths are more for initial 5 blows (around 96 cm) than for next 5 blows (around 37 cm)
- At crater # 7 to crater # 9, crater depths are more for initial 5 blows and less for next 5 blows.

- Slightly more crater depths for 5 blows at crater # 7 to crater # 9 are because strata at this location were relatively loose

Crater depths of research project have been plotted and compared with normalized crater depths of DDC case histories of non-collapsible soils proposed by Mayne et al., – 1984 as shown in Figure 18. Comparison of crater depths indicates that crater depths of research project are towards lower limit of DDC case histories. Since relationship of normalized crater depths and no. of blows is nonlinear therefore best fit curve has been obtained using a second order polynomial equation, given in Figure 18.

Relationship of crater depths and depth of improvement of research project was nearly linear as shown in Figure 19; however the depth of improvement was more for larger crater depths. A best fit line was obtained with linear equation (equation 5) where depth of improvement was dependant variable and crater depth was independent variable. The equation of the best fit line gives somewhat higher values of depth of improvement for crater depths of 0.5 m to 1 m and lower depth of improvement at crater depths of more than 1.5m. In view of the lower depth of improvement of project after full scale compaction, the

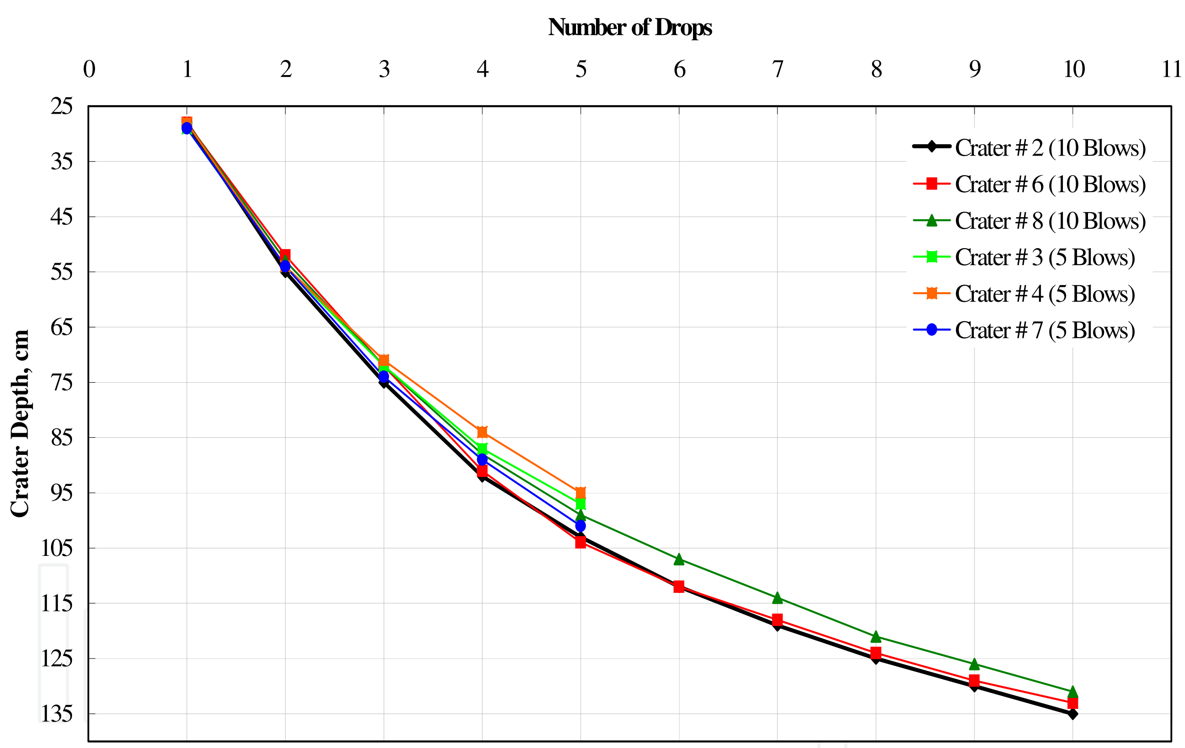

Fig. 17. Crater Depth Measurements after 5 and 10 Blows

results of equation 5 would be towards lower bound of the expected range of improvement of a similar compaction program under similar soil conditions. Since R^2 value of the equation is 0.98, equation 51 shows a good correlation.

$$D = 3.7\,d - 0.11 \tag{5}$$

Where,
D = Depth of improvement, m
d = depth of crater, m

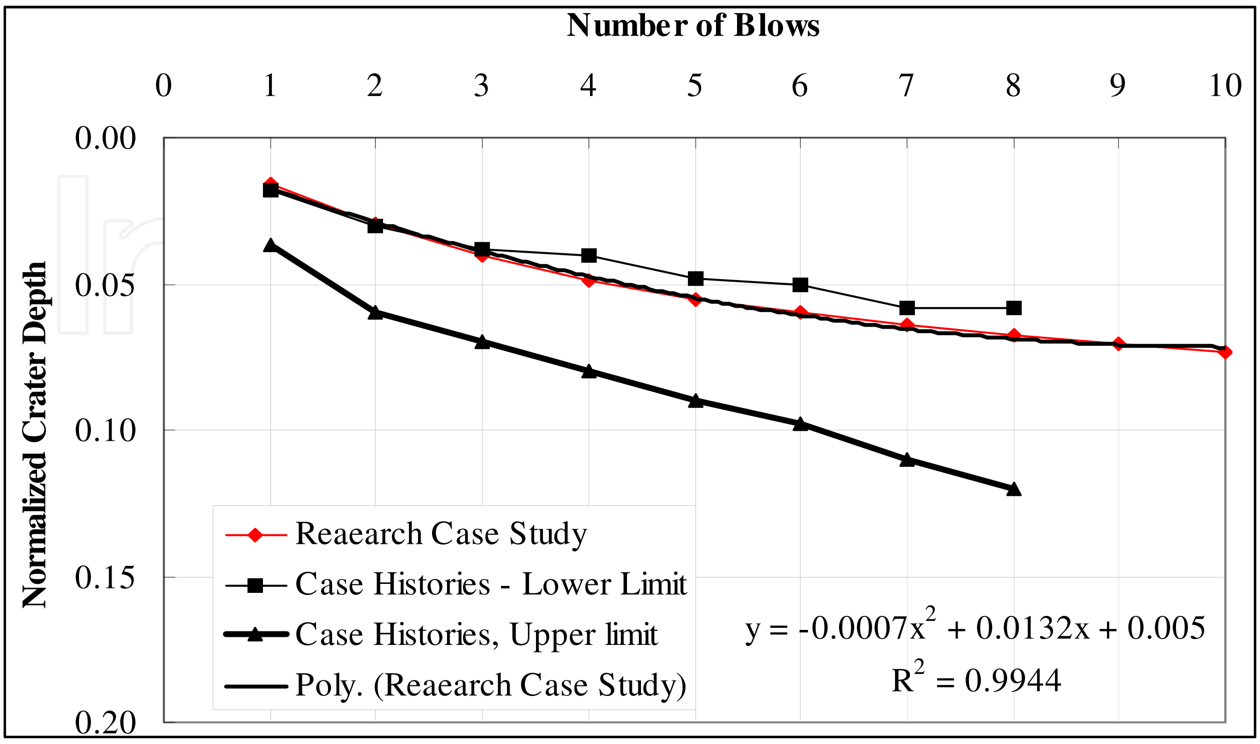

Fig. 18. Comparison of Normalized Crater Depths Vs No. of Drops with the Range proposed by Mayne et al., (1984)

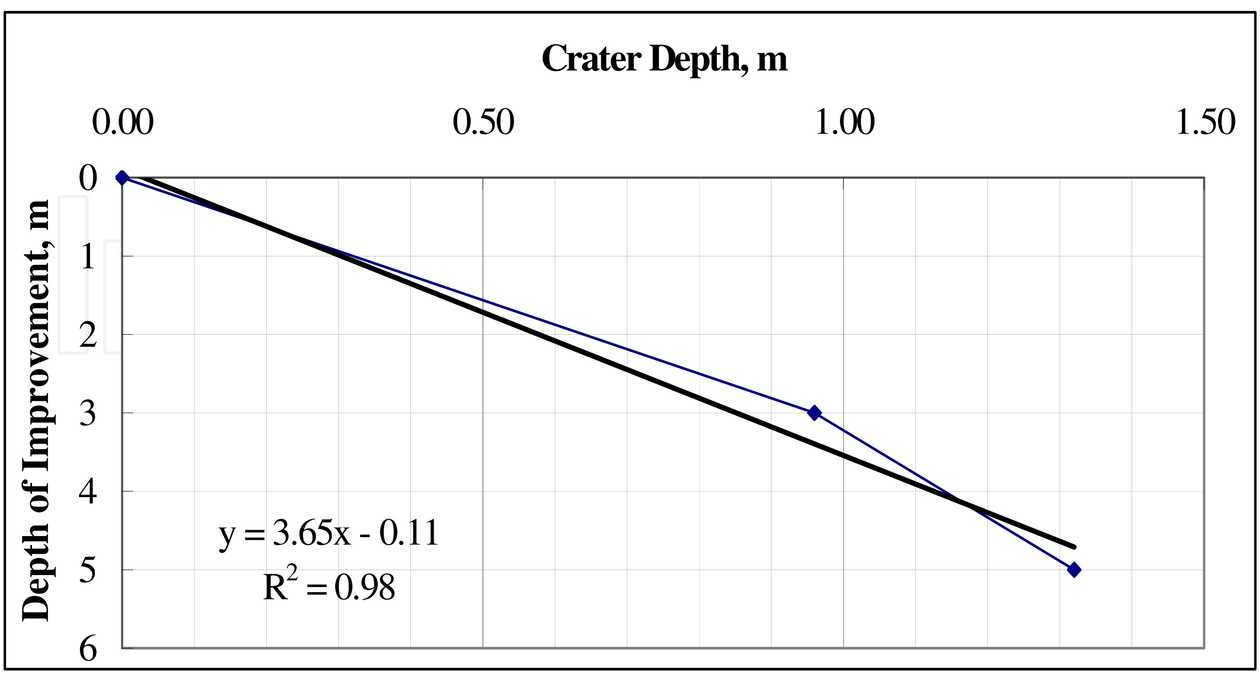

Fig. 19. Plot of Crater Depths vs. Depth of Improvement

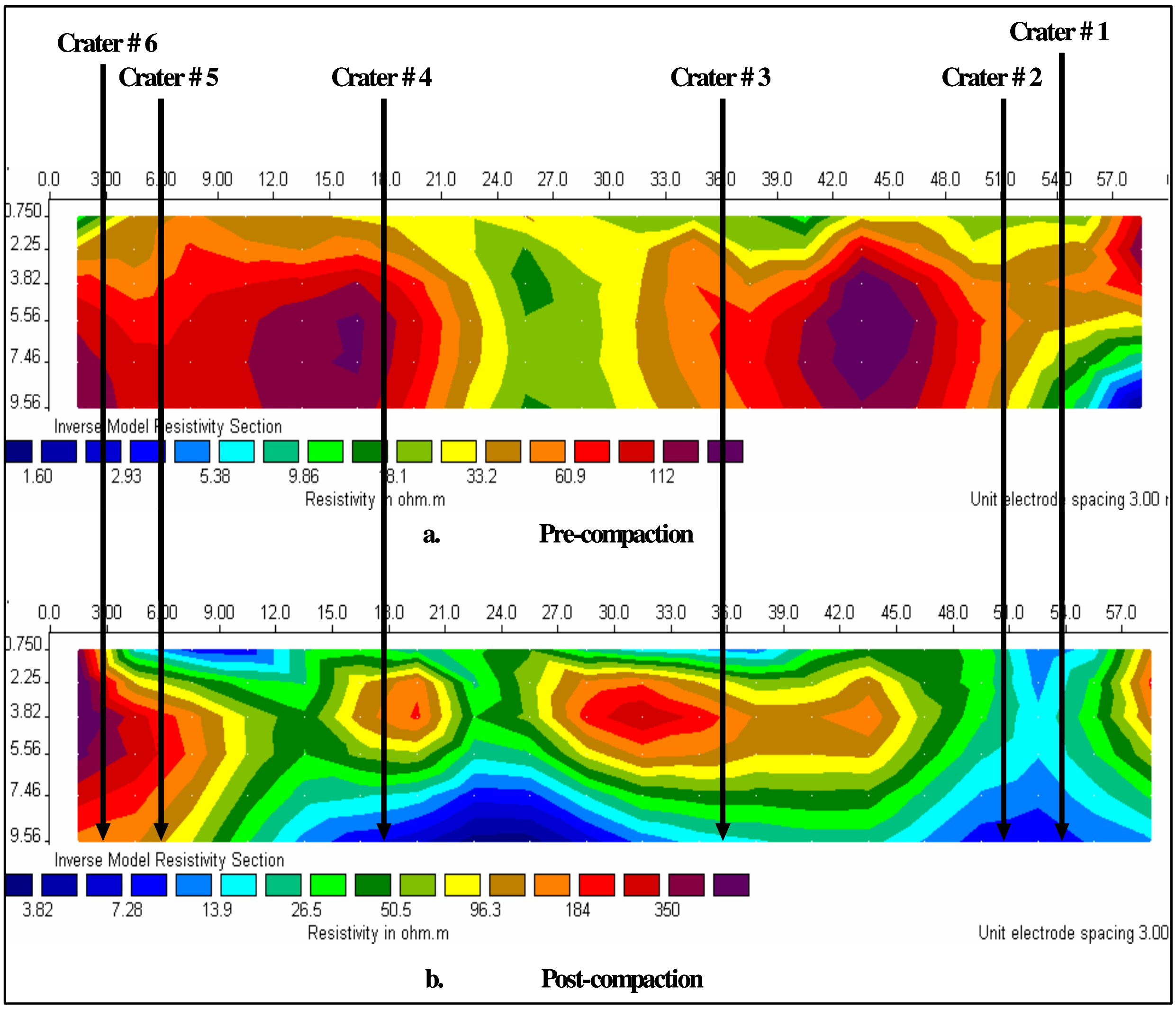

Fig. 20. ERT Profiles, Crater # 1 to Crater # 6

7.1 Evaluation of improvement from ERT profiles

Pre-to-post compaction ERT profiles of strata at crater # 1 to crater # 6 are shown in Figure 20. The profiles are based on relative densities of various layers in the strata therefore the colour legend of pre and post-compaction profiles indicate different resistivity ranges. At crater # 1 to crater # 6, over all pre-compaction resistivity range of 1.60 Ω-m to 112 Ω-m increased to post-compaction range of 3.82 Ω-m to 350 Ω-m.

Pre-to-post compaction ERT profiles of strata at crater # 7 to crater # 9 are shown in Figure 21; the overall increase in resistivity was more in this case. Pre-compaction resistivity range of 0.370 Ω-m to 509 Ω-m increased to post-compaction resistivity range of 6.34 Ω-m to 1054 Ω-m. Salient of pre-to-post compaction ERT profiles are:

- Post-compaction ERT profiles confirm in general, the 5 m depth of improvement evaluated from SPT
- Relatively loose strata are also indicated at crater # 1, crater # 2 and crater # 8
- Keeping in view the same nature of strata and SPT results at all other craters, presence of loose strata could be attributed to some error in recording of resistivity data

- More no. of BH could have better explained the variation in resistivity in between test craters

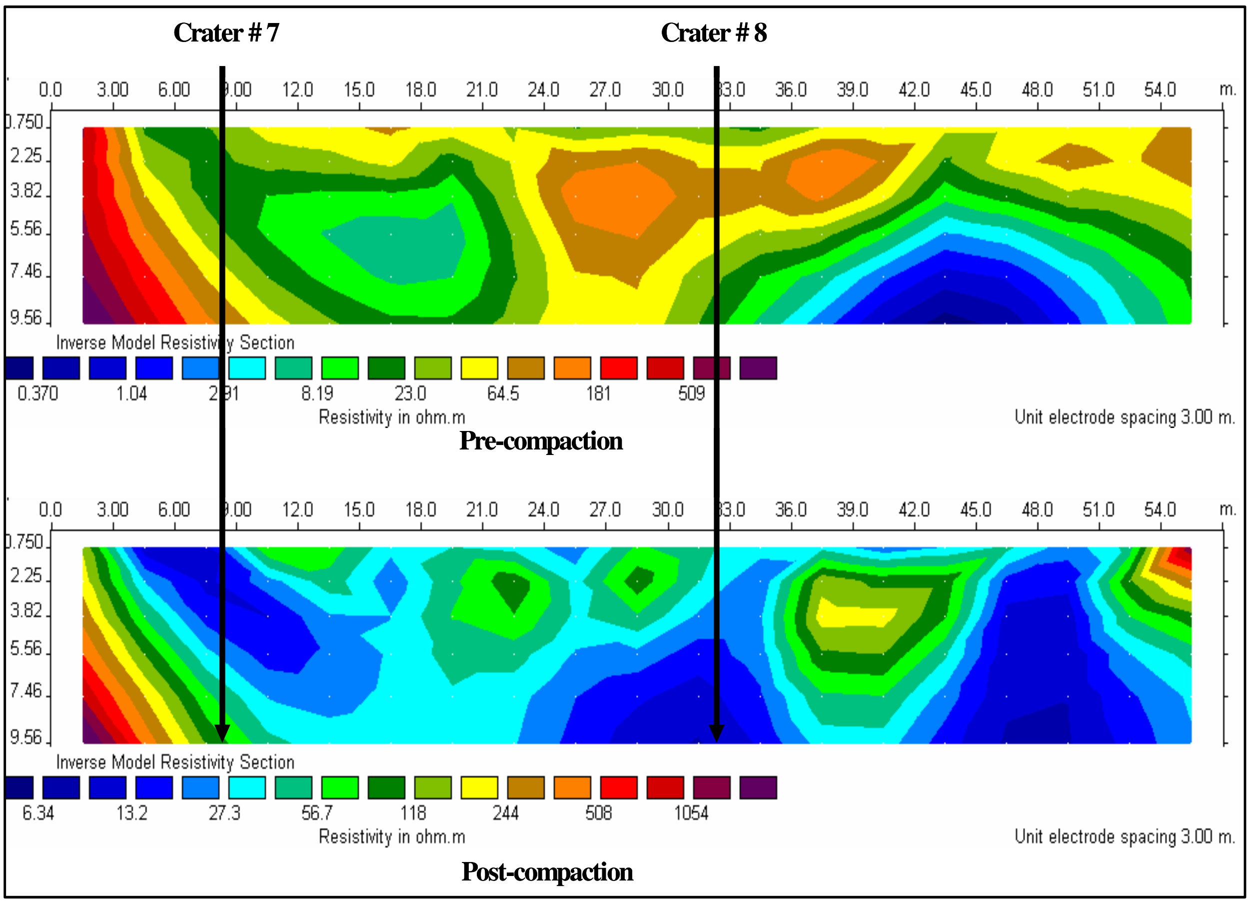

Fig. 21. ERT Profiles, Crater # 7 to Crater # 8

8. Ground vibrations

Ground vibrations from dynamic compaction can be dangerous to the adjacent structures and therefore needs to be determined prior to compaction operation. Effect of vibrations on various structures and human beings is shown in Table 3 and 4. Vibrations were predicted using Scaled Distance Approach proposed by Mayne (1985). To assess attenuation effect of this trench, vibrations were measured across the trench using "Linear Variable Displacement Transducer (LVDT)"-an approach analogous to "Impulse Response Function" approach proposed by Svinkin (1996a, 1997), refer to Figure 5. In a dynamic compaction project in Pakistan, a 2 meter wide and 4 meter deep trench was excavated at the edge of the compaction site to reduce vibrations to avoid any damage to a building some 50 meters away from the site. Vibrations were measured across the trench at a distance of 25 m to assess the attenuation effect of trench and compare with the value obtained from empirical correlations. Vibrations were also measured on compaction site after the primary pass to assess the effect of primary impacts on the magnitude of vibrations.

Linear Variable Displacement Transducers (LVDT) and accelerometer was used for measurement of vibrations. These were installed on top of 1 inch dia, 1.25 m long -

deformed steel bar. The steel bar was drilled into ground with only the top surface visible at the ground surface 2 inch thick foam was placed under the base of the LVDT assembly to minimize inertia effect of the LVDT assembly. Layout of the test is shown in Figure 22 and 23. Data was recorded for 10 consecutive drops at a distance of 25 m from point of impact across the trench and at distance of 25 m at place where primary pass was completed. The data recorded was numerically differentiated to obtain peak particle velocity (PPV) and was further differentiated to obtain acceleration.

Type of Structure and their Natural Frequencies	Safe level of Vibrations (mm/s)
Reinforced or framed structures, industrial and heavy commercial buildings at 4 Hz and above	50
Un-reinforced or light framed structures, residential or light commercial type buildings at 4 Hz –15 Hz	15-20
Un-reinforced or light framed structures, residential or light commercial type buildings at 15 Hz –40 Hz and above	20-50

Table 3. Effect of vibrations on structures (British Standard 7385: Part 2-1993).

Level of Vibrations (mm / sec)	Effect of Vibration
0.1	not noticeable
0.15	nearly not noticeable
0.35	seldom noticeable
1.00	always noticeable
2.00	clearly noticeable
6.00	strongly noticeable
14.00	very strongly noticeable
17.8	severe noticeable

Table 4. Effect of vibrations on human beings (British Standard 7385).

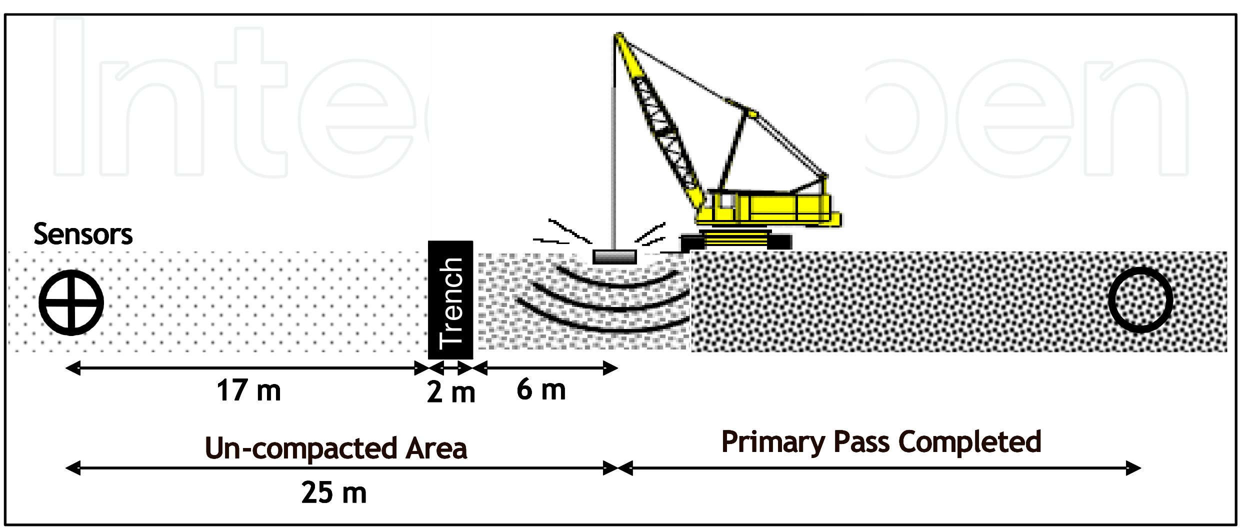

Fig. 22. Layout of Vibration Measurement Points at 25 m Distance

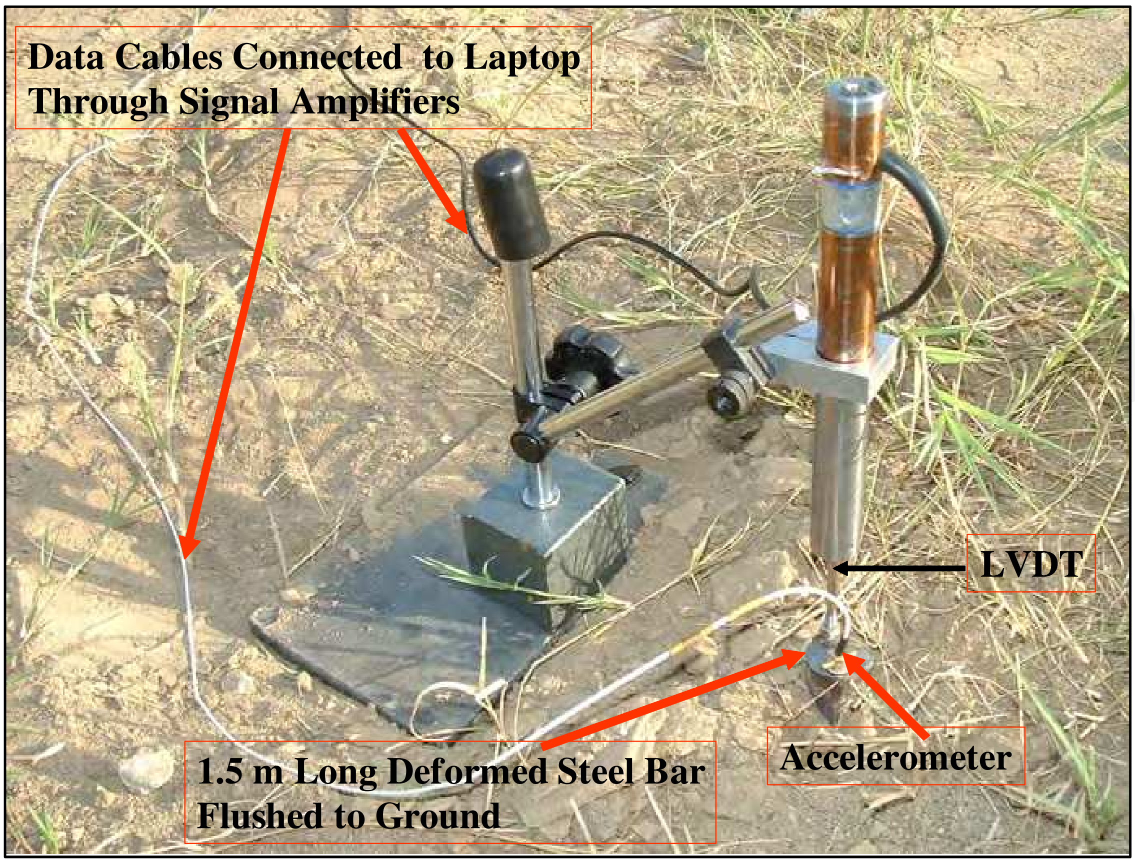

Fig. 23. Installation arrangement of LVDT and accelerometer

The vibrations are estimated through empirical correlations or measured with the help of instruments such as portable seismograph, accelerometers, velocity transducers, linear variable displacement transducers (LVDT), etc. Mayne et al. (1984) proposed an empirical relationship between PPV and inverse scaled distance to estimate minimum and maximum range of PPV against various scaled distances and is shown in Figure 3. Inverse scaled distance is square root of the compaction energy, $\sqrt{WH}$ divided by the distance, d from the impact point. From Figure 24, a minimum PPV of 7 mm/sec and maximum PPV of 70 mm/sec were estimated at a distance of 25 m from point of impact.

Empirical Equation (6), proposed by Rollins and Kim – 1992 [6] for estimation of the PPV, is based on field monitoring data of several dynamic compaction projects. PPV in these projects was measured using portable seismograph. The frequency of these vibrations ranged from 5 - 40 Hz. At a distance of 25 m away from impact point, a PPV of 14.16 mm/sec is estimated using Eq. (6).

$$PPV = 20\left(\sqrt{WH}/d\right)^{1.03} \qquad (6)$$

Where,
PPV = Peak Particle Velocity, mm/sec
W = Weight, tons
H = Height of fall, m
d = Distance from point of impact, m

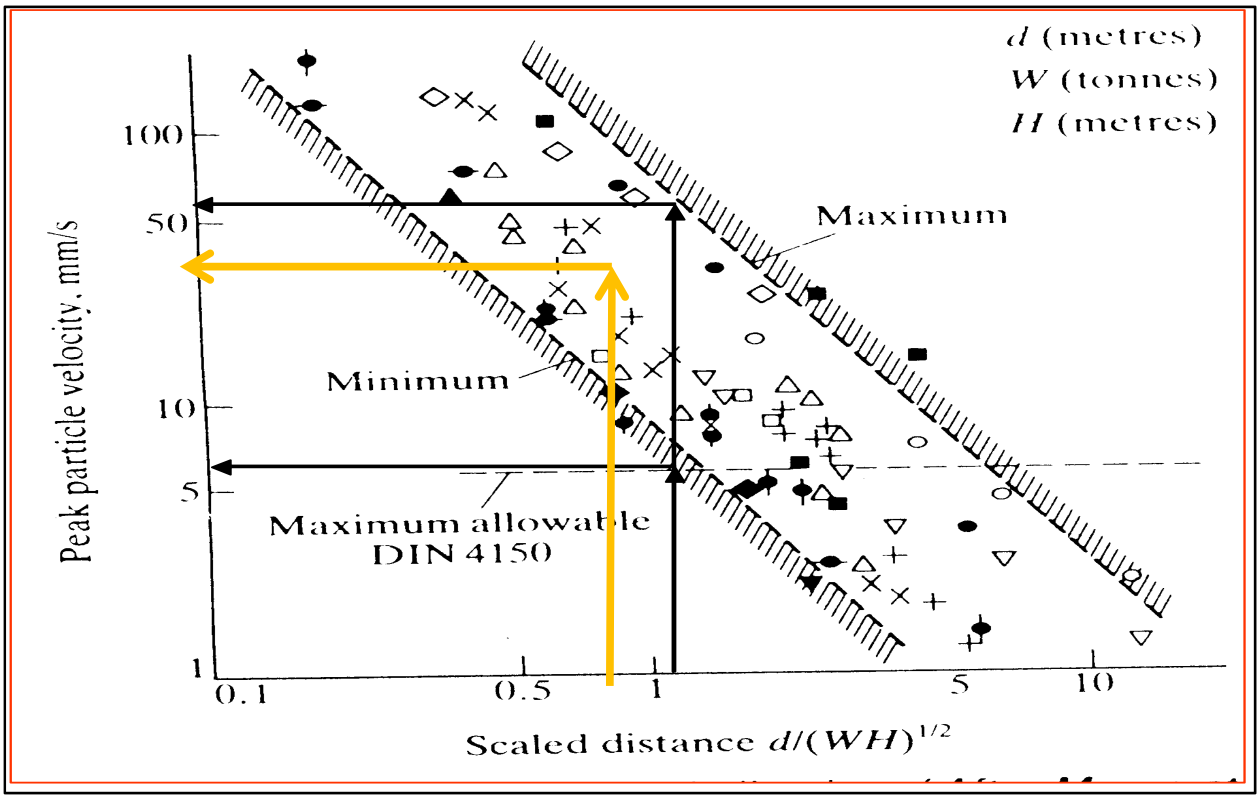

Fig. 24. Peak Particle Velocity (PPV) Vs Scaled Distance, Estimated PPV of Research Project shown in Yellow Lines

8.1 Across isolation trench

Across isolation trench, maximum PPV of 22.5 mm / sec was measured at 25 m distance from the point of impact as shown in Figure 25. This maximum velocity was obtained at 9th consecutive drop. PPV could not be measured at distance of 50 m point either due to very weak signals received by sensors or some malfunctioning of sensors.

8.2 Over partially compacted site

Over partially compacted site (a place where primary pass of compaction was completed), a maximum PPV of 14.1 mm / sec on 10th consecutive drop was calculated at a distance of 25 m away from point of impact as shown in Figure 25 and Table 5 and accelerogram is shown in Figure 26 and 27. Vibrations at a distance of 50 m away from impact point were very low and are not mentioned. The best fit curve is given by exponential equation. Increase in PPV at 25 m point across trench with each successive drop indicates that the PPV increased with increase in soil density due to DDC. An overall decrease in PPV after completion of first pass indicates that the initial pass created several denser / loose soil mediums. The primary impact points at grid spacing of 6 m centre to centre with un-compacted soil in between caused almost 50 % attenuation of PPV.

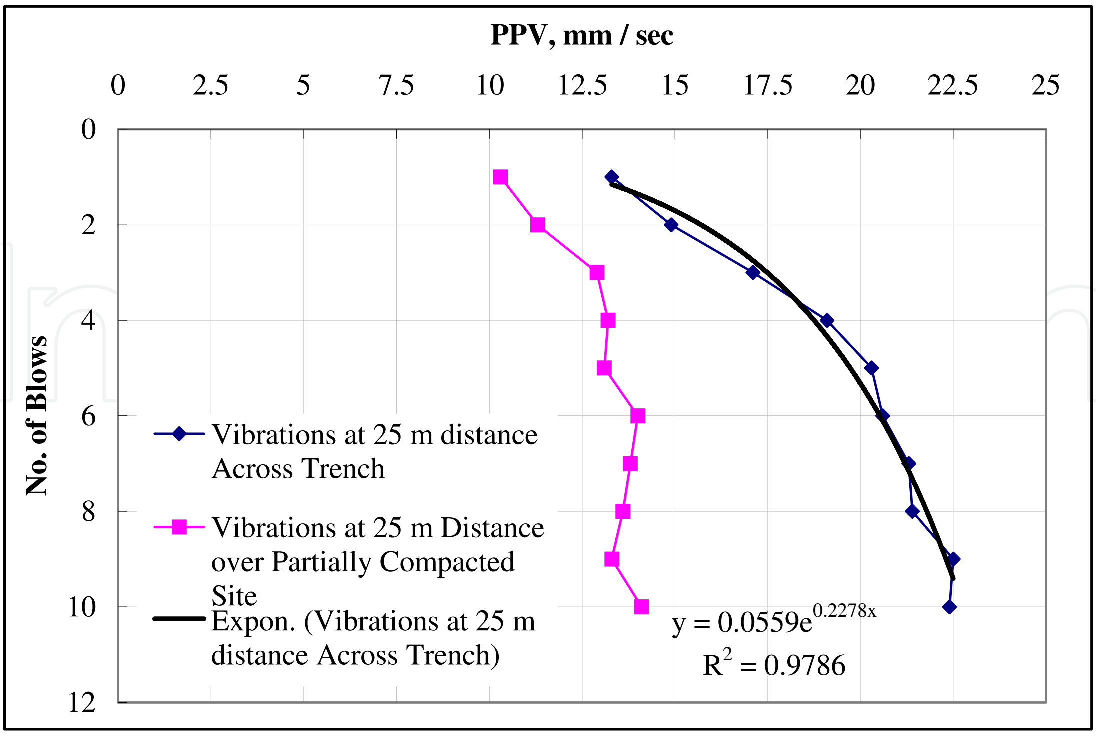

Fig. 25. Measured Ground Vibrations

No of Blow	PPV Across Trench (at 25 m Distance) mm/sec	PPV Over Partially Compacted Site (at 25 m Distance) mm/sec
1	13.3	10.3
2	14.9	11.3
3	17.1	12.9
4	19.1	13.2
5	20.3	13.1
6	20.6	14
7	21.3	13.8
8	21.4	13.6
9	22.5	13.3
10	22.4	14.1

Table 5. Range of PPV for Various Numbers of Blows

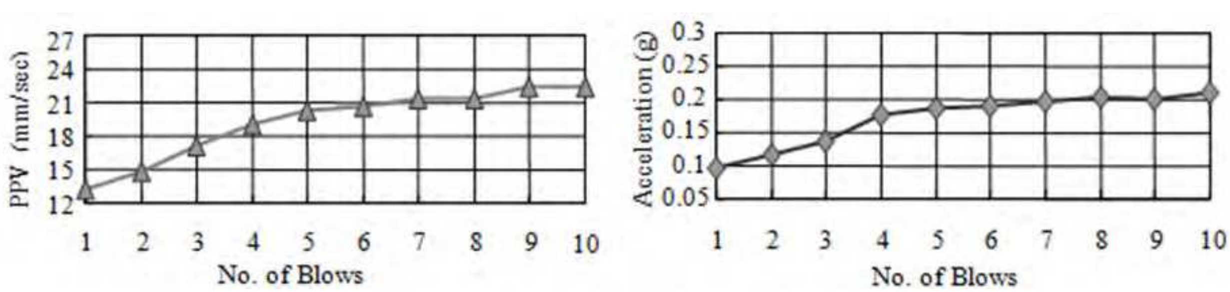

Fig. 26. Accelerograms for successive blows

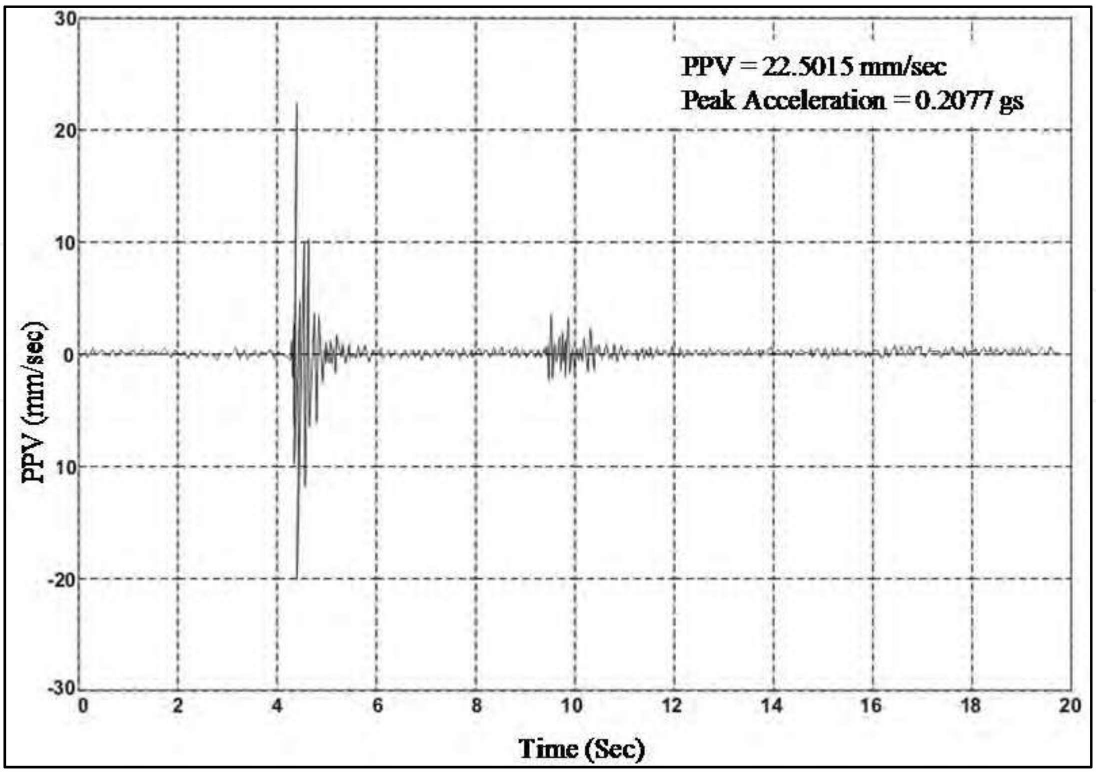

Fig. 27. Accelrogram of PPV at 10th blow

DDC technique can remediate liquefaction potential and enhance bearing capacity up to desired limits at desired depths. Factor of safety evaluated after DDC is given in Table 6 which shows an increased value of factor of safety at three different depths.

Depth (m)	$(N_1)_{60}$	CSR_L	σ'_{vo} (kN/m^3)	$\tau_{cyc,L}$.	τ_{cyc}	FS_L
3.5	23	0.24	71.43	17.14	7.46	2.3
5.5	14	0.17	82.12	13.96	9.46	1.5
7.5	12	0.15	92.81	13.92	11.41	1.2

Table 6. Cyclic Shear Resistance of Soil after Dynamic Compaction and Factor of Safety (FS_L)

9. Conclusion and recommendations

In the light of research conducted for remediation of liquefaction hazard, improvement under impact points, lateral direction after 5 blows, 10 blows and full scale compaction, following conclusions are presented:

- directly under impact points, the bearing capacity of soil improved to 160 kPa upto a depth of 5 m.
- maximum improvement under impact points occurred at depths from 2 m to 4 m.
- sharp decrease in improvement is observed below 4 m depth.
- improvement was negligible below 6.5 m depth.
- with the increase in no. of drops from 5 blows to 10 blows, the degree of improvement also increased from maximum 35 blows to 45 blows.

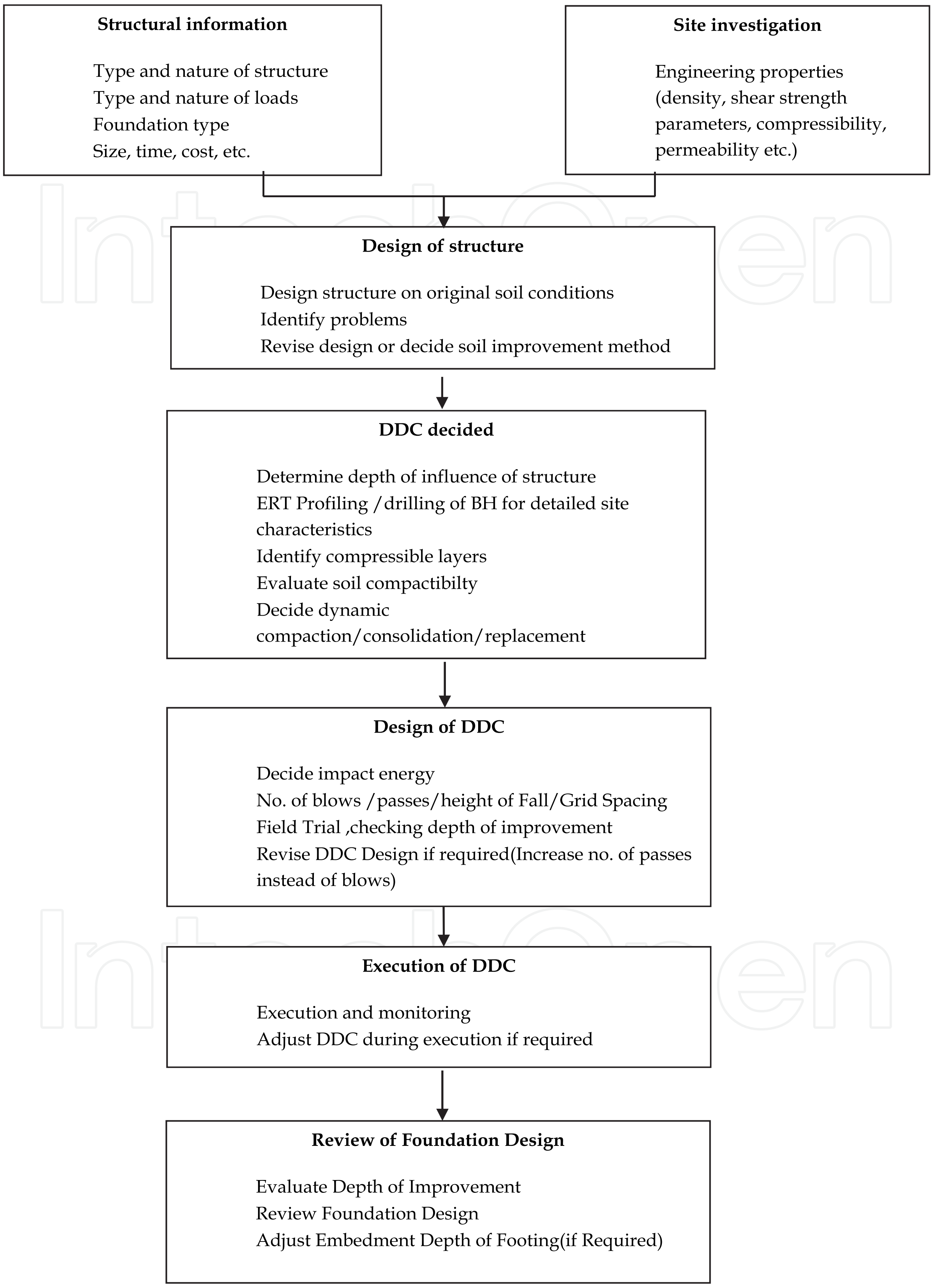

Fig. 28. Suggested Procedures for DDC Project

- increase in no. of blows from 5 to 10, had negligible effect on degree of improvement below 6.5 m depth.
- in upper 2 m of strata, improvement at the middle of any two adjacent impact points was comparatively less than improvement under the impact point.

From results of soil improvement project of alluvial soils, it is concluded that deep dynamic compaction technique can remediate liquefaction potential and enhance bearing capacity upto desired limits at desired depths. The compacted layers at top of strata and reduced grid spacing can significantly reduce the overall depth of improvement. In the light of this research project, a flow chart in Figure 28 is suggested for more efficient and effective DDC program.

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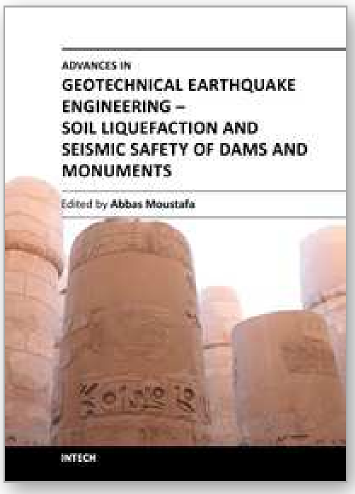

Advances in Geotechnical Earthquake Engineering - Soil Liquefaction and Seismic Safety of Dams and Monuments
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This book sheds lights on recent advances in Geotechnical Earthquake Engineering with special emphasis on soil liquefaction, soil-structure interaction, seismic safety of dams and underground monuments, mitigation strategies against landslide and fire whirlwind resulting from earthquakes and vibration of a layered rotating plant and Bryan's effect. The book contains sixteen chapters covering several interesting research topics written by researchers and experts from several countries. The research reported in this book is useful to graduate students and researchers working in the fields of structural and earthquake engineering. The book will also be of considerable help to civil engineers working on construction and repair of engineering structures, such as buildings, roads, dams and monuments.