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SOIL IMPROVEMENT USING HEAVY TAMPING – A CASE HISTORY

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ABSTRACT

Heavy tamping provides a quick and economical method of densifying in-situ soil deposits. The method has been used with advantage in sands, gravels and silts, and soils with low plasticity. This method with some modifications was recently used to improve the soils at a site in the flood plain of a major river in the United States. The soils at the site consisted of sand from depths of 3 m below ground level to depths of more than 22 m. However, the soils from depths of 3 m to approximately 12 m were found to be loose to medium-dense and susceptible to liquefaction in the case of a major seismic event. It was determined that the conventional method of dynamic compaction alone will not be able to achieve improvements to required depths of 12 m. A combination of heavy tamping and short stone column installation was therefore used. The improvement in soil was monitored by conducting SPT tests before compaction, after heavy tamping, and finally after installation of short stone columns. Subsequent analysis for liquefaction susceptibility demonstrated significant reduction in potential for liquefaction.

KEYWORDS: Compaction, Heavy tamping, Liquefaction, Remediation, Stone Column

INTRODUCTION

Soil improvement by compaction has been used since ancient times. However, the methods of compaction and equipment used have seen a lot of improvements during the last three decades. Densification of soils has been identified as a means of improving the soils for supporting structural loads as well as a remedial measure against soil liquefaction due to seismic shaking. The method used for compaction depends on the size of the area involved and depths to which densification may be required. Densification of existing deposits may usually be achieved by heavy tamping, vibroflotation, construction of stone columns and compaction piling (NRC, 1985). The decision to use any particular method depends on the required benefits and engineering judgment. Combination of methods may be used at a site to enhance the effectiveness of compaction. Heavy tamping has been mainly used for deep compaction in cohesionless soils. However, fine-grained soils with plasticity index of less than 8 are also considered suitable for heavy tamping. Once the decision to use heavy tamping has been made, proper selection of the "tamper", design of the grid for tamping locations, number of drops or passes, and pre-and post-compaction monitoring are planned. The heavy tamping itself involves a deep compaction phase known as area pass (also referred to as high-energy phase), and a shallow compaction phase known as ironing (also referred to as low-energy phase). The effective depth of dynamic compaction is generally determined by the following equation (Lukas, 1995):

$$D = n\sqrt{WH} \tag{1}$$

where

D = depth of improvement in m

W = mass of tamper in mega grams

H = drop height in m

n = empirical coefficient with typical values ranging from 0.3 to 0.8

The value of n for pervious deposits and granular soils ranges from 0.5 to 0.6 and is generally taken as 0.5. From Equation (1), it appears that the soil can be improved to any depth by using a heavier tamper and higher values of drop. However, the depth of effective compaction is practically limited because too deep craters form stiffer layers near the compaction surface restricting the depth to which energy is effectively transferred into the soil. In granular soils, the practical depth of compaction due to heavy tamping is approximately 7.5 to 9 m from the compacting surface (Lukas, 1995). Figure 1 provides approximate depths of improvement measured by FHWA (1986).

The effectiveness of compaction is generally monitored by conducting either Standard Penetration Tests (SPT) or Cone Penetration Tests (CPT). This paper presents results of a field study of heavy tamping enhanced by construction of stone columns at a site in the United States which resulted in improvement in site soils to depths greater than typical depths of improvement with heavy tamping alone. The compaction was monitored by performing standard penetration tests before and after soil improvements.

The site where the soil improvement was done is located in the floodplain of a major river in United States. The project consisted of construction of a five-story office building with a plan area of approximately 3,700 square meters. Maximum estimated normal load on a column was approximately 2,700 kN. The ground surface at the time of construction was relatively flat, with an elevation change of less than one meter. Finished floor level was about 0.6 m above the existing natural ground level. The closest existing structure was more than 1000 feet away from the site.

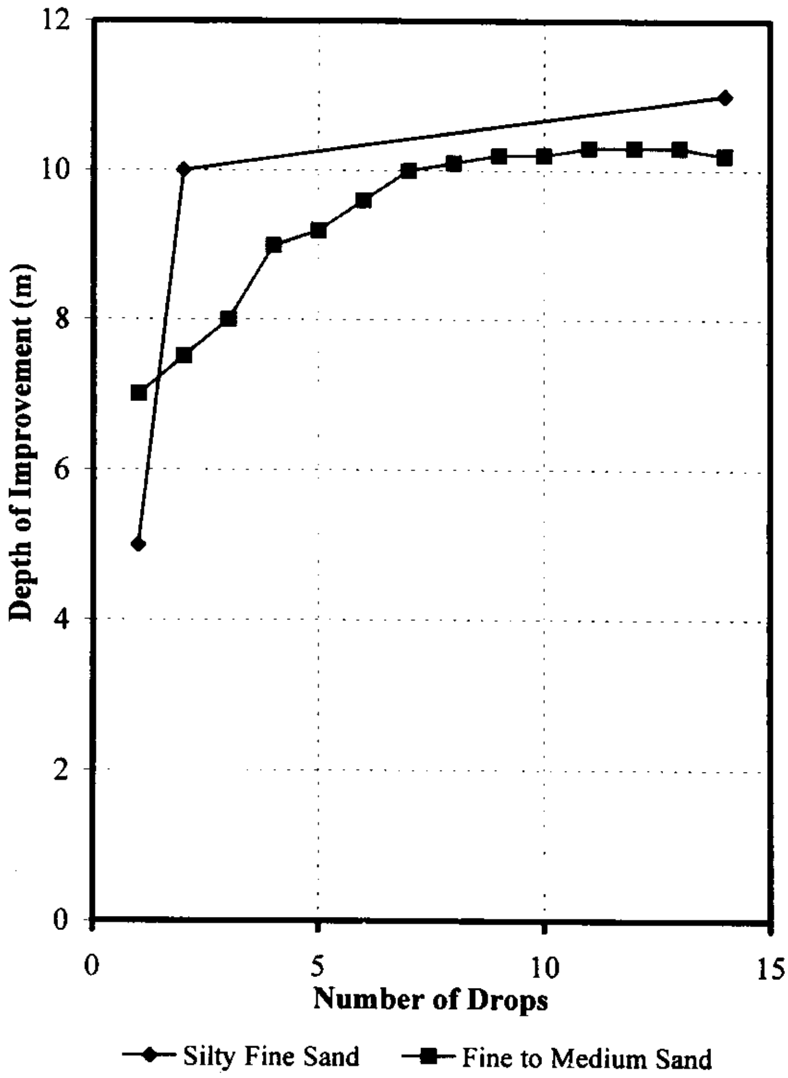

Fig. 1 Depth of improvement for two different soils (FHWA, 1986)

SUB-SOIL CONDITIONS

The preliminary sub-surface exploration was conducted by drilling several boreholes within the footprint of the building. Soil stratigraphy at the site consisted of a thin layer of fill underlain by a comparatively thick sequence of floodplain alluvium and bedrock. Fills in the planned building area included weathered shale and silty clay to clay to depths of approximately 0.6 to 1 m. The natural soils at the site consisted of three distinct strata: medium stiff, silty clay to clayey silt with occasional traces of sandy silt and silty sand; loose, fine to medium sand; and medium-dense to dense, fine to coarse sand with traces of gravel. The uppermost soil stratum of fine-grained soils was approximately 3 m thick. The loose, fine sands were encountered to depths of approximately 12 m. An approximately 1.5 m thick layer of medium-dense, fine sand was encountered at depths of 10.5 m from the ground surface, in all the borings drilled. Groundwater at the site is greatly influenced by water levels in the nearby river. Groundwater at the time of sub-surface exploration was encountered at the interface of fine-grained soils and sand which was approximately 3 m below the ground surface. A typical soil profile is shown in Figure 2.

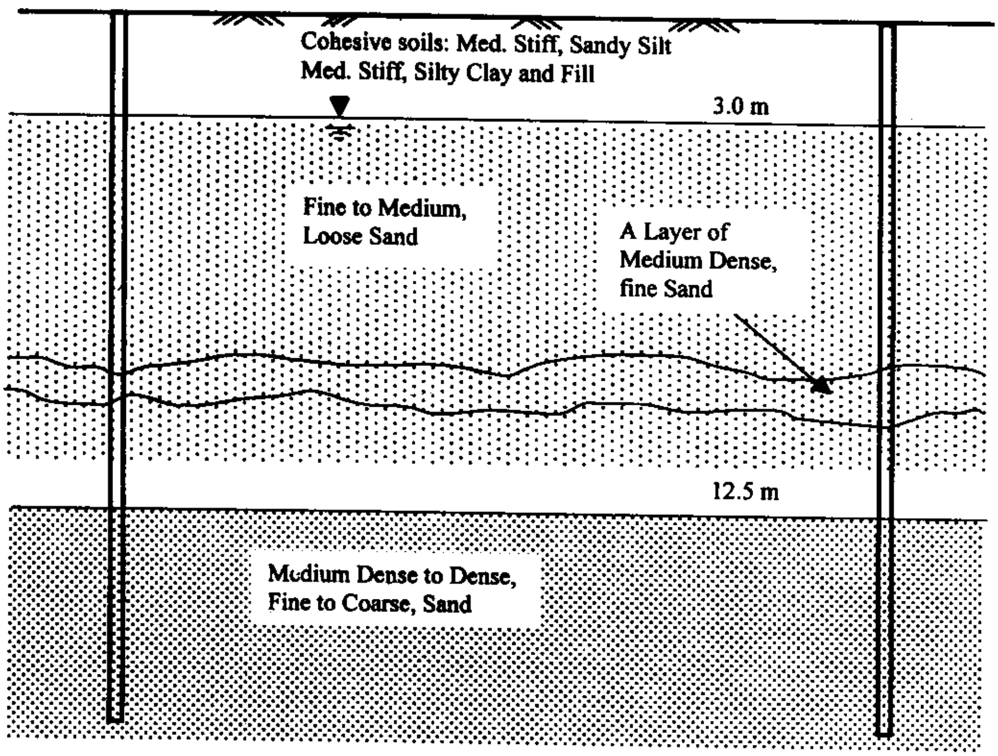

Fig. 2 A typical soil profile at the site (not to scale)

PURPOSE OF GROUND IMPROVEMENT

Borings drilled at the site as a part of initial soil investigation showed that the existing soils to depths of approximately 12 m are loose sands having a uniform grain size distribution. Groundwater was encountered at a depth of approximately 3 m at the time of exploration, which fluctuates depending on the water levels in the nearby river.

Typical grain size distribution curves of site soils along with boundaries for most liquefiable soils recommended by Tsuchida (1970) are shown in Figure 3. As can be seen from Figure 3, the grain-size distribution of the site soils falls within the zone of most liquefiable soil. Due to the presence of low density, saturated sands having relatively uniform grain size distribution, and level of ground shaking expected at the site from an earthquake, it was concluded that the site had potential for liquefaction. Therefore, the liquefaction analysis was performed to determine the density of sands required to minimize the potential of liquefaction. These densities were then compared with the densities of the existing soils to determine the magnitude of liquefaction potential of the site and the level of site improvement needed.

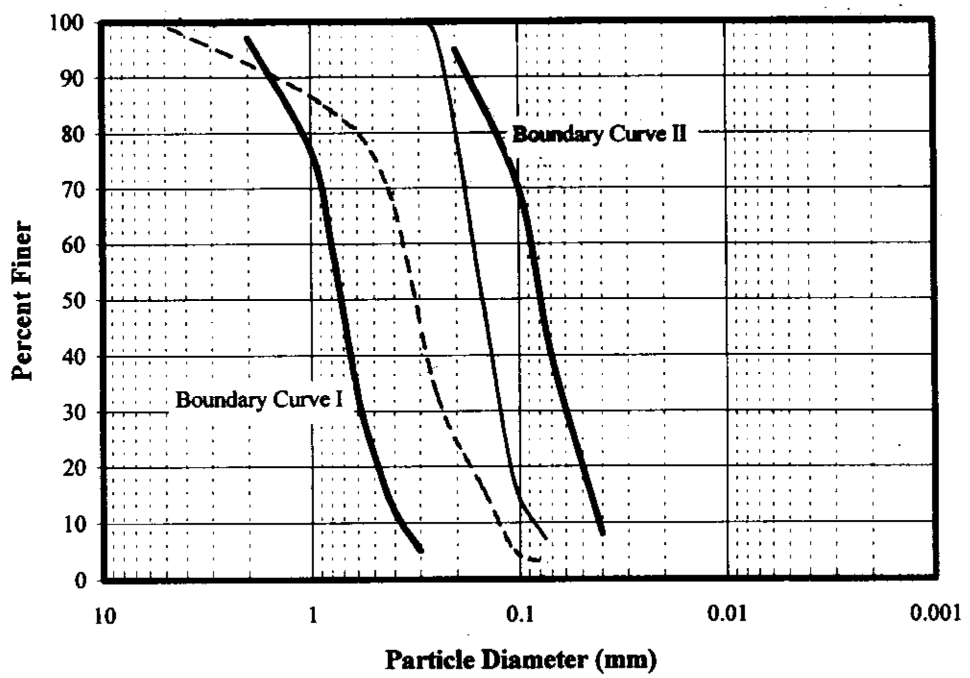

Fig. 3 Typical grain-size distribution curves and boundary curves for most liquefiable soils

Uncorrected blow counts (N-Values) measured in the borings drilled at the site along with blow counts required to reduce liquefaction potential are shown in Table 1. The measured N-values which are lower than those required to avoid liquefaction are shown in italics and bold font. Although N-values higher than those required to avoid liquefaction were observed in some of the borings at various depths, when looked at spatial variation of N-values, it was concluded that low values measured in the borings are not in isolated zones. Therefore, it was concluded that the site soils to depths of approximately 12 m have potential for liquefaction and require densification to reduce their potential of liquefaction. Below depths of 12 m, the low values observed are in localized zones. Liquefaction potential at the site was determined based on the work of Seed et al. (1983, 1984) and Seed and Idriss (1971, 1982). The results were also checked using the recommendations of NCEER workshop (NCEER, 1997).

The current seismic hazard maps developed by U.S. Geological Survey for ground motions that have a 10 percent probability of being exceeded in 50 years, show the peak ground acceleration (PGA) at the site to be approximately 0.08g. Results of the ground response analyses indicated that the bedrock peak ground acceleration of 0.08g could be amplified to peak ground acceleration of 0.11g to 0.18g at the ground surface (i.e., by a factor of 1.4 to 2.25). The amplification computed in the present study is consistent with the 1997 NEHRP recommended provisions (BSSC, 1998) and Hwang and Huo (1997). Peak ground acceleration of 0.16g was used to perform the liquefaction analyses. Ground response analyses were performed by using a computer program, SHAKE91 (Idriss and Sun, 1992), which is an updated version of a well-known computer program, SHAKE (Schnabel et al., 1972). The SHAKE91 program uses the theory of one-dimensional wave propagation through layered media.

Several methods for remediation of the existing soils were reviewed. After considering factors such as technical adequacy, cost, long-term performance, environmental impact, and filed verifiability, heavy tamping was selected to densify the existing soils at the site. Factors such as no structure within 500 feet of the site, depth of groundwater table, and presence of fine-grained soils near the top made the heavy tamping more attractive. However, depths to which the improvement was required, presence of an energy observing layer of medium dense sand at an approximate depth of 10.5 m, and thickness of the top stratum (approximately 3 m) of fine-grained soils were the factors which needed to be addressed to make this approach cost effective. In order to achieve the project goals using heavy tamping, the top 0.6 to 1.5m of fine-grained soils were removed and construction of stone columns were added as discussed in the following sections. Pre-densification survey of the nearby existing buildings was thoroughly conducted before start of heavy tamping.

Table 1: Blow Counts Measured at the Site and Required to Reduce Liquefaction Potential

Depth (m)	Measured N- Values (Uncorrected)								Required N-Values
	B-1	B-2	B-3	B-4	B-5	B-6	B-7	B-8	
1.5									13
3.0			*3*					*5*	13
3.8	*2*	*5*	*11*		*8*	*9*	*3*	*8*	13
4.6	*1*	*5*	*5*	*10*	*5*	*5*	*5*	*5*	13
6.1	*7*	*4*	18	*11*	*12*	16	18	17	14
7.6	34	*13*	20	*11*	*8*	*9*	*8*	*9*	14
9.1	16	*9*	*7*	21	20	*12*	23	*5*	15
10.7	21	23	17	20	14	19	40	15	16
12.2	*7*	28	*9*	18	*11*	*12*	*13*	18	16
13.7	21	30	24	*7*	49	35	21	*12*	17
15.2	31	20	7	10	15	17	12	16	17
16.8	16	21	12	18	19	22	17	12	17
18.3	25	14	22	30	30	22	22	16	
19.8		15			42		19		
21.3		17			23		47		
22.9		31			10		18		
24.4							48		

PROCEDURE FOR HEAVY TAMPING

The conventional method of densifying in-situ soils using heavy tamping consists of two steps: area-wide compaction and ironing pass. The area-wide compaction consists of repeatedly dropping a 90 to 180 kN weight from heights of 12 to 30 m. The energy is generally applied to the soil in phases on a grid pattern over the entire area using either single or multiple passes. Following each pass, the craters are either leveled with a dozer or filled with granular fill material before the next pass of energy is applied (Lukas, 1995). The depth of improvement generally depends on the total amount of energy applied to the soil, which is a function of the weight of the tamper and the drop height. In general, a tamper of 90 kN weight falling through a height of 21 to 24 m, will be able to compact a sand deposit to depths of 7.5 to 9.0 m. The in-place soils below the depth of craters are compacted due to vibrations and dissipation of excessive pore pressures generated during compaction.

The ironing pass consists of dropping a lighter tamper (approximately 45 kN) and smaller drop heights (7 to 9 m) on an overlapping grid pattern. The purpose of the ironing pass is to compact the soils at shallower depths, generally to the depths of craters. The size of the tamper for ironing pass is generally larger than that used for deep compaction during the first phase.

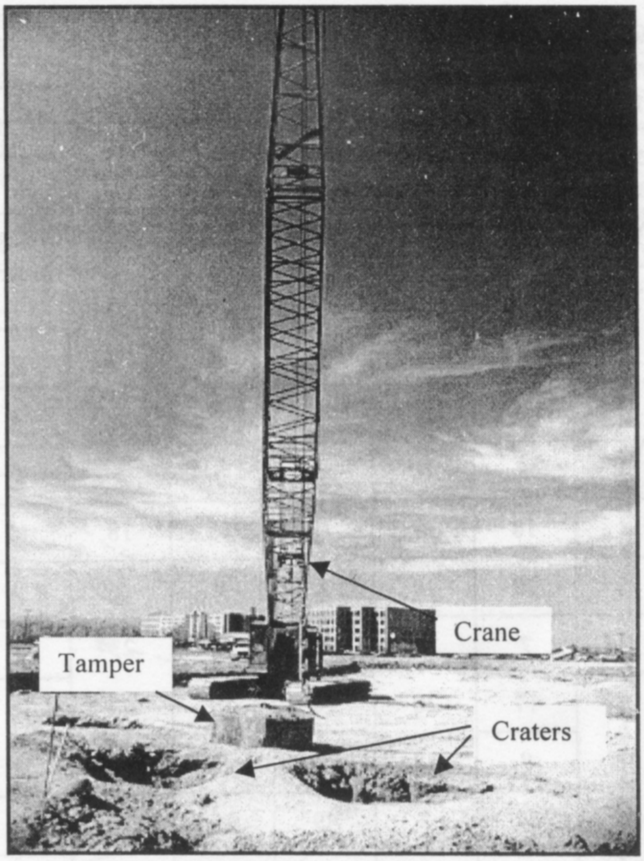

Fig. 4 Heavy tamping at the site in progress

As discussed earlier, the presence of approximately 3 m of fine-grained soils (silty clays and clays) near the ground surface, and a layer of medium dense sand at a depth of 10.5 m made the heavy tamping less attractive at this site. To accomplish densification to the required depths and to reduce the effect of presence of fine-grained surface soils, top 0.6 to 1.5 m of the fine-grained surface soils were removed before the start of the heavy tamping. Therefore, in order to achieve the densification to the required depths of about 12.0 m, the entire operation was divided into three distinct phases:

Phase I: Area-wide deep compaction

Phase II: Construction of stone columns at location of the column footings

Phase III: Area-wide shallow compaction (ironing pass).

The effectiveness of compaction was monitored after Phases I and II. The area-wide deep compaction was accomplished by dropping a 170 kN weight, 3 to 6 times, from a height of approximately 21 m on a grid spacing of 4.5 m. The craters were backfilled using locally available, 8 mm minus limestone screenings mixed with site soils. Figure 4 shows the heavy tamping in progress at the grid points and the resulting craters. Following the area-wide heavy tamping, standard penetration tests were conducted at predetermined locations. Figures 5 and 6 present a typical comparison of the standard penetration test values in different boreholes before and after the heavy tamping. It is observed from Figure 5 that the SPT values within depths of 3 and 8.5 m show a substantial improvement after area-wide compaction. Data in Figure 6 also shows a maximum depth of improvement to be approximately 9.0 m which is consistent with the maximum depth of improvement reported by Lukas (1995) and FHWA (1986). In general, the test results indicated that the area-wide compaction densified the existing soils to maximum depths between 5.5 and 10 m, and was generally less than the required depth of 12 m.

The effective depth of compaction was enhanced by construction of stone columns at the location of column footing of the building. The following procedure was followed for the construction of stone columns.

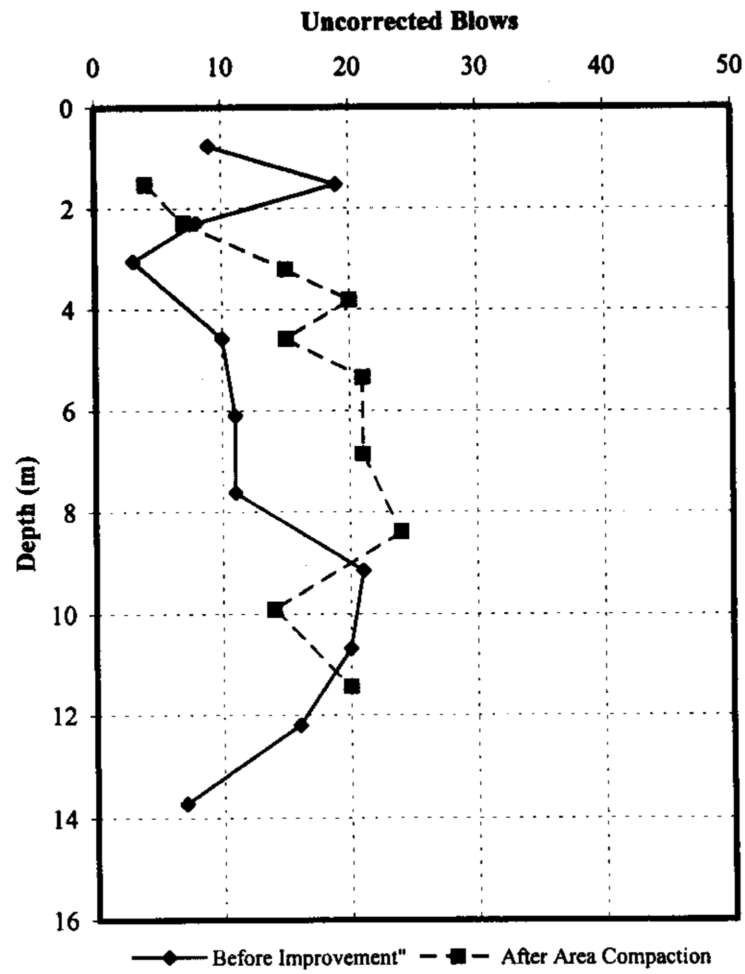

Fig. 5 SPT data before and after compaction

1. Construction of Stone Columns

At each predetermined column location, either one or five stone columns were constructed depending on the size of the footing. For footings smaller than 2.4 x 2.4 m, one stone column was constructed at the center of the footing location. For larger footings, stone columns were constructed at the center and at each corner of the footing locations (a total of 5 stone columns).

Stone columns at the center of the footing locations were constructed using 170 kN weight and drop heights of approximately 21 m. At the center of the footing, the weight was dropped 10 times from a height of approximately 21 m and the resulting crater was filled to a depth approximately equal to half the depth of the crater, with 20 mm minus crushed stone. Depths of craters after 10 drops were generally between 1.8 to 2.4 m. The weight was then dropped again 6 times at the same location and the resulting crater was backfilled to a depth approximately equal to half the depth of the crater, with 20 mm minus crushed stone. The same weight was once again dropped 6 times and crater was backfilled with 20 mm minus crushed stone. The stone in the crater was compacted by dropping the weight once from a height of 9 to 12 m. The number of drops and procedure to construct a stone column was determined based on observations of heavy tamping at a test footing location.

At the center of each footing location, the weight was dropped 22 times, as discussed earlier, to construct a stone column. Based on field observations during ground improvement, and borings performed after ground improvement, the stone columns constructed at the center of the footing locations were approximately 2.4 to 3 m in diameter and approximately 2.4 to 3.6 m deep. Stone columns at the corners of the footing locations were constructed using approximately 15 drops because significant heave was observed with greater number of drops. The compaction enhancement resulting from stone columns was also evaluated by conducting SPT tests at selected locations and comparing with the pre-compaction values. Some typical results comparing the SPT values for three cases after compaction are shown in Figure 7. Figure 7 presents one set of SPT values with depth before compaction, one set of SPT values after area compaction, and two sets of, SPT values after installation of stone columns. The set of N-values referred to as 'After Stone Columns (I)' was obtained in a boring drilled at the center of a footing location through a stone column. The set of N-values, which is referred to as 'After Stone Columns (II)', was obtained from a boring drilled at the center of space formed by four footings. It is observed from this figure that the effective compaction with stone columns was more than that achieved with area compaction alone, and in most cases, was close to the required depths of 12 m.

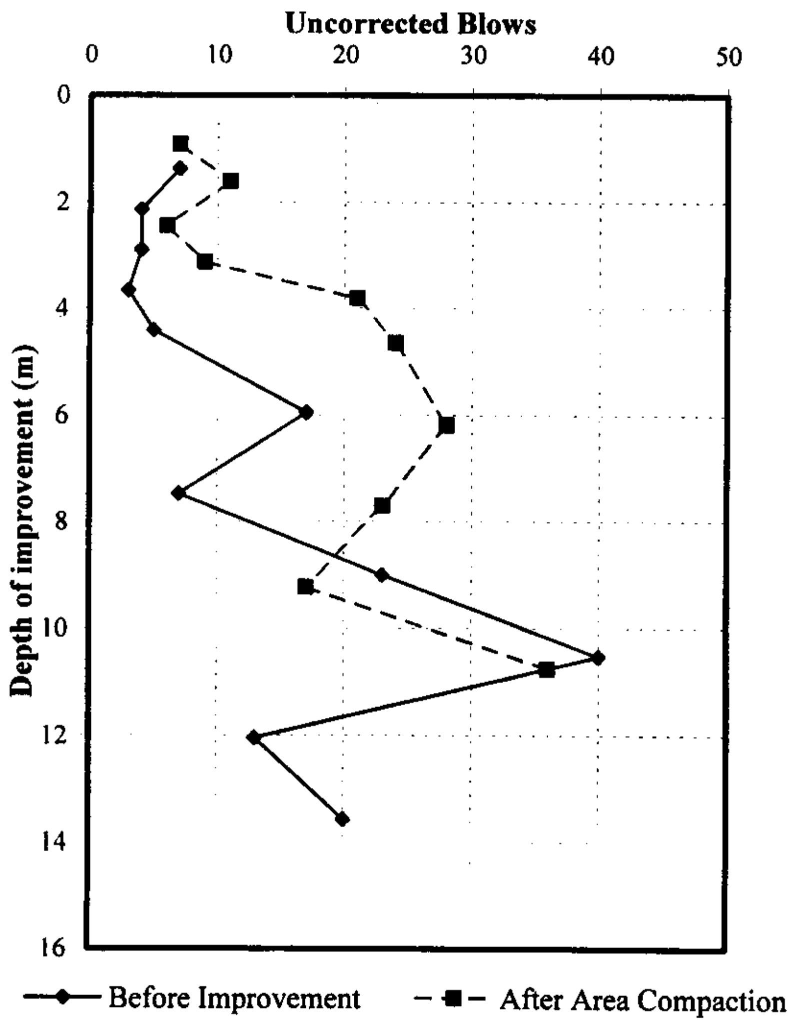

Fig. 6 SPT data before and after compaction

Figure 8 shows the N-values measured (uncorrected) in several borings drilled after the site improvement along with the N-values required to reduce potential for liquefaction (shown by a thick line). As can be seen from Figure 8, most of the N-values measured are greater than those required to reduce liquefaction potential. Few N-values were lower than the targeted values but these occurred in isolated zones. Subsequent liquefaction analysis incorporating the soil properties in terms of N-values obtained in the borings drilled during the post-compaction phase indicated that the liquefaction potential was significantly reduced.

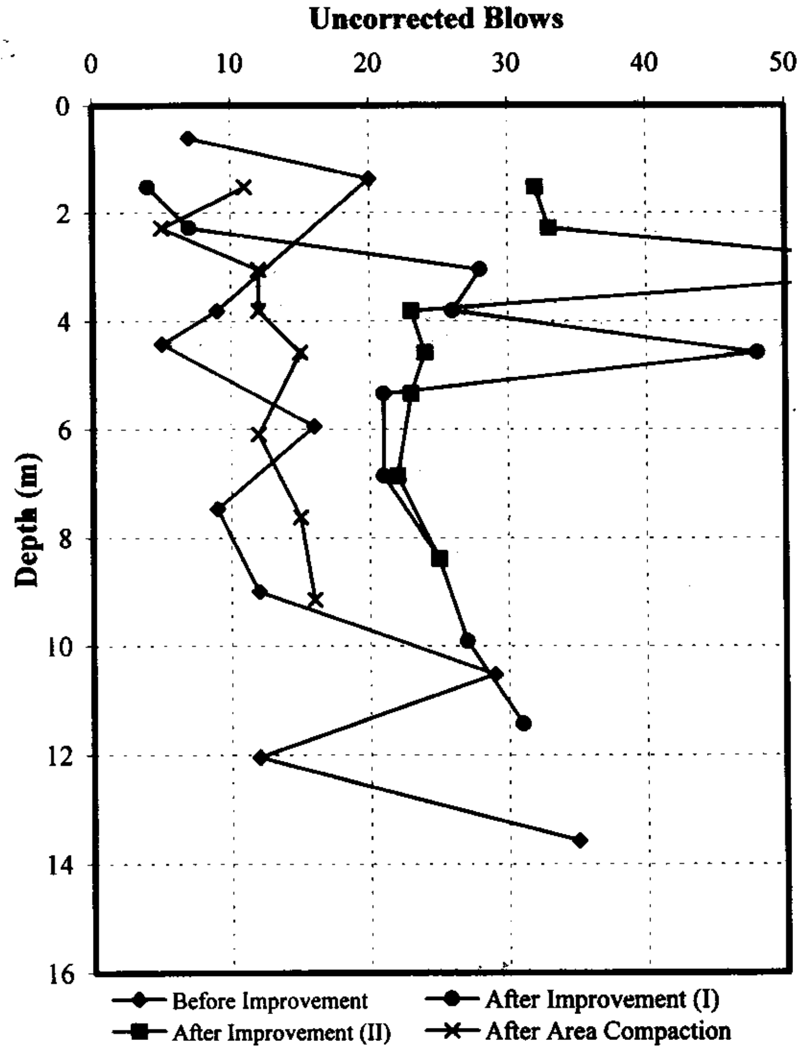

Fig. 7 SPT data before and after compaction

CONCLUSIONS

Based on the results of this investigation, it may be concluded that for this particular site,

(1) The heavy tamping was effectively used to compact the granular soils up to depth of 6 to 9 m.

(2) Stone columns in conjunction with conventional heavy tamping were effective in densifying the soils to the greater depths.

(3) The compaction achieved was adequate to reduce the liquefaction susceptibility of the soils at the site.

(4) The building columns, which needed pile foundations prior to soil improvement, could now be supported on shallow foundation.

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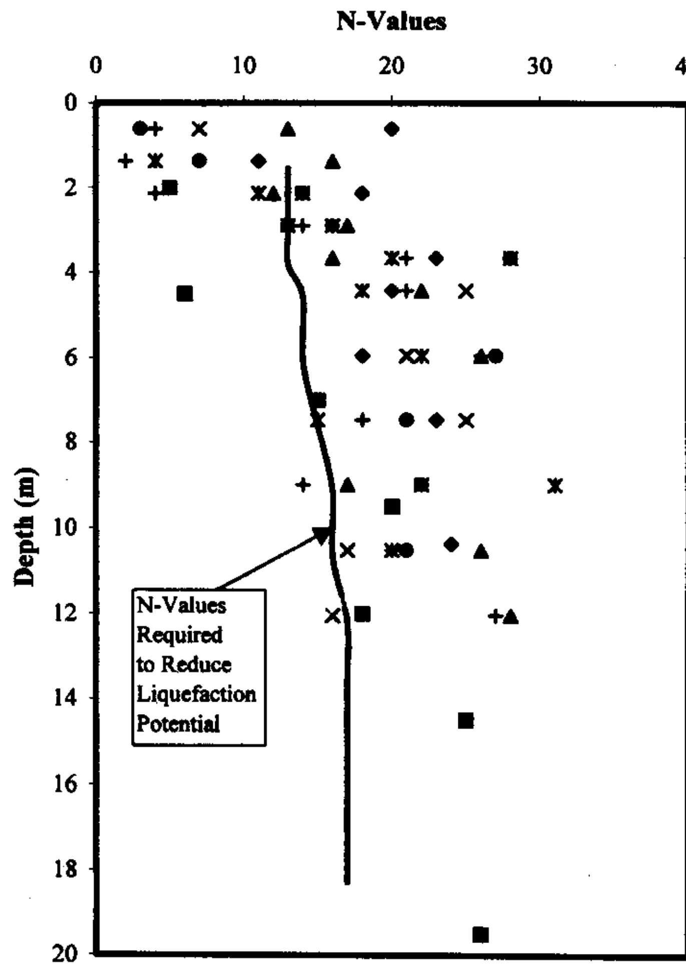

Fig. 8 SPT data after site improvement

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