Ground Improvement and Testing of Random Fills and Alluvial Soils

UPUL D. ATUKORALA, DHARMA WIJEWICKREME, AND RICHARD C. BUTLER

Case histories of ground improvement are presented for three sites located along the coastal margin of southwest British Columbia, a zone of moderate to high seismic risk. Materials consisting of random fills and unconsolidated alluvial soils have been improved to reduce settlements, increase bearing capacity, and provide adequate protection against liquefaction for seismic loading. Two of the sites were treated using dynamic compaction, whereas ground improvement at one site was carried out using vibroreplacement techniques. The degree of ground improvement achieved has been assessed using a variety of in situ penetration resistance methods including SPT, electronic CPT methods with dynamic pore pressure measurements, Becker penetration tests, and pressuremeter testing. At one of the sites, postconstruction settlement measurements were carried out over a period of 4 months to assess the effectiveness of the treatment program and in situ test data.

For satisfactory performance of foundations, the supporting soils must be capable of carrying the applied loads with tolerable deformations. With the ever-increasing demand for land, sites that do not have naturally occurring competent bearing strata at shallow depth are being treated by some form of ground improvement to permit suitable development with conventional foundation support. In areas of high seismicity, such as the west coast of Canada, protection against liquefaction of foundation soils is an added requirement in foundation design. Limited by the mountainous terrain of the region, most otherwise desirable sites are located near the shoreline and frequently consist of loose, unconsolidated granular soils of variable composition with groundwater levels close to the surface. Many of these areas have been filled in the past by random, often coarse soils or construction debris placed or end dumped with little or no control or compaction. Even those granular soils that perform satisfactorily for static loads generally perform poorly under earthquake shaking, with resulting risk of liquefaction-induced loss of support, excessive deformations, and failure. Ground improvement methods have been used at many such sites to enhance the performance of these soils and fills under both static and seismic loading conditions. The three sites presented as case histories have been identified as A, B, and C. At Sites A and B, the uppermost layer and bearing stratum consists of coarse granular soils and random fills. Ground improvement by densification has been carried out using dynamic compaction. The overburden soils at Site C consist of sands, silts, and silty sands. Ground improvement has been achieved using vibroreplacement methods.

Golder Associates, Ltd., Vancouver, British Columbia, Canada.

Following completion of ground improvement treatment, the strucures at these sites were constructed using conventional shallow foundations, or at Site C, shallow foundations plus pile support under several concentrated column loads.

GROUND IMPROVEMENT METHODS

The suitability of a particular ground improvement method depends on a number of factors including but not limited to the type of soil, depth of treatment required, site location, environmental concerns, and site access restrictions. Of the various ground improvement methods available, dynamic compaction and vibroreplacement have been successfully used at the three sites reported in this paper. A brief description of each technique is presented herein for completeness.

Dynamic Compaction

Dynamic compaction is a method of ground improvement in which poor foundation soils are densified by the systematic application of very high energy impact loadings using steel or concrete tamper weights typically weighing 10 to 20 tonnes, dropped from heights of up to 40 m but most commonly limited to 10 to 25 m by practical and economic considerations with respect to the lifting equipment. The high impact loading induces large shock waves that penetrate the underlying soils to considerable depths, inducing restructuring and densification of granular soil. Fills consisting of mixtures of gravel, sand, silt, and clay; hydraulically placed sand fills; and most natural granular soils with a clay fraction less than 15 percent respond favorably to dynamic compaction. The presence of coarse cobble- or boulder-size particles or construction debris up to 1 m in diameter is not normally a significant restriction on the suitability of this technique. Natural soils with more than 50 percent passing the 0.075-mm (USS200) sieve and clay fraction in excess of 15 percent show little or no improvement with dynamic compaction (*1*).

The compaction effort imparted by the impact loading is usually computed in terms of energy per unit area (i.e., T-m/m^2 and for most applications ranges from about 100 T-m/m^2 to in excess of 400 T-m/m^2. The maximum depth of influence, d_{max}, has been found to be proportional to the square root of the energy applied and is given by

$$d_{max} = C \cdot (WH)^{1/2} \tag{1}$$

where

W = weight (tonnes),
H = free fall (m), and
C = coefficient dependent on a number of factors, including the soil type and stratigraphic features, efficiency or energy loss of lifting/tamping equipment, the ratio of mass to surface area of the weight, and the method of application of energy.

Typically, C is close to 0.5, although variations on the order of $+0.25$ have been reported.

Noise generated by the tamper impacts is generally muffled and not environmentally objectionable. In cases where direct impact occurs onto coarse materials, both the potential for increased noise and, more important, possible projection of surface particles due to air pressure outflow around the edge of the tamper may require provision for safety screening of fencing around the treatment area. The most frequently encountered limitation in the use of dynamic compaction methods is the potential risk of ground vibrations being induced beyond the treatment area that are sufficient to be unacceptably irritating to persons and, less commonly, potentially damaging to nearby structures.

Vibroprocesses

The improvement of foundation soils by the use of deep vibratory techniques forms one of the most frequently used methods of ground improvement. This technique involves the use of a large, downhole poker type vibrator, operating at frequencies ranging from 20 to 50 Hz with up to 125 kW (165 hp) of energy. The vibrations are induced by rotating eccentric weights mounted on a shaft driven by an electric motor located within the casing. The vibrator probe is advanced to the maximum depth of treatment required, typically using water jetting, and occasionally air, to aid in penetration. Densification is achieved as the vibrator probe is extracted at a controlled rate to permit compaction of the soil, often aided by water injection. The current drawn by the motor is generally used as a field guide of the densification achieved.

In granular soils having less than 20 percent fines passing the 0.075-mm (USS200) sieve, the vibroprocess is generally referred to as vibrocompaction. The natural soils or fill being densified may be used as backfill or ground surface may be allowed to subside as the volume of the underlying soil decreases because of densification.

In finer-grained silt or clayey soils, the amount of densification that can be achieved is generally limited. In these cases, the vibroprocess is referred to as vibroreplacement, where essentially the same equipment and procedures are used except that select gravel or coarse sand is used as backfill to fill the void created during installation of the probe and any densification of the surrounding soil. This select backfill forms "stone columns" within the overall treated soil mass that increase the bearing capacity and provide rapid dissipation of pore water pressure through shorter drainage paths.

The spacing and pattern of the probe installations are selected to achieve the desired densification of granular soils. Similarly, the spacing of stone column vibroreplacement installations is selected to provide the added strength or stiffness.

VERIFICATION TESTING

In situ verification testing is normally carried out on completion of ground improvement treatment or at specific stages during such treatment. Typically, verification testing uses the same geotechnical drilling, sampling, or probing techniques as that carried out during the initial geotechnical investigation or preconstruction assessment of the site to permit a direct comparison of the change in conditions resulting from treatment. In some instances, additional in situ testing, pressuremeter testing for example, is carried out either after or both before and after ground improvement treatment when specific bearing capacity and settlement criteria must be achieved.

However, differing investigation and testing techniques must often be used to achieve the desired verification while accommodating the soil or fill conditions and characteristics at specific sites. At Site A, because of the very coarse granular soils at this site, Becker penetration testing (BPT) was considered the most suitable means of assessing the penetration resistance of soils. The Becker percussion drill rig consists of a double acting ICE-180 type diesel hammer that is capable of delivering 11 kJ (15,000 ft-lb) per blow. The penetration resistance is measured in terms of the number of blows required to advance a closed-end 140-mm nominal diameter double-walled steel casing over an incremental distance of 300 mm. Although correlations have been proposed (*2*) between BPT values and the conventional SPT values to compute settlements beneath the tank foundations, settlement measurements have also been taken during initial operation of the SC tanks as further verification of improvement.

At Site B, the densification effort has been evaluated by comparing both standard penetration test data and pressuremeter moduli and limit pressures measured with depth. A Menard type pressuremeter was used, which was inflated inside a special 125-mm-diameter protective steel split casing.

At Site C, treated using the vibroreplacement technique, the improvement effort has been evaluated by comparing the pre- and posttreatment electric cone penetration (CPT) resistance measurements with depth.

SITES, SOIL CONDITIONS, AND TREATMENT PROGRAM

The following summarizes site and subsurface characteristics, the foundation design requirements, selection, and installation of the ground improvement treatments at the three sites.

Site A

Site A is located on a small deltaic and shoreline deposit in southwest British Columbia where two new secondary clarifier (SC) tanks, each approximately 70 m in diameter, were constructed as part of the modernization ad expansion program of an existing pulp mill. The two SC tanks were located side by side, separated by about 10 m, and consisted of 0.3-

m-thick grade-supported floor slabs that were conical in shape. The outer walls were 5.5 m high and supported on perimeter ring foundations. The height of fluid retained inside these tanks varies between about 7.3 m at the center to about 4.5 m at the outside walls. The structural design of the foundations imposed stringent differential settlement criteria of 25 mm across the diameter of tanks with not more than 6 mm over a 6-m distance locally.

Soil Conditions

The southern SC tank and a portion of the northern tank were located within an old bay area east of the existing pulp mill as shown in Figure 1. The site grade was raised 2 to 6 m by placing granular fill from excavations carried out in other parts of the mill site. Part of this fill supporting the SC tanks was placed under water by end dumping. In situ penetration resistance measurements carried out subsequent to end dumping indicated that the fills were in a loose state of compaction and likely to induce excessive ground settlements.

To the north, the site consists of a cobble and boulder layer of 6 to 8 m in thickness overlying an extensive deposit of compact, medium to coarse, angular alluvial sand. On the basis of BPT results obtained within the cobble and boulder layer, the density of this layer has been inferred to be compact with possible loose zones, in which the cobbles and boulders are embedded within a loose sand matrix. These loose zones were also expected to produce settlements in excess of the acceptable values when subject to the design foundation loads.

Treatment Program

Ground improvement was considered necessary to achieve the required settlement characteristics as well as to provide adequate resistance to liquefaction of the foundation soils in the event of a major earthquake. Ground improvement techniques that involve penetration of a probe as in vibrocompaction, compaction grouting, and jet grouting were considered inapplicable for this site because of the difficulties anticipated in penetrating the cobble and bouldery stratum.

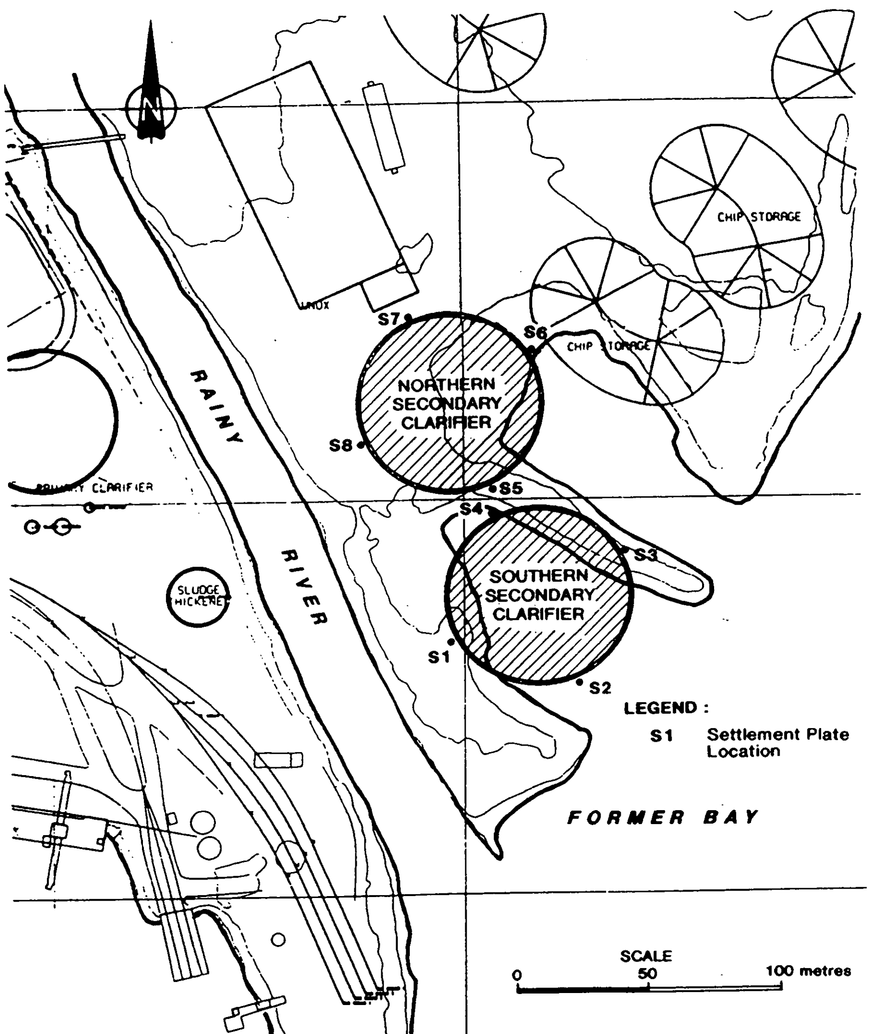

FIGURE 1 Location of SC tanks before site upgrading.

Dynamic compaction was selected as the most suitable method of ground densification in view of the cobble and bouldery stratum, reasonable thickness of material requiring treatment, and since the site was located in an area having no existing structures housing vibration-sensitive equipment.

Dynamic compaction was carried out over the plan area and 8 m beyond the perimeter of the SC tank foundations. Primary densification points were located at a center-to-center spacing of 6 m arranged in a square grid pattern. Approximately 220 T-m/m^2 of energy was imparted at these compaction points by dropping a 20-tonne weight over a height of about 26 m. Approximately seven to eight drops were recorded at each location, resulting in craters that were 0.5 to 1.5 m deep. The secondary densification points were located at the centers of the primary points.

Site B

Site B is located north of the Burrard Inlet in Vancouver, British Columbia, where an extensive office building up to six stories in height has been constructed. The structural loadings at major foundations impose bearing pressures up to 300 kPa with the exception of some peripheral footings where a design bearing pressure of 200 kPa was designated. The structural design required that the total settlements of the footings be limited to 20 mm and the differential settlement between any two adjacent columns be limited to 10 mm (1 in 1,300).

Soil Conditions

A major portion of the site is located within land reclaimed by uncontrolled filling of the tidal area of the Burrard Inlet over a period of many years. The site was generally flat. Along the southern boundary of the site, the fills were retained by a sheetpile bulkhead anchored to concrete deadmen.

Site investigation revealed that most of the site was underlain by heterogeneous fill material, which varied in composition from silty sand and gravel to sandy silt, with extensive local zones of coarse construction fragments, and large cobble or boulder sizes near surface. In general, the fill thickness increased from some 2 m at the northern boundary to as much as 10 m near the southern boundary. Generally, the upper 1 m of the fill was compact, and the remainder varied from very loose to dense with SPT values ranging from 1 to 8 blows/0.3 m. Bay bottom deposits consisting of very loose to loose silt and sand up to 2 m thick with SPT values varying from 1 to about 12 blows/0.3 were encountered below these fills in certain areas within the south portion of the site. The generally loose fills and bay bottom sediments were in turn underlain by compact to dense sand and gravel beach sediments containing shell fragments, organics, occasional cobbles, and a trace of silt, typically less than 5 m thick with SPT values in excess of about 25 blows/0.3 m. The beach sediments in the south and the fill in the north rest on a thick and extensive deposit of randomly stratified dense to very dense glacial drift, with SPT values in excess of 100 blows/0.3 m. The groundwater level was located at a depth of some 1 to 2 m below ground surface.

On the basis of the SPT values and gradation data, it was concluded that much of the fill, the bay bottom deposits, and loose zones of the beach deposits would have a high probability of liquefaction if subject to a design earthquake firm ground acceleration of 0.16 g. Liquefaction of soils at this site was expected to induce loss of bearing capacity, excessive total settlements of the foundations, and risk of large horizontal ground movements or failure toward the nearby harbor shoreline.

Treatment Program

To improve the foundation performance for static loading, permitting use of shallow foundations, and to provide an adequate factor of safety against liquefaction, ground improvement using dynamic compaction was selected, taking into consideration the presence of coarse materials detrimental to other ground improvement methods even though this work was to be carried out adjacent to a major transit exchange, which had to be maintained in operation throughout the treatment program. Dynamic compaction was carried out using tampers weighing 18 tonnes dropped from heights of up to 28 m. An 11-tonne tamper was used for low-energy ironing applications.

The average settlement of the site induced by dynamic compaction was on the order of 0.55 m, equivalent to an average vertical strain of about 8 percent within the compressible strata. Because of the presence of existing structures as well as business uses and transit operations, ground vibration measurements were determined throughout the course of the treatment. All ground vibrations at off-site structures or other vibration-sensitive facilities were significantly below a value of 50 mm/sec, normally considered to be the upper safe limit of vibration velocity to limit risk of structural damage.

Site C

Site C is located within the floodplain, close to the confluence of two mountain rivers in southwest British Columbia north of Vancouver, where an aquatic center covering a plan area of 45 m by 55 m was to be constructed immediately adjacent to an existing recreation center and arena. The aquatic center was to house a swimming pool about 500 m^2 with a maximum depth of 3.5 m. The average column loads imposed by the structure are generally less than 450 kN with the exception of two columns located close to the swimming pool imposing loads up to 2200 kN per column. The lighter columns are to be supported on shallow spread footings, whereas the heavier columns are to be supported at a depth of about 4 m below pool deck level.

Soil Conditions

On the basis of a field investigation carried out during the initial design phase, which consisted of a series of shallow test pits, sampled boreholes, and dynamic cone penetration tests, the soil conditions at the site were inferred to consist of loose to compact sand and gravel fill about 1 m thick, overlying 5.5

to 6 m of very loose to loose sand and silty sand with interbedded thin layers of silt and significant wood or organic fragments both within the granular soils and as thin layers or lenses. Compact to dense layered sands, gravels, and cobbles underlie these upper soils to depths in excess of 10 m below ground surface. The overall site is generally flat with groundwater table within 1.5 m of original ground surface.

The site is located within Seismic Zone 3 of the current (1985) Canadian and British Columbia building codes with a design ground acceleration level of 0.14 *g*. Liquefaction potential assessment indicated that the very loose to loose sand and silty sand interbedded with sandy silt having a high probability of liquefaction during the design earthquake. The risks of liquefaction-induced loss of bearing capacity, excessive total settlements of the foundations, and large horizontal ground movements were considered unacceptable for this public building.

Treatment Program

Several forms of ground improvement were considered during design (i.e., dynamic compaction, compaction piles, and vibroreplacement). Dynamic compaction was eliminated because of the high probability of excessive and damaging ground vibrations at the existing and adjoining arena structure. Cost-benefit considerations indicated that ground improvement at this site could be best achieved by the vibroreplacement "stone column" method. Vibroreplacement treatment to depths of up to 10 m below ground surface was selected, covering the entire building plan and up to 8 m outside the perimeter of the structure. The timber piles supporting the heavily loaded central columns were driven to the dense granular soils strata about 10 m below ground surface following vibroreplacement treatment.

Vibroreplacement columns were put down on a 2.5-m equilateral triangular grid pattern using a vibratory probe with a rated energy of 125 kW (65 hp). Stone columns were installed to a depth of 8 m. The average amperage buildup was between 180 and 220 A, and the stone consumption was on the order of 1.5 to 2 tonnes per linear meter of "stone column."

On the basis of vibration monitoring carried out during treatment, the maximum measured velocities within the adjacent arena building were less than 7 mm/sec when operations were carried out up to 6 m from the existing structure.

VERIFICATION TESTING

Site A

Figure 2 shows the pre- and postdensification BPT results at selected locations within the SC foundations. The comparisons indicate that measurable and often significant ground

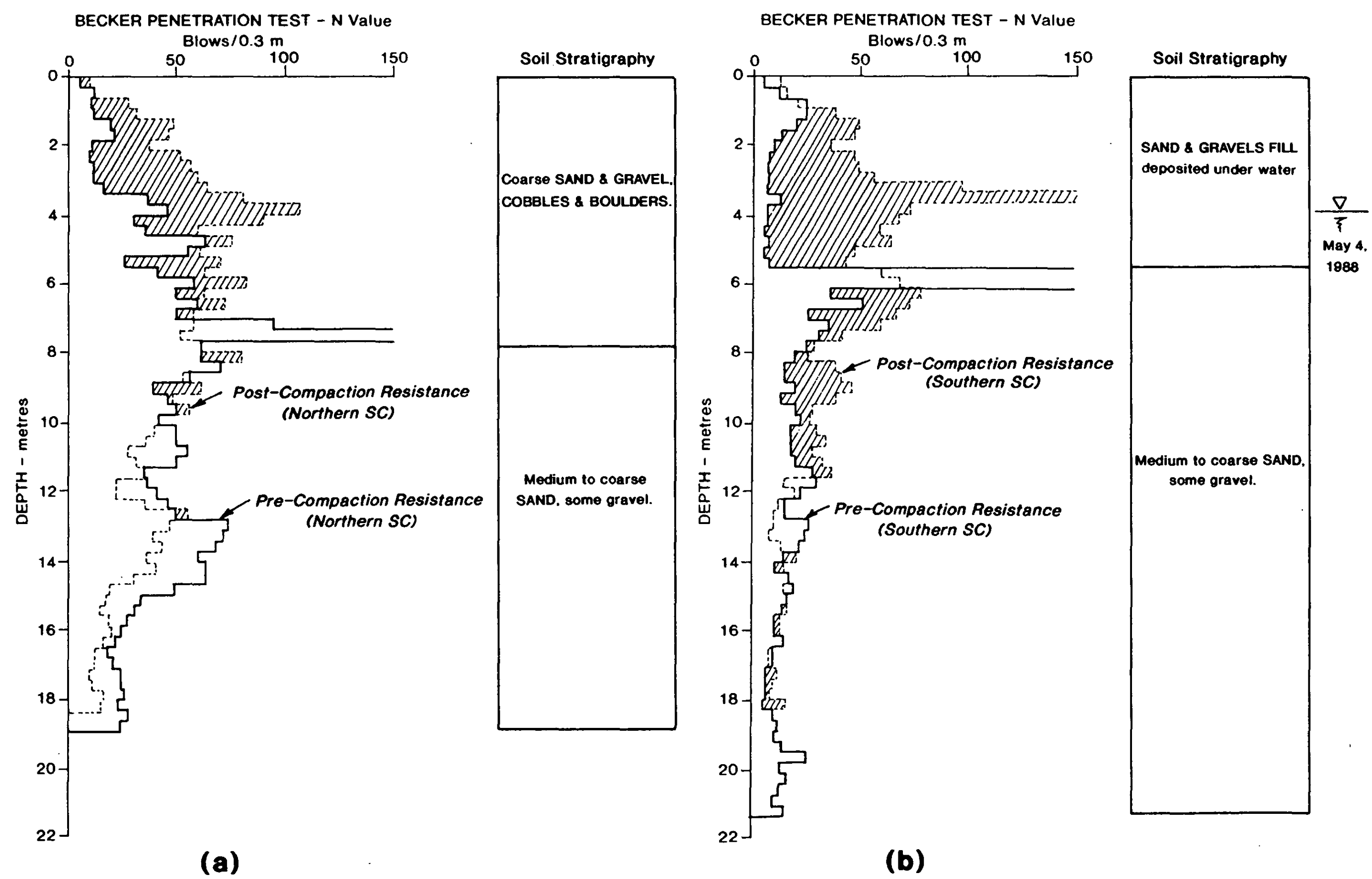

FIGURE 2 Comparison of pre- and postcompaction BPT *N*-value profiles at the SC tank sites.

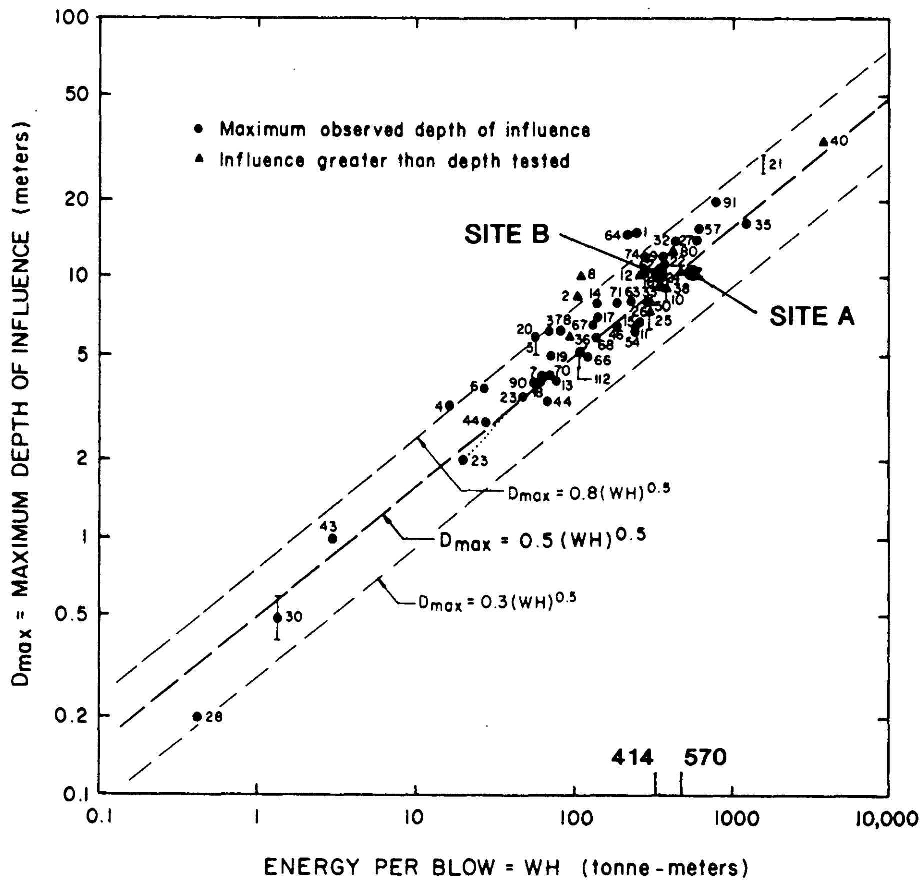

FIGURE 3 Trend between apparent maximum depth of influence and energy per blow (*3*). (Numerals refer to sources of data.)

improvement has been achieved to depths of 9 to 12 m below ground surface. Figure 3 shows the imparted energy per tamper drop versus the depth of improvement anticipated on the basis of published correlations (*3*). The depth of influence inferred based on BPT data is in good agreement with these results.

As part of the postconstruction monitoring program, a series of eight settlement points was installed at the outer ring walls during foundation construction. Four settlement points were installed at diametrically opposite locations in each tank. The locations of the settlement plates S1 through S8 are shown in Figure 1.

The maximum settlements recorded over a period of 4 months of operation of the SC tanks are summarized in Table 1.

TABLE 1 Summary of Settlements

Location	Settlement after 120 days (mm)
South SC Tank	
S1	3.6
S2	8.1
S3	7.8
S4	5.7
North SC Tank	
S5	6.0
S6	4.5
S7	2.4
S8	2.4

Settlements of about 8 mm were measured at two settlement points, S2 and S3 on the southern SC tank, which is located within the former bay area, underlain by the greatest thickness of end-dumped fill. At the northern tank, the maximum measured settlements were 6 mm or less. Overall, the measured settlements were within the limits required for structural considerations and provide additional confirmation of the effectiveness of the dynamic compaction ground improvement in reducing postconstruction settlements to acceptable levels.

Site B

The degree of densification achieved by dynamic compaction was verified by carrying out a series of standard penetration tests as well as pressuremeter (Menard) tests. Figure 4 compares SPT values obtained with depth before and after dynamic compaction. The penetration resistance of soil within the upper 10 m of the site has achieved the design values, with the upper portion of the fills having been densified well in excess of that required for resistance to liquefaction.

Comparison of the pre- and postcompaction pressuremeter moduli and limit pressures shown in Figure 5 indicates a trend similar to that of the SPT values and typical of that observed for dynamic compaction ground improvement. As shown, the posttreatment moduli or SPT values increase with depth to a

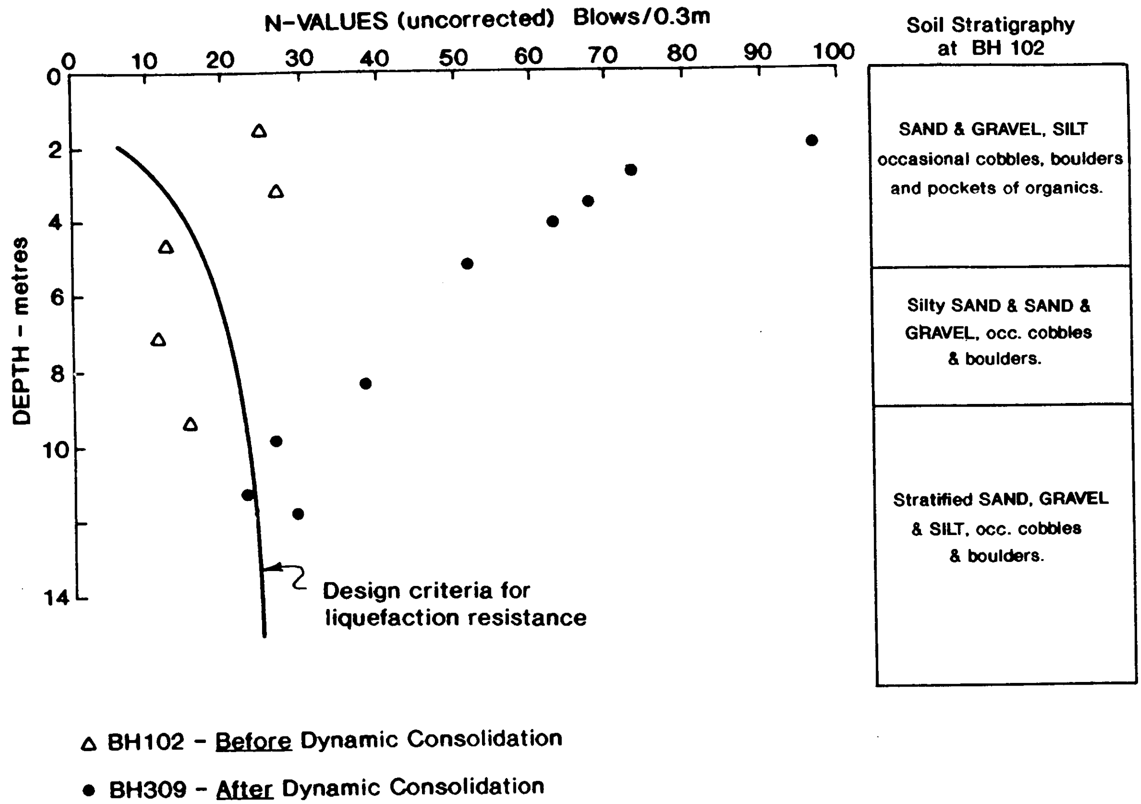

FIGURE 4 Comparison of pre- and postcompaction SPT values (Site B).

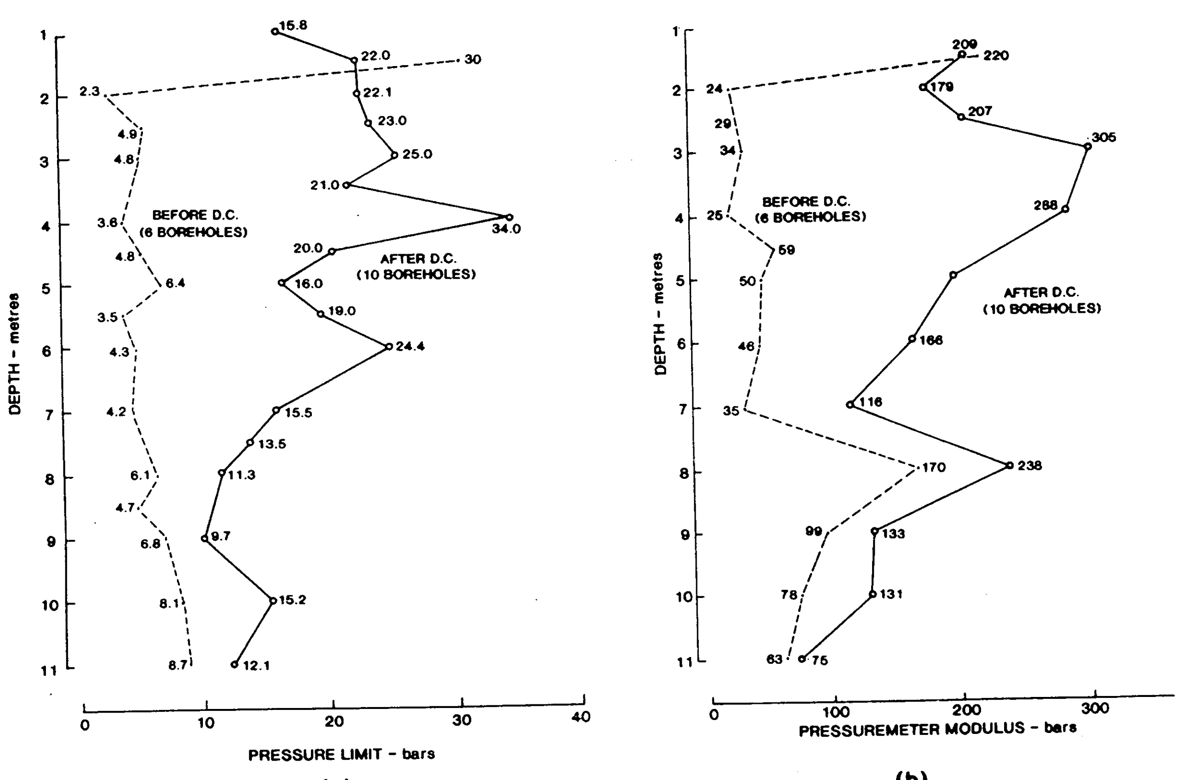

FIGURE 5 Comparison of pre- and postcompaction pressuremeter moduli and limit pressures (Site B).

maximum value at 2 to 4 m and then decrease gradually with depth to nominal improvement at the depth of maximum influence, 9 to 11 m. This depth of maximum improvement was achieved using an average tamper impact energy of about 415 tonne-m per impact. Backcalculation indicates that the coefficient C varies from 0.44 to 0.54, which is consistent with the relationship shown in Figure 3.

Analysis of the pressuremeter data confirmed that the required allowable bearing capacity of 300 kPa had been achieved, whereas the predicted settlements using this information indicated that postcompaction settlements would generally meet the specified limit. The assessment of liquefaction resistance of the site after dynamic compaction also indicated that an acceptable factor of safety against liquefaction had been achieved for the design earthquake.

Site C

Typical penetration resistance variations observed for standard penetration tests and electrical cone penetration tests carried out before vibroreplacement are shown in Figure 6. A cone tip bearing ratio, qc/N, ranging from 3 to 4 is considered representative for the soil types at the site. The CPT is a continuous logging test and, being able to reflect the nature of a soil in terms of friction ratio, indicated the presence of thicker layers of silts compared with those indicated by the SPT sampling, which was carried out at approximately 1.5-m intervals of depth. The CPT tip bearing profile required to provide the desired level of protection against liquefaction is also shown in Figure 6. This criterion applies to that portion of the predominantly sandy subsoils with less than 20 percent

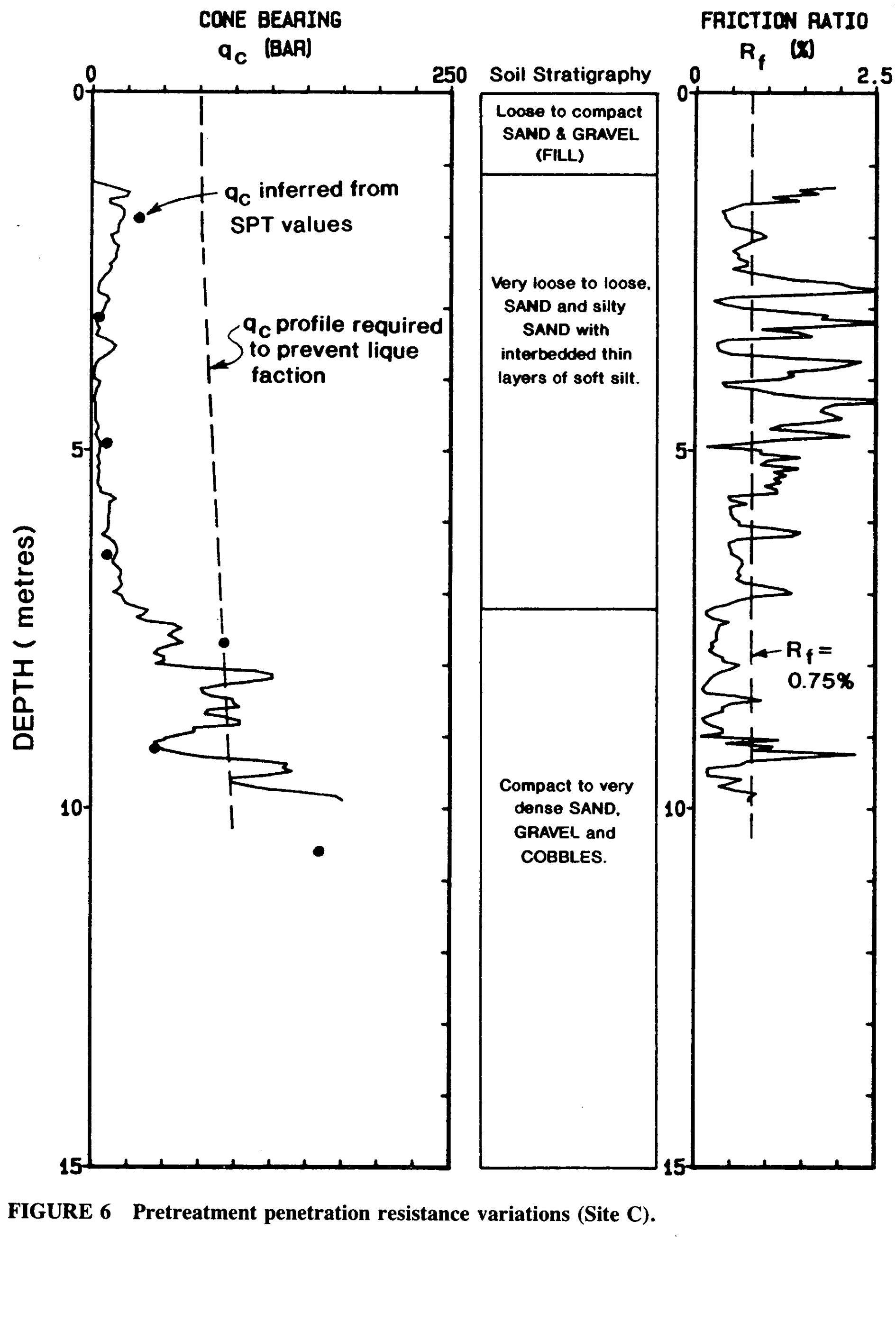

FIGURE 6 Pretreatment penetration resistance variations (Site C).

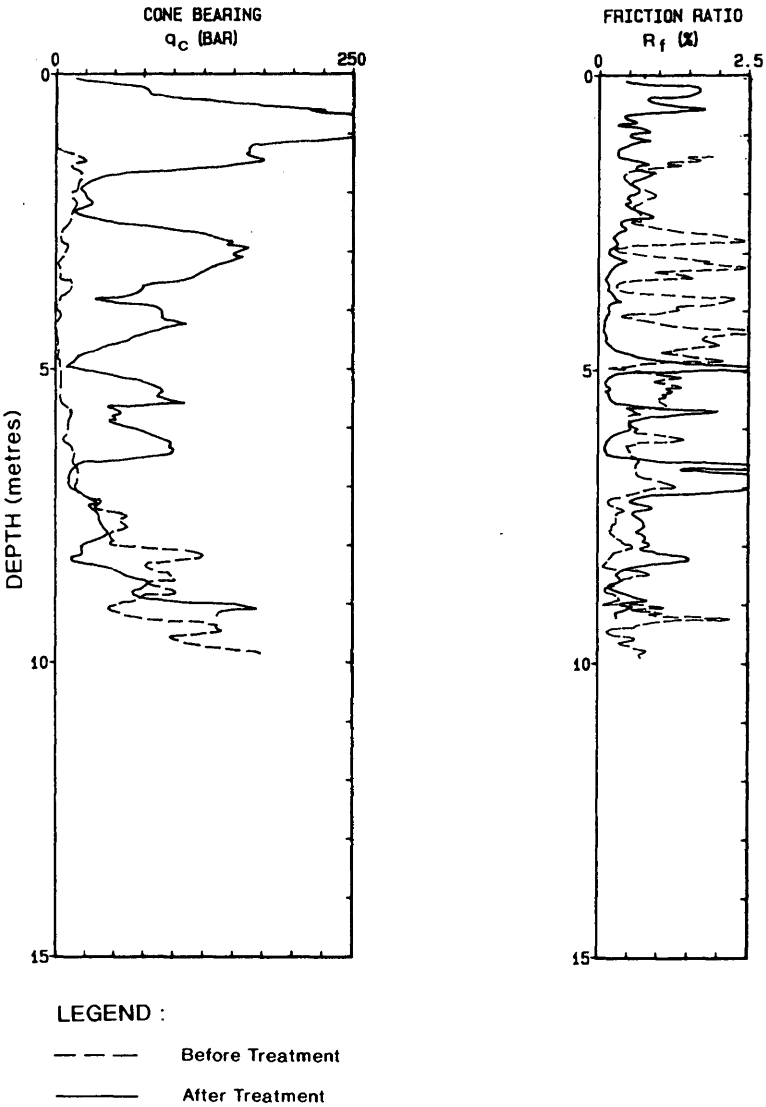

FIGURE 7 Comparison of pre- and posttreatment penetration resistance (Site C).

passing the 0.075-mm sieve, defined as those strata having a friction ratio less than 0.75 percent.

Figure 7 shows the typical CPT profiles after vibroreplacement compared with typical pretreatment CPT data. The post-improvement tests were carried out at the approximate centroid of the triangular grid. As indicated, the predominantly granular soils (friction ratio of 0.75 or less) show large increases in cone bearing, meeting the requirements for resistance to liquefaction and improved bearing capacity. Some but generally limited improvement in cone bearing was recorded within those layers having high friction ratios that are considered to have high silt content. As shown, improvement is limited to the 7- to 8-m depth of vibroreplacement treatment. Although the high silt content layers are not considered to represent a significant risk of liquefaction, the limited densification of the silty strata confirms the suitability of the stone column vibroreplacement method to provide additional stiffness and load-carrying capacity through these fine-grained layers.

SUMMARY

Dynamic compaction and vibroprocesses form viable means of enhancing the foundation performance of granular soils consisting of random fills and alluvial soils as illustrated by the three case histories described. In situ verification testing has been carried out using a number of different testing methods. These methods were selected for each specific site taking into consideration both the constraints due to the soil conditions, such as the presence of coarse materials, and the design and performance requirements to be established.

Given the heterogeneous and often variable subsoil conditions both with depth and areal extent, use of continuous profiling in situ test methods is recommended in all cases to permit correlation between individual tests such as SPT or pressuremeter testing. Equally important, similar test methods should be used before and after ground improvement treatment to permit rapid and direct comparison of results.

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