13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 496

GROUND IMPROVEMENT USING RAPID IMPACT COMPACTION

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SUMMARY

Geotechnical evaluation and assessment of data from Becker Penetration Testing (BPT), Cone Penetration Testing (CPT), Dynamic Cone Penetration Testing (DCPT) and solid stem auger drilling indicated liquefaction susceptible soils at the location of a proposed fire hall and office building complex. Ground improvement works were required to satisfy building performance criteria in the event of a design seismic event. Several ground improvement methods were evaluated and the Rapid Impact Compaction (RIC) method was selected based upon the specific site requirements. The influence depth of RIC is typically around 5 to 6 m (16 to 20 ft) although this is depending on several issues such as soil type, degree of saturation, soil stiffness and other factors.

A RIC pilot program was carried out to assess the specifications for the RIC method to achieve the required ground improvement. Based upon the presence of significant amounts of more granular material (e.g. gravel sized sediments), the BPT was selected as the in-situ evaluative tool. The quality control BPT program carried out after completion of the RIC pilot program indicated that the RIC method could meet the requirement for ground improvement with influence depth extending to almost 9 m (30 ft). Based on the results of the RIC pilot program, the RIC method was used within the entire building footprint. The successful results of the RIC method within the proposed building footprint were confirmed by an additional quality control BPT program. In summary, this paper presents a case history where the RIC method, a ground improvement method recently introduced to the North-American market, was selected for the required ground improvement works and provided desired results for meeting liquefaction prevention design criteria. The RIC method may be a viable alternative to more traditional, and typically more costly, ground improvement methods on projects similar to the one described in this paper.

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INTRODUCTION

A geotechnical site investigation was completed for a proposed combined fire hall and office building complex located in Chilliwack, which is approximately 100 km (60 miles) east of Vancouver in the province of British Columbia, Canada, as shown on Figure 1. The proposed design/build complex will have a relative rectangular footprint of approximately 40 m by 80 m (130 ft by 260 ft). The fire hall section will be approximately two storeys high with five apparatus bays and an about three-storey hose tower, while the office building section will be four storeys high. The proposed complex has been designated as a post-disaster structure required to withstand a 1 in 475 year earthquake with at worst only limited structural damage. The structural damage could be minor repairs, which would not necessarily be required to be carried out prior to re-occupying and operating the complex.

Figure 1 Site location.

Work previously completed by others in the near vicinity of the subject site indicated liquefaction susceptible soils, which resulted in completion of a ground improvement program to densify these soils. The ground improvement method selected at this other site comprised vibroflotation combined with stone columns. Based on this ground improvement work at the adjacent site, the design/build tender for the fire hall project included similar ground improvement to about 8 m (26 ft) depth.

SITE INVESTIGATION PROGRAM

The site investigation program was carried out in several stages using different equipment as deemed suitable as the program progressed. Initially, a track mounted drill rig was used to advance solid stem augers on 3 October 2002. A total of five drill holes (DHs 1 to 5) were advanced to depths between 4.6 m (15 ft) and 9.1 m (30 ft) to provide a preliminary indication of ground conditions. The drilling program was augmented by Dynamic Cone Penetration Testing (DCPT) adjacent to each drill hole to provide preliminary indication of the density of granular soils. DCPTs are used in the local geotechnical practice for initial site screening purposes. The DCPTs were advanced to depths between 11.7 m (38 ft) and 14.0 m (46 ft).

A preliminary assessment of the results from the drilling program and the DCPTs indicated that, like that concluded at the adjacent site, there was indeed soils demonstrating probable liquefaction susceptibility. As a result of this concern, a second investigation program consisting of electronic Cone Penetration

Testing (CPT) was carried out on 15 October 2002. The CPT is universally considered the most accurate geotechnical and repeatable in-situ testing tool for assessing liquefaction susceptibility. Lunne [1] details the advantages of the CPT over other methods. A total of three CPTs (CPT 1 to 3) were advanced to depths between 9.7 m (32 ft) and 10.7 m (35 ft). Several zones of very dense material were encountered, which required drill out with solid stem augers followed by advancement of the CPT below the drilled out zones.

The combination of seemingly spurious (e.g. consistently low) DCPT results and no CPT date within drilled out zones and below CPT refusal depths resulted in necessitating the tertiary investigation program to avoid developing an overly conservative solution resulting in a potentially excessively costly foundation improvement requirement. This final stage of the site investigation program consisted of Becker Penetration Testing (BPT) using a HAV-180 Becker Hammer drill rig equipped with an ICE Model 180 diesel hammer on 22 October 2002. Five BPTs (BPTs 02-1 to 02-05) were advanced to depths between 9.5 m (31 ft) and 21.5 m (70 ft).

The BPT consists of driving an approximately 170 mm (6.6 inch) diameter close-ended casing into the ground with a diesel hammer and recording the number of blows required to advance the casing each 305 mm (1 ft). The hammer energy varies with the penetration resistance, which is indicated by the bounce chamber pressure in the diesel hammer. The bounce chamber pressure was continuously monitored for each blow. In addition, the casing friction was measured at BPTs 02-4 and 02-5 by installing a load cell between the BPT rig's hydraulic ram and the BPT casing. The casing friction was assessed by sequentially installing a 2.4 m (8 ft) section of BPT casing as described above followed by measuring the friction force required to pull out the casing 150 mm (6 inches) and recording the peak friction force every 25 mm (1 inch). The recorded BPT data was then converted to SPT-N_{60} values using the empirical correlations recommended by Harder [2] and Sy [3].

The approximate locations of DHs, CPTs and BPTs are shown on Figure 2, which includes the proposed footprint of the combined fire hall and office building.

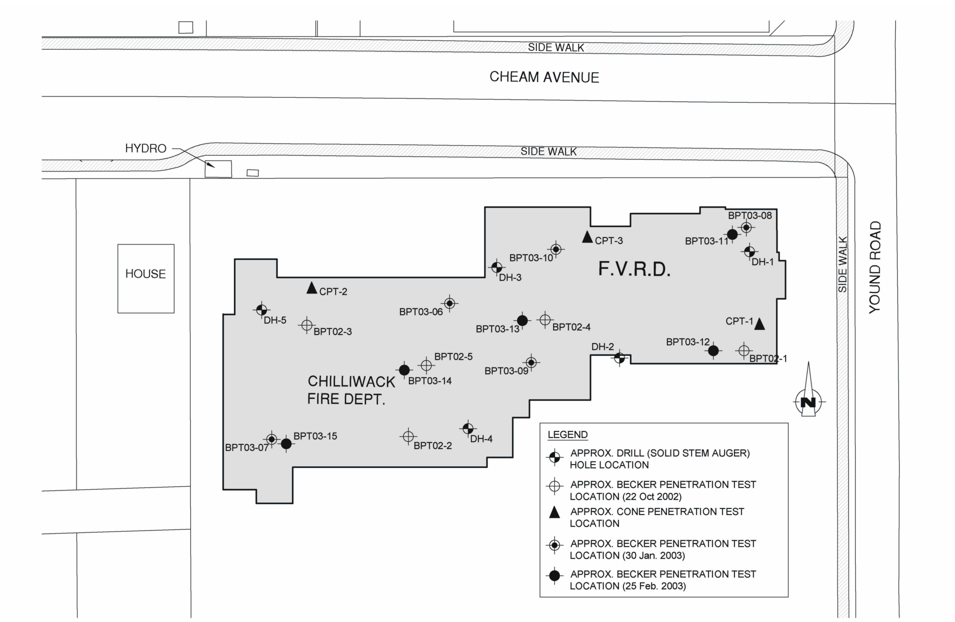

Figure 2 Site plan with test hole locations.

SUBSURFACE CONDITIONS

The soil conditions generally consisted of granular fill over interbedded sand and silt layers underlain by granular deposits. The fill thickness was typically about 0.3 m (1 ft), but soft silt fill extended to 1.5 m (5 ft) depth at one location. The sand content in the underlying interbedded deposit appeared greater than the silt content and the sand content was even significant in the silt zones, which resulted in a generally cohesionless deposit. However, cohesive silt zones up to about 0.3 m (1 ft) thick were occasionally encountered immediately below the fill at a few test hole locations. The cohesive and cohesionless zones were typically firm and loose to compact, respectively. The underlying native granular deposit was typically encountered at about 3 m (10 ft) depth and consisted of sand with variable gravel content and minor silt content and occasional cobbles. The upper zone of this granular soil deposit was compact to very dense with typically equivalent SPT-N_{60} values of the order of 17 blows/ft or more to approximately 6.5 m (21 ft) depth. However, loose to compact zones up to about 2.5 m (8 ft) thick existed between 6.5 m (21 ft) and 10 m (33 ft) depth. Interpretation of BPT data indicated dense to very dense granular soil from about 10 m to 15 m (33 ft to 49 ft) depth over compact to dense granular soil to about 20 m (66 ft) depth, which in turn was underlain by very dense granular soil.

The general soil stratigraphy is shown on Figure 3, which includes the SPT-N_{60} values converted from the BPTs to provide a trend of the density of the granular soils. The shown BPT data was converted to equivalent SPT-N_{60} values by using the Harder method and this method was also used consistently in the following sections of the paper.

The groundwater table was estimated at about 2.5 m to 3.0 m (8 ft to 10 ft) depth based on measurement in a monitoring well and interpretation of the recorded CPT data.

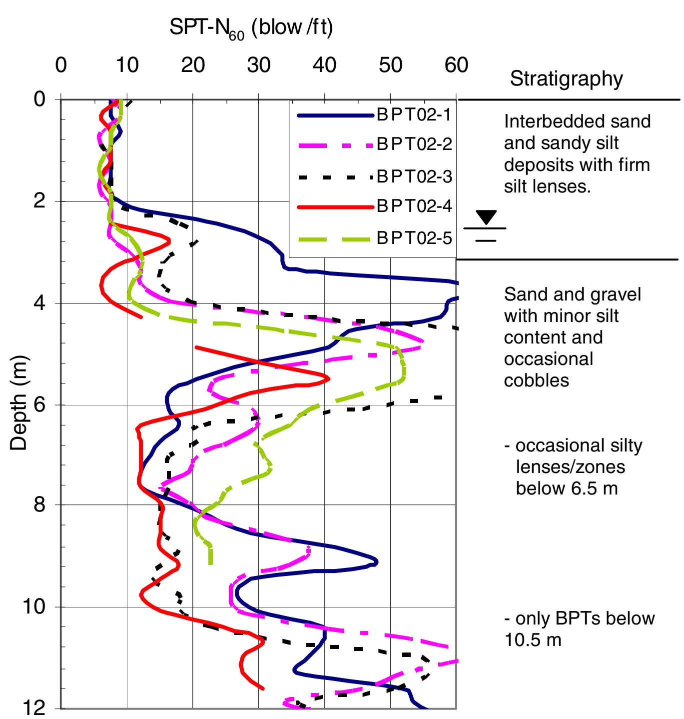

Figure 3 General soil stratigraphy and SPT-N_{60} values from BPTs.

GEOTECHNICAL EVALUATION

Analyses were carried out to assess the liquefaction susceptibility of the encountered soils. These analyses were based on potential triggering shear stresses from a 1 in 475 year design earthquake exerting a firm ground (i.e. bedrock) peak horizontal acceleration of 0.17 g, which was assessed to result in a potential ground surface peak horizontal acceleration of 0.23 g due to soil amplification. The results of the analyses indicated that up to approximately 1 m (3 ft) of liquefaction susceptible soil may exist at about 3 m (10 ft) depth, which may be underlain by thin discontinuous lenses of liquefaction susceptible soil within a zone from about 3.5 m to 4.5 m (11 ft to 15 ft) depth. In addition, liquefaction susceptible zones may exist between 6.5 m to 9.5 m (21 ft to 31 ft) depth with an accumulated thickness estimated up to about 2.5 m (8 ft). It was deemed necessary to implement ground improvement for foundations underlain by these potentially liquefaction susceptible zones to conform with the performance requirements to a post-disaster building.

Several ground improvement methods were evaluated to eliminate the susceptibility of liquefiable soils. Vibroflotation with stone columns to minimum depths of about 10 m to 11 m (33 ft to 36 ft) was a feasible solution, but the cost would exceed that assumed in the design/build tender. The Dynamic Compaction method could also address the liquefaction concerns by dropping a weight from a crane and thereby densifying the liquefaction susceptible soils. However, several fairly vibration sensitive structures were located in the vicinity of the site, which made the application of this ground improvement method less desirable. The third ground improvement method considered was Rapid Impact Compaction (RIC). The RIC was developed in the early 1990s in Europe and is a fairly new ground improvement method to the North American market. RIC consists of hydraulically dropping a 7.5 tons weight from a controlled height with a conventional tracked excavator equipped with a 1.5 m (5 ft) diameter foot. A photo of the RIC equipment is shown on Figure 4.

Figure 4 The RIC equipment.

The RIC hammer rate is 40 to 60 blows per minute with the foot maintaining in contact with the ground to allow efficient and safe transfer of energy. The hammer drop height, number of blows, penetration per blow and total penetration are recorded by the RIC data acquisition system, which can also control the final set to a predetermined penetration per blow. The RIC is carried out at close spacing with many compaction locations within an area of 6 m by 6 m (20 ft by 20 ft). Locally, the depth of impact is often of the order of minimum 6 m (20 ft), but depth of impact up to almost 10 m (33 ft) has been observed on projects in Asia. In comparison to vibroflotation with stone columns, the RIC method can be as much as about three to four times less expensive.

Based on an evaluation of these ground improvement methods, the RIC method was selected to address the soil liquefaction concerns. It was assessed that completion of the RIC method would eliminate the liquefaction concerns in the upper zone, but a limited thickness of liquefaction susceptible soils could still remain near the bottom of the lower zone ending at about 9.5 m (31 ft) depth. The geotechnical recommendations for the proposed building included completion of RIC followed by construction of conventional shallow-depth spread footings with a recommendation to structurally stiffen the structure to accommodate post-earthquake settlements due to the potential existence of deeper liquefaction susceptible soil. Post-earthquake total settlements were estimated to be of the order of 50 mm to 70 mm (2 to 3 inches) with differential settlements of the order of 50 mm over 10 m (2 inches over 33 ft).

RIC PILOT TESTING PROGRAM

The potential effectiveness of the RIC method was evaluated in a pilot program that provided requisite information for preparation of specification for the RIC work. A one-day RIC pilot program completed in December 2002 was followed a few days later by advancement of BPTs to evaluate the density increase in granular soils due to the ground improvement works. This in-situ investigation method was chosen due to the possibility of premature refusal with other in-situ investigation methods such as the CPT.

The RIC pilot program was initiated after completion of limited stripping of surficial vegetation to expose a general granular soil, which in some areas had a relatively high content of friable silt material. The weather conditions in the days prior to the RIC were dry with negligible to no precipitation. The number of RIC locations and passes were varied in five areas (Areas 1 through 5). These approximately 6 m by 6 m (20 ft by 20 ft) areas were located such that each area contained a BPT advanced prior to the RIC pilot program. Each RIC pass within the areas resulted in depressed zones as shown on Figure 4. The areas were re-leveled upon completion of each RIC pass to provide an indication of the average settlement of the 6 m by 6 m (20 ft by 20 ft) area. The number of RIC passes and number of compaction locations for each area were added to the results of BPTs completed before and after the RIC pilot testing program shown on Figure 5, which also indicate the total average settlement estimated for each area.

Quality control data was only recorded by the RIC data acquisition system at Areas 1 and 4, which indicated average blows per RIC location of about 30, an average final set at about 10 mm/blow ($^2/_5$ inch per blow) and average total penetration of about 0.4 m (16 inches). It was judged that the relatively high silt content at many locations in the upper zone prevented a smaller final set such as 5 mm/blow ($^1/_5$ inch per blow).

The combination of the observed ground settlements upon completion of the RIC pilot testing program and the estimated equivalent SPT-N_{60} values clearly showed that the RIC densified and, consequently, improved the soil's in-situ state and hence improved the ground in terms of resistance to potential liquefaction. All post-BPTs indicated higher SPT-N_{60} values above a dense to very dense zone at about 5 m (16 ft) depth, whereas improvement was only encountered below this zone when the 6 m by 6 m (20 ft by 20 ft) area received two passes and contained at least 13 RIC locations (i.e. Areas 1 and 4). Also, the equivalent post SPT-N_{60} values were generally higher in the zone from about 5 to 9 m (16 ft to 29) depth at Area 1, which had four more RIC locations in the second pass than Area 4.

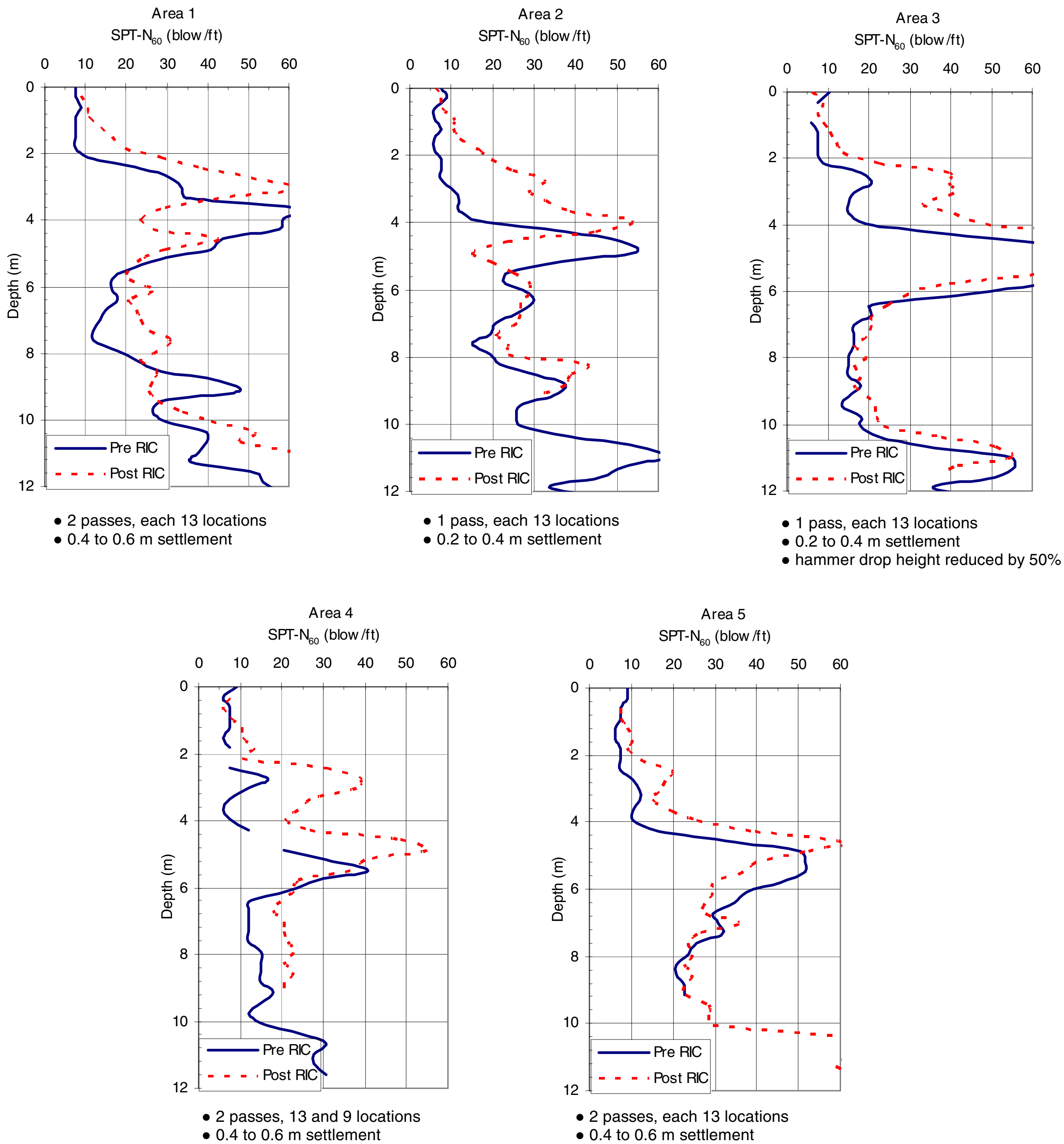

Figure 5 Results of pre- and post-RIC in different areas.

Based on the above observations and results, it was assessed that the RIC could sufficiently increase the density and improve the soil conditions to almost 9 m (30 ft) depth at the subject site provided the RIC was carried out in a similar fashion as at Area 1 with a final set of maximum 10 mm/blow ($^{2}/_{5}$ inch per blow). A liquefaction assessment of the soil conditions at Area 1 after completion of the RIC indicated that the risk of seismic liquefaction induced by a 1 in 475 year earthquake would be below an acceptable risk threshold defined by the project's design criteria. Thus, provided the entire footprint of the proposed building was induced to the same RIC works as at Area 1, it was deemed possible to reduce the structural stiffness requirements for the building creating a more favourable economic development.

CONSTRUCTION PROGRAM

Wet weather conditions prior to and during the RIC construction program necessitated sub-excavation and replacement of upper soils containing significant fines (particles finer than 75 μm). Generally, the sub-excavations extended to about 0.5 to 1.0 m (2 to 3 ft) depth, which were backfilled with one lift of sand with minor gravel content followed by compaction with a smooth drum ride-on vibratory compactor. The RIC works were carried out on the entire building footprint after completion of sub-excavation and replacement. Monitoring of the RIC work and review of information recorded by the RIC data acquisition system indicated that the RIC work was completed in compliance with the above criteria.

Vibration sensitive structures were located close to the outer edge of the RIC works as indicated on Figure 6. The shown residential house was located approximately 6 m (20 ft) away from the RIC area and pre- and post-construction surveys completed for this house indicated no structural damage had been caused by the RIC. A vibration sensitive buried utility line existed along the north edge of the proposed complex, which was located approximately 5 m (16 ft) away from the RIC area. Vibration monitoring directly on the exposed utility line indicated Peak Particle Velocities generally below the defined threshold of 25 mm/sec (1 inch/sec). A shallow trench was excavated between the vibration sensitive structures as shown on Figure 6 to dampen the impact of the RIC work.

a) Residential house about 6 m from RIC area

b) Buried utility line about 5 m away from RIC area

Figure 6 Vibration sensitive structures located next to RIC area with excavated trench to dampen impact of RIC.

A total of five BPTs were advanced to about 12 m (39 ft) depth on 25 February 2003, which was approximately one month after completion of the RIC work. The equivalent SPT-N_{60} values for these five BPTs are shown on Figure 7 to provide a general trend of the soil density after completion of the RIC work. The figure generally shows equivalent SPT-N_{60} values of minimum 20 blows/ft below the estimated ground water level at about 2.5 m to 3.0 m (8 ft to 10 ft) depth except for zones generally between 6.5 m (21 ft) and 9.0 m (30 ft) depth at BPTs 03-11 and 03-13. Based on the results of CPTs carried out in the vicinity of these two BPTs, it is judged these deeper zones with less than 20 blows/ft is associated with increased fines content in the granular deposit.

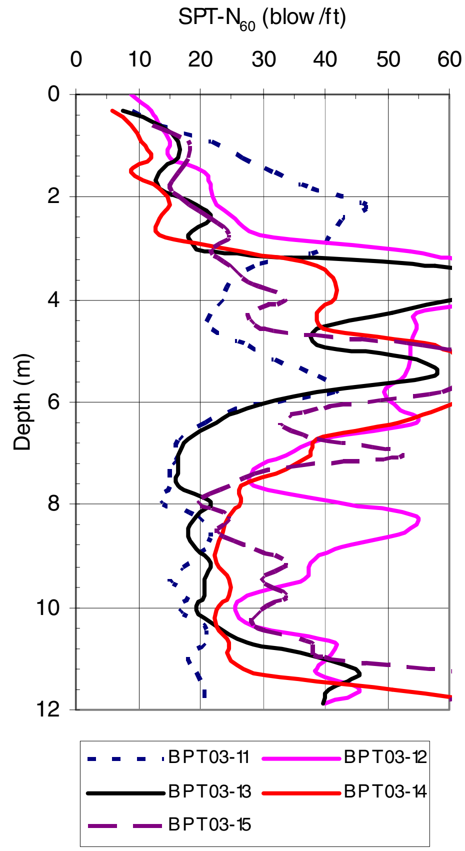

Figure 7 Results of BPTs carried out after completion of the RIC works.

Comparison of the general trends of the SPT-N_{60} values on Figures 3 and 7 indicate that the RIC works appreciably densified both the upper and lower liquefaction susceptible soils at the project site. BPTs were advanced before and after the RIC works at almost the same location in three areas. The pre- and post equivalent SPT-N_{60} values are shown on Figure 8 at these three locations. Figure 8a indicates that the RIC works densified the soil to a depth of about 8.5 m (28 ft) with a minimum increase of approximately 10 blows/ft. Figure 8b shows a considerable increase in blow counts to a depth of about 5.5 m (18 ft) and a smaller increase below this depth. It is possible that this lesser blow count increase is associated with soil heterogeneity and inaccuracy with the testing method. Figure 8c shows a considerable increase in blow count to approximately 6.0 m (20 ft) depth and a negligible impact in the soils below. It should be noted that the pre- blow counts below a depth of about 6.0 m (20 ft) at the location shown on Figure 8c was fairly high at a minimum of 20 blows/ft.

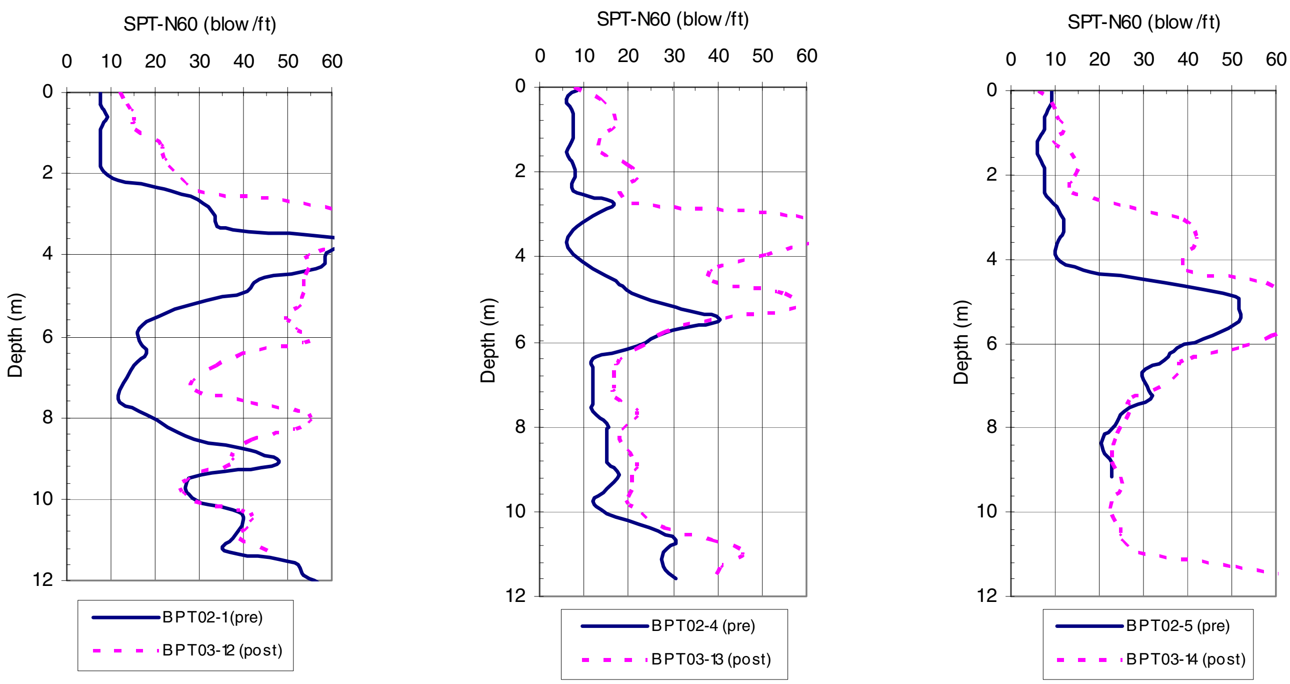

Figure 8 Comparison of BPTs carried out before and after completion of RIC at three locations.

CONCLUSION

The RIC ground improvement method proved to be successful in densifying liquefaction susceptible soils to an acceptable level for the subject site. The general trends of SPT-N_{60} values shown on Figures 3 and 7 of pre- and post- BPT data clearly indicates that the RIC method densified the in-situ soils appreciably to a depth of about 6.0 m (20 ft). In addition, it was judged that granular zones on the subject site with equivalent SPT-N_{60} values of about 15 blows/ft or less between depths of 6.0 m (20 ft) and almost 9 m (30 ft) were densified to equivalent SPT-N_{60} values of about 20 blows/ft or more.

In addition, the RIC completed at the above site and experience from other similar sites that the authors have been involved with indicate that RIC can often be completed as close as about 5 m (16 ft) to adjacent structures without vibration from the compaction works inducing structural damage.

The RIC method appears to offer an effective alternative to other more commonly used ground improvement methods. This appears to particularly be the case where the required depth of in-situ ground improvement is less than about 6.0 m (20 ft) and even up to depths of almost 9 m (30 ft) at sites with similar subsurface conditions as those described in this paper.

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