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ABSTRACT

For moderately loaded structures founded on liquefiable soils, spread footings on improved ground can provide considerable cost savings over deep foundation options. Liquefaction mitigation by ground improvement must be properly designed and executed; and should include a field verification program. Although densification is the most effective method of achieving verifiable mitigation of liquefaction susceptible soils, vibro-densification methods are often disregarded for urban sites due to concern for adjacent structures and utilities. An alternative to vibratory methods is compaction grouting, which can achieve densification of cohesionless materials while avoiding excessive vibration of adjacent structures.

Recently, compaction grouting was successfully applied to densify a thick loose sand layer (up to 40 feet) for a large development site in an urban environment. This densification significantly increased the factor of safety against liquefaction and reduced potential liquefaction-induced settlement to under 0.5 inch. The compaction grouting program included automated data acquisition and processing and three-dimensional visualization components to ensure quality control and assurance. In addition, the site improvement program was fully verifiable, as the ground improvement program included a comparison of cone penetrometer tests (CPT) conducted prior to and following treatment.

Although compaction grouting has been well utilized for several years, the potential for liquefaction mitigation in urban environments is not well established. However, ground improvement through compaction grouting can be a cost-effective alternative to drilled shafts or driven piles on liquefiable sites. This paper includes a description of the site conditions, the compaction grouting program (including automated data acquisition instrumentation and visualization), site instrumentation, post-treatment evaluation of the mitigation procedures, and analysis of the response of adjacent structures.

INTRODUCTION

Once the location of several buildings, the site proposed for a new patient pavilion for an adjacent hospital complex in New York City had been converted to a paved parking lot. Re-development of this lot for the new building in its urban, hospital setting posed several foundation challenges given the subsurface conditions.

Structures previously on the site had basement levels and tunnels. Personnel knowledgeable with razing of these structures reported that the basement and tunnel walls remained below the parking lot and that areas between these walls were backfilled with building demolition debris. The basement floors of these structures, seated 8 to 10 feet below grade, were broken and left in-place. Other structures without basement levels had also been razed. Reinforced concrete slabs and the foundations of these structures were also present. For one of these structures, numerous, closely spaced concrete filled pipe piles remained below grade.

Complicating these surficial conditions was the presence of relatively loose sand extending to depths as great as 50 feet below grade. The upper 10 to 15 feet of this sand was of fill origin while that below was natural. With the groundwater table located approximately 15 feet below grade, the loose relative density of these soils rendered them subject to liquefaction.

Consideration was first given to supporting the new building on deep foundations. Drilled shafts were favored over driven piles due to their resistance to buckling under a seismic event and concerns with pile driving in close proximity to an operational hospital facility. Complicating installation of drilled shafts were the buried remains of structures once present at the site and the need to install many drilled shafts to support the structural floor slab of the building's ground floor. The costs of this foundation system were weighed against the costs of employing compaction grouting to mitigate the potential for soil liquefaction and allow the use of conventional spread foundations to support the building on the improved ground conditions. The compaction grouting option proved to be considerably more cost effective but required careful execution and monitoring to ensure the ground was densified to the degree required and that the densification process did not harm adjacent structures and underground utilities.

DESCRIPTION OF NEW BUILDING

Plans for the new building were developed for the construction of a six (6) story structure with a mechanical floor above and a basement level below a portion of the building. At some time in its future, four (4) more stories are to be added to the structure requiring that its foundations be proportioned for loads corresponding to a 10-story building. For this building height, the maximum interior column loads were estimated to equal 1,600 kips.

The structure plan dimensions are 160 feet by 260 feet, resulting in a footprint of 41,600 square feet. Finished floor for its ground floor is stepped to match or slightly elevate the floor above existing site grades and match the floor levels of the two adjacent buildings. The basement of the new building is approximately 15,400 square feet in size and is located in the northwest corner of the structure. It has a corridor connecting it to an existing utility tunnel servicing the adjacent existing buildings. Finished floor of the basement and the corridor is 16 feet below the ground floor.

Outside of the basement area, the structure is supported on conventional spread foundations. Columns along the basement's perimeter and within its interior are supported on a mat foundation. The spread footing sizes range from 3 feet square to 16.5 feet square, with bottom of footing (BOF) depths of 7 to 14 feet below existing grade. The BOF depths were varied to eliminate imposing lateral loads on the walls of the new basement and existing tunnel, and to seat the foundations below the basement floor level/construction demolition debris of former structures. With the mat foundation bearing 19 feet below existing grade as well as below the groundwater table, the applied structural load is totally compensated by the weight of soil removed for its construction.

SITE DESCRIPTION

Figure 1 illustrates the outline of the new building relative to the location of former structures, existing buildings, roadways, and utility and subway tunnels. The existing buildings are operating medical facilities, one being a 4-story structure supported on conventional spread footings bearing on compacted structural fill and the other being a 6 to 16 story structure supported on H-piles end-bearing on bedrock.

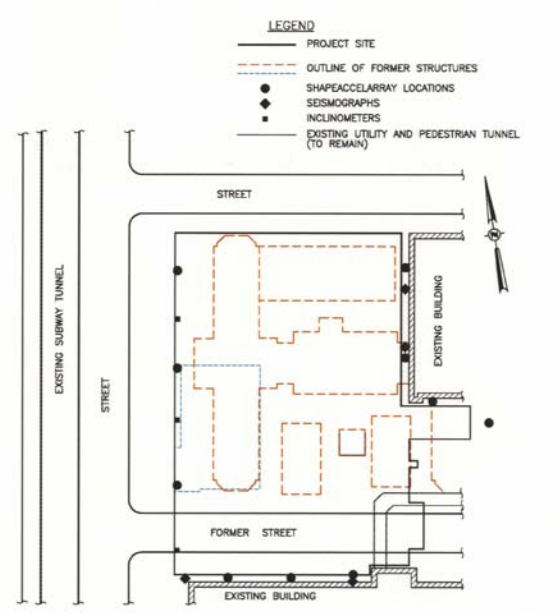

Fig. 1. Schematic of Site Dimensions, Adjacent Structures, Infrastructure and Instrumentation.

A trailer-mounted MRI Facility, not shown in this figure, was also present on the site and was relocated within 50 feet of the planned compaction grouting operations. The facility, with strict vibration criteria, remained operational throughout the compaction grouting program.

Both the utility tunnel and the subway tunnel running along the west and east sides of the site were active. Permits to work in close proximity to the subway tunnel were required by the Metropolitan Transportation Authority (MTA) of New York City and included specific conditions regarding the sequence of work and the monitoring of the tunnel for vibrations.

In addition to existing buildings, a number of underground utilities were present within the proposed building footprint and along roadways bordering the site. These utilities included electric, gas, telephone, water, stormwater and sanitary sewer. Water and sanitary sewer lines along one street (identified as Former Street below in Figure 1) were planned for relocation while all others were not. Sanitary sewer lines along the west side of the site had deep invert elevations (8 to 13 feet below grade) that required particular care be exercised to avoid damaging the lines during compaction grouting.

SUBSURFACE CONDITIONS

The site's subsurface conditions were investigated through the advancement of test borings and cone penetration tests (CPT's), the excavation of test pits, and laboratory testing of selected samples recovered from the test borings.

A total of 21 test borings were advanced and 5 CPT's performed. Each test boring was extended to refusal on bedrock and included the performance of standard penetration tests and the recovery of "undisturbed" samples of cohesive soils. They were advanced through drilling of flush joint casing and the use of "drilling mud." One CPT was extended to refusal whereas the remaining four were terminated at a depth of 50 feet.

To closely examine the nature of the fill present on-site and to ascertain whether the location of the former structures matched those illustrated on record drawings, a total of 12 test pits were excavated. They were strategically located along the perimeter and at specific interior points of these structures to examine the thickness and composition of the existing basement walls and floors and the composition of the demolition debris placed within the interior of the building footprints. The locations of the walls and slabs exposed in the test pit excavations were surveyed and compared to the outline of the former buildings shown on record drawings.

Laboratory testing of samples recovered from the test borings included moisture content, Atterberg Limits, particle size analyses, and one-dimensional consolidation tests.

A generalized soil profile developed from the subsurface investigation program is presented in Figure 2. The upper 10 to 15 feet of the profile consists of an uncontrolled fill, which, in its upper half, included 2-foot wide basement walls of concrete and masonry construction, floors of the former structures and structural debris containing sections of steel columns, brick, concrete and other miscellaneous building materials.

Sand, 35 to 70 feet in depth, is present below the fill in two distinct zones; an upper, poorly graded zone of loose relative density and a lower well graded zone with a medium dense relative density. The recovered samples were typically fine to medium in texture and contained trace amounts of gravel and silt. Within both of these zones, the sand was found to contain seams and/or layers of silt or sand containing appreciable amounts of silt.

Varved silt and clay with a consistency of medium stiff to stiff underlies the sand and extends to depths of 70 to 120 feet. Consolidation tests performed on several samples indicate that the silt and clay is overconsolidated to stresses ranging from 400 to over 20,000 pounds per square foot.

Glacial till of limited thickness is present below the silt and clay and overlies bedrock, the surface of which dips some 30 feet in elevation across the site. The upper several feet of bedrock was found to be weathered to varying degrees and thicknesses.

Groundwater was observed at approximately 15 feet below existing grade.

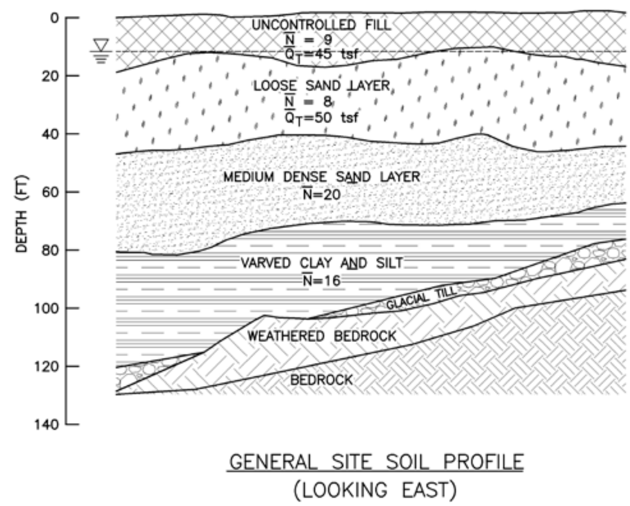

Fig. 2. Approximate Soil Stratigraphy.

PRETREATMENT LIQUEFACTION AND SEISMIC SETTLEMENT ANALYSIS

The potential for liquefaction and liquefaction-induced settlement of the upper loose sand was evaluated using the CPT data and current methods within the geotechnical practice. The peak ground acceleration (PGA) employed in the analysis was 0.225g, based on consultation with the NYC Building Department. A USGS probabilistic hazard deaggregation analysis (USGS, 2009) identified the mean earthquake magnitude (M) as 5.68.

The factor of safety (FS) against liquefaction and the liquefaction-induced settlement was calculated for each CPT test using LiquefyPro, a commercially available software package (LiquefyPro, 2009). The factor of safety against liquefaction was calculated based on the procedures proposed by Robertson and Wride (1998), also described in the NCEER summary report (Youd et al., 2001). The factor of safety against liquefaction is based on an 'equivalent clean sand cone penetration resistance' ($(q_{C1N})_{cs}$), which accounts for fines content and overburden stress. The fines content was estimated from correlations to tip resistance and sleeve friction based on Robertson and Wride. The estimated fines contents correlated well with the laboratory gradation test results conducted for this site and shown as a range in Figure 3.

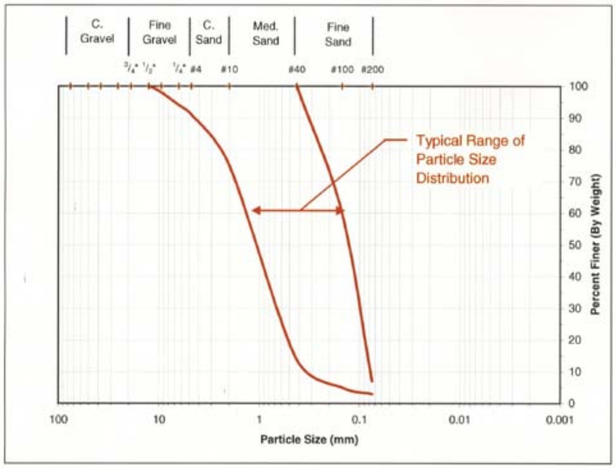

Fig. 3. Typical Range of Loose Sand Gradation

The liquefaction-induced settlement was calculated using the method proposed by Ishihara and Yoshimine (1992). The volumetric strain was calculated as a function of relative density, which was determined by correlating tip resistance to SPT blow count and then to relative density. The dry sand settlement (not liquefaction-induced) was calculated based on Tokimatsu and Seed (1987).

Figure 4 shows a sample plot of the factor of safety and cumulative seismic settlement as a function of depth for one of the CPT's conducted on this site. Although all CPT results are not presented in this paper, a total of 10 tests were conducted prior to treatment. The computed factors of safety for each were similar, typically being equal to or slightly greater than 1.0 but with seismically induced settlements ranging from 2 to 5 inches.

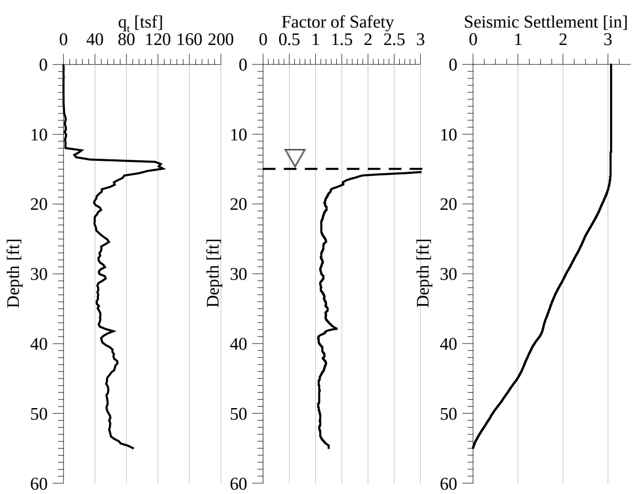

Fig. 4. Sample CPT and Calculated Factor of Safety against Liquefaction and Seismic Settlement as a Function of Depth. This CPT test was conducted prior to ground treatment.

COMPACTION GROUTING OBJECTIVES

With the potential for excessive seismically induced settlements, the initial goal of the compaction grouting program was to increase the factor of safety against liquefaction and reduce the potential seismically induced settlements. Specifications developed for the work required that the post-treatment factor of safety against liquefaction equal at least 1.40 and that the seismically induced settlements not exceed one (1) inch. Methods of computing each were specified so as to eliminate any ambiguity in assessing the end result of the work.

An additional objective was the densification of the ground to a degree which would allow the use of spread foundations for the building's support. The performance specifications required that the ground be densified to support spread foundations proportioned for a net allowable soil bearing pressure of 4,000 psf. Under the building's static loads, the total and differential settlement of the foundations proportioned for this pressure could not exceed one (1) inch and one-half (1/2) of an inch, respectively. Methods of computing these settlements were again specified to eliminate any ambiguity in assessing the end result of the work.

ADJACENT STRUCTURES AND SITE INSTRUMENTATION

With the new building to be constructed directly adjacent to two existing buildings, an MRI facility, utility tunnel, and underground utilities, movements and vibrations had to be closely monitored and the compaction grouting program modified when limits were exceeded. Conventional methods of monitoring included optical and laser leveling, seismograph, and slope inclinometer measurements. ShapeAccelArray (SAA) real-time monitoring devices were employed to collect continuous three-dimensional displacement and acceleration data.

Elevation marks were established at regular intervals along each building and at various locations on their ground/basement floors. Upon their exposure, telltales were attached to the top of the utility tunnel and a sanitary sewer line. Each of these utilities had to remain in service throughout construction. The elevation surveys of these locations were performed on a regular basis during adjacent grouting and, if necessary, immediately after daily laser survey measurements of their movement. The laser surveys were conducted continuously throughout each workday. Targets with receivers were set on the exterior of the buildings and, for one building, on an interior basement wall adjacent to the work areas. The utility telltales were also equipped with targets with receivers. Audible sounds were emitted if vertical movements occurred at any of these locations, prompting the need to evaluate the compaction grouting methodology and make changes which would eliminate additional structure movement. Any movement detected by these surveys required that the work be stopped immediately.

Conventional seismographs were installed alongside the buildings to monitor the ground vibration levels (peak particle velocity). A limit of one (1) inch per second was established as the maximum allowable peak particle velocity which the buildings and tunnels could experience.

Slope inclinometer casing was installed along the west side of the site as a condition of the work permit received from the MTA. The conditions of this permit allowed for an initial line of compaction grouting to be completed closest to the subway tunnel. Following this work, the lateral ground movements induced by work performed on the interior of the site (inside this line) could not exceed two (2) inches at the slope inclinometer locations, from the base of the subway tunnel to its top. The slope inclinometer casings were located approximately 66 feet away from the subway, 2 to 3 feet from this initial line of grouting.

Monitoring of lateral ground displacements and vibrations was also performed using a 3-D monitoring system known as ShapeAccelArray manufactured by Measurerand, Inc. This system was chosen for its sensitivity and ability to measure vibrations in units of acceleration, and, most importantly, it allowed for real-time monitoring of the vibrations and lateral deflections. Its use was critical is assessing the lateral ground movements which were occurring adjacent to H-piles supporting the 6 to 16 story building. Lateral ground deflections within one (1) foot of these piles could not exceed 0.25 inches. A laptop computer with a visual alarm display activated when such displacements were occurring was utilized at each SAA location as adjacent grouting took place. Grout holes were located as close as eight (8) feet from the face of the building and, at a few locations, at a distance of four (4) feet from the edge of a pile cap.

Figure 1 includes the locations of the instrumentation described above.

Compaction Grout Installation

Low mobility grout was installed on a 9-foot triangular grid layout, between depths of 10 to 55 feet. The sequencing was designed to have primary, secondary, tertiary, and quaternary grout points, such that adjacent grout locations were not installed sequentially. The spacing and grout mix were determined after several iterations of on-site testing, where grout mixes and hole spacing combinations were refined to reach the target cone penetration test values. The grout was injected through a continuous steel casing with an inside diameter of 4 inches. This casing was advanced to full depth using a vibratory hammer and downward thrust of the drill rig. The upper 10 to 15 feet was predrilled to bypass the rubble and urban fill within that layer. The photograph in Figure 5 shows one of the drill rigs and its vibratory hammer used during construction; the grout casing is almost advanced to full depth in this photograph.

Fig. 5. Photograph of Grout Casing Installed to Approximately 45 ft depth.

Volume and pressure criteria were established during the on-site field testing stage. If either of the criteria was achieved during a 2 foot stage, the casing was advanced upward to the next stage. The volume, pressure, and injection rate were monitored in real-time and logged with an automated data acquisition system mounted to the drill rig. The automated data acquisition and presentation allowed for efficient decision-making by field engineers and operators. Figure 6 presents an example of the automated data acquisition and processing system output.

In addition to the site-based data acquisition system, a three-dimensional visualization package was employed to assist in evaluation of the grouting process. The three-dimensional rendering of grout volume and pressure was completed automatically, based on data acquired from the field. Figure 7 shows an example of the 3D graphics employed for this project.

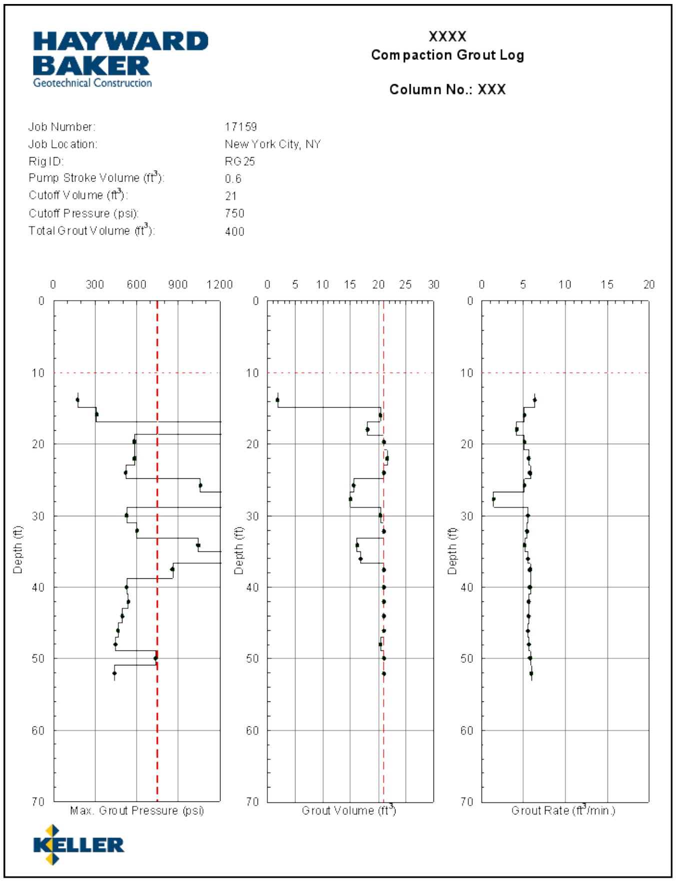

Fig. 6. Example of Automated Data Collection and Processing Output.

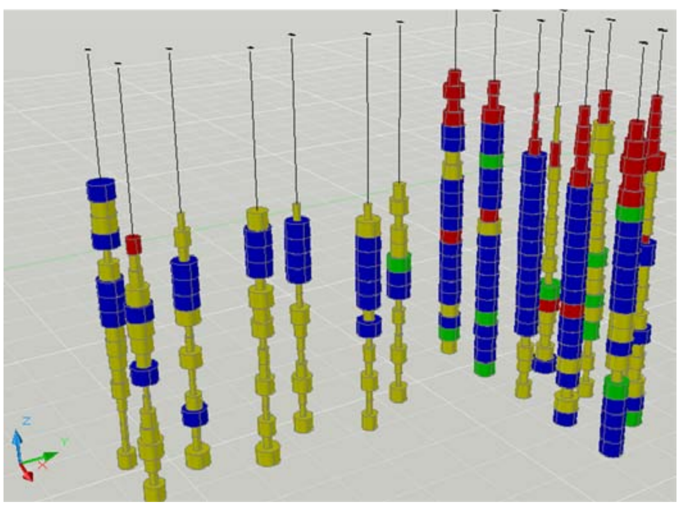

Fig. 7. Example of Grout Visualization Output. Cylinder size represents grout volume and cylinder color represents grout pressure.

GROUND IMPROVEMENT RESULTS

The success of the compaction grouting program was verified by over 90 CPT tests, conducted at the interstice of three treatment points, as shown in Figure 8. Figure 9 shows the comparison between the post-treatment CPT and the pre-treatment CPT conducted between a primary, secondary, and tertiary hole (153, 152, 131, respectively, as shown in Figure 8). The cone tip resistance increased by over 100% throughout the majority of the loose sand layer, and the factor of safety against liquefaction was increased to over 1.5 for all depths. In addition, the anticipated liquefaction-induced settlement was reduced from 2 to 5 inches to less than 0.1 inch.

Grout volume and pressure for each stage is shown in Figure 10. The primary hole (153) reached the volume criteria throughout the majority of the sand layer, whereas the secondary and tertiary holes mainly reached pressure refusal. This example was representative of the majority of test locations.

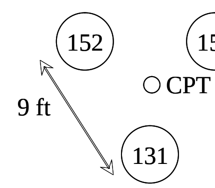

Fig. 8. Location of Post-Treatment CPT Test, relative to primary (153), secondary (152), and tertiary (131) grout holes.

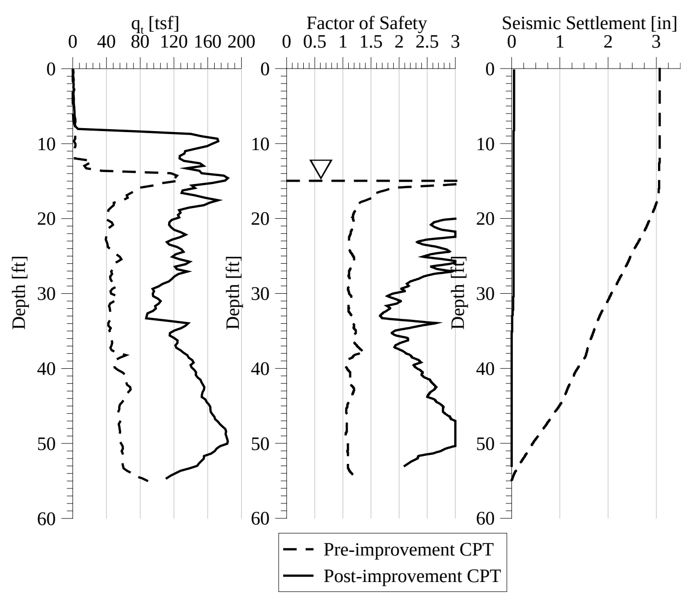

Fig. 9. Post-Improvement Tip Resistance, Factor of Safety against Liquefaction, Liquefaction-Induced Settlement.

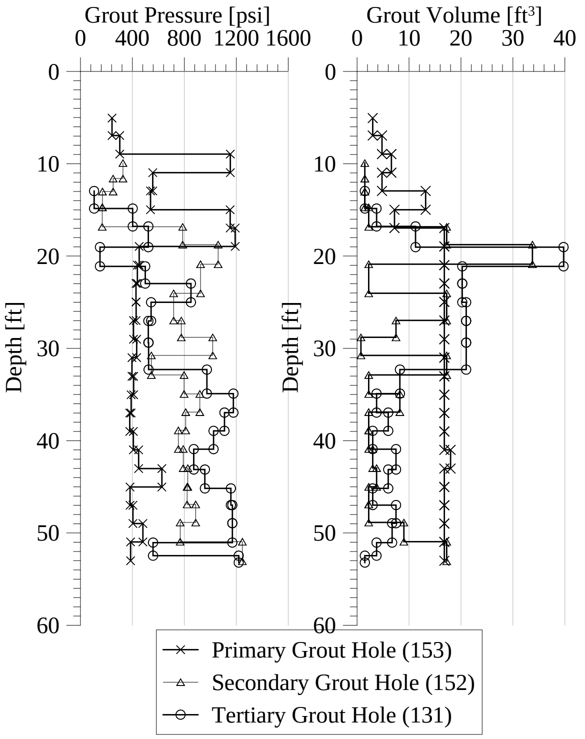

Fig. 10. Grout Stage Pressure and Grout Stage Volume corresponding to results shown in Figure 9.

ADJACENT STRUCTURE RESPONSE

The optical and laser surveys of movement of the two existing buildings and their ground/basement floors indicated that the structures did not experience any movement throughout the compaction grouting program. Of particular concern was the building with conventional spread foundations and a ground floor that was seated on-grade. The closest row of grout holes from the face of this building was 7.5 feet. Grouting was extended to a level of 3 to 4 feet below the bearing grade of foundations supporting this building. With the soils below these foundations and the building interior being well compacted structural fill, the structural fill provided resistance to the grout's injection and facilitated densification of the soils between the building and the line of grout holes. Post-treatment CPT results indicated these soils were adequately densified by the work's performance.

In contrast to grouting adjacent to dense ground conditions surcharged by foundation loads, movement was observed where the grouting was performed adjacent to a sanitary sewer line and utility tunnel where such "overburden" conditions did not exist. Movement of these utilities was limited to one-eighth (1/8) of an inch as grout injection was immediately stopped upon hearing the audible alarms of the laser survey receivers. Their movement was not experienced until the final grouting stage was reached, the elevation of this stage being approximately 2 feet below the utility inverts. Post-treatment CPT results indicated the soils below these utilities were adequately densified by the work's performance even though compaction grouting was prematurely stopped.

Seismograph monitoring performed at four locations immediately adjacent to the two existing buildings resulted in recorded peak particle velocities due to the compaction grouting operation of no greater than 0.39 inches per second. Accordingly, the maximum allowable peak particle velocity of one (1) inch per second was not exceeded during the work's performance. While monitoring the interior of the buildings for movement, the ground vibrations were not perceptible inside these structures.

Vibration monitoring was performed at ten (10) ShapeAccelArray (SAA) installations at the locations shown in Figure 1. Each installation had eight recording depth intervals (referred to as octets), each of which was 6 feet in length. Figure 11 is a display of the ground vibrations monitored by one of the installations, SAA1 located along the western boundary of the site approximately 66 feet from the subway tunnel. The figure presents the recorded ground vibrations in units of acceleration while compaction grouting was being performed 31 feet from the installation. Vibrations shown on the figure were recorded at the third octet with an average depth of 22.5 feet, close to the base elevation of the nearby subway tunnel. The figure has text boxes indicating the stage of the operation over the time required to install the grout ("stinger") pipe to a depth of 50 feet and to withdraw it incrementally in 2-foot stages as compaction grouting was performed. The recorded vibrations were modest and judged to be of no impact to the subway tunnel when compaction grouting was to be advanced to its nearest point to the tunnel, approximately 70 feet.

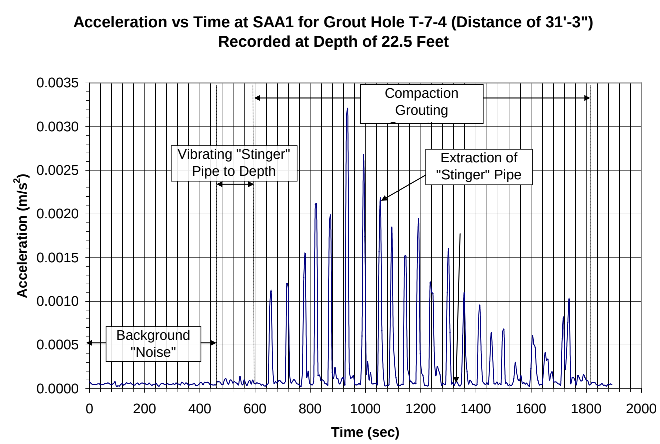

Fig. 11. Example of Recorded SAA Vibration Monitoring Data

Vibration limits given for the operation of the MRI Facility were the most restrictive. They are illustrated in Figure 12 along with the peak velocities recorded by several of the SAA installations. The points plotted on this figure are numbered with the location at which the grouting was being performed. Multiple points are plotted for each grout location as the peak vibration varied as a function of frequency. Variations in the frequency occurred predominately due to the variable speed of the vibratory hammer used to advance and incrementally withdraw the grout pipe. This hammer, an RTG MR 125 V, had a maximum centrifugal force of 1,250 kilonewtons and an operating frequency ranging from 0 to 38.33 hertz. The majority of the plotted points fell below the vibration limit but a few fell above the limit. The majority of these occurred at low frequencies (< 10 hertz) during brief start-up and shutdown moments of the vibratory hammer. Otherwise the vibrations were only slightly exceeded at the longer lasting higher operating frequencies of the hammer. A clear relationship of vibration to distance from the source could not be developed as the ground between work areas and the SAA locations had been treated to varying degrees. Concern over the safe operation of the MRI Facility was not raised as a utility tunnel ran between the unit and the closest point of compaction grouting and it was believed that the tunnel would provide some degree of vibration isolation. The MRI Facility remained in operation throughout the grouting program and did not experience any related operating problems.

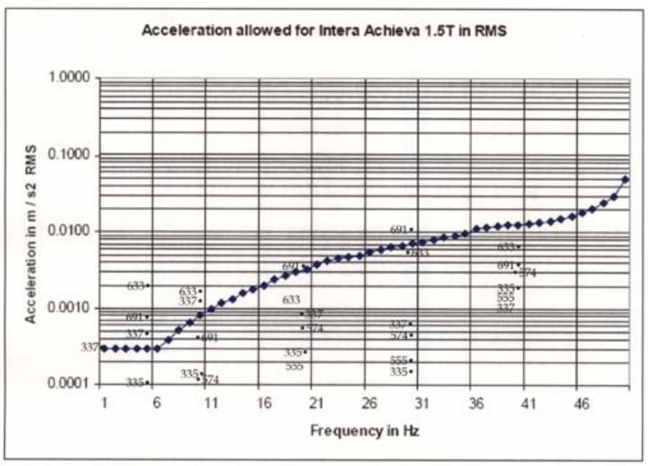

Fig. 12. Vibration Limits & Measured Vibrations for Intera Achieva 1.5T MRI Unit

Three (3) SAA installations were located along the site's south side adjacent to the pile supported building. Grouting of the holes along this building was closely monitored at these locations using a laptop computer. Grouting of the initial hole of the line of holes closest to this building clearly indicated that the deflection criteria of 0.25 inches within one foot of the piles would be exceeded if grout was injected at the pressure refusal criteria that had been established for production grouting. Grouting was therefore performed under reduced pressures and stopped when the visual alarm on the laptop computer indicated the deflection criterion was being exceeded. An injection pressure limit of 400 pounds per square inch was established to limit the lateral ground deflections to 0.25 inches. A row of grout holes along the centroid of the 9-foot triangular grid closest to the pile supported building was added where premature grout returns to the surface occurred. Post-treatment CPT results indicated that the ground was adequately densified by the work's performance.

SAA5 was located directly opposite a pile cap, approximately one (1) foot from its plan location and 3 feet from the nearest grout hole. Figure 13 is a photograph of the field monitoring in progress at SAA5. Figure 14 illustrates the lateral ground movements recorded at this location under the revised pressure criterion. The Y-direction on this figure is perpendicular to the pile supported building and the deflections toward the building are negative. The deflections become positive (away from the building) above a depth of approximately 20 feet as the building has a basement level and grouting above this level is suspected as causing a net "push" of the wall's backfill away from the building's basement wall.

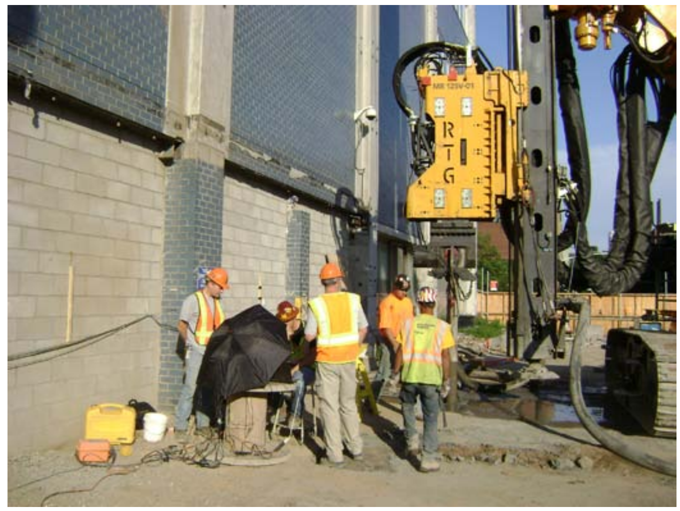

Fig. 13. Photograph of Field Monitoring of SAA5.

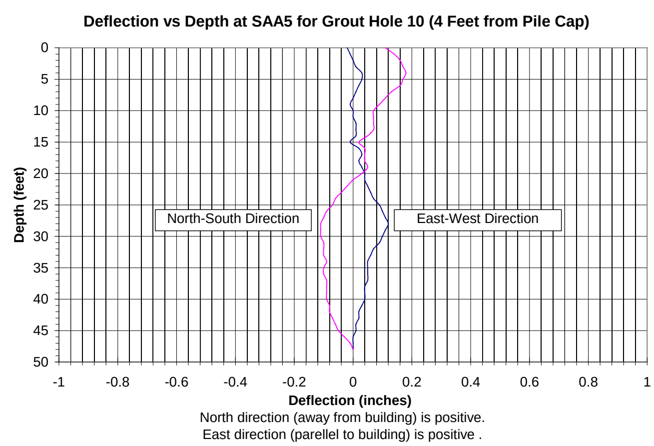

Fig. 14. Lateral Ground Deflection Adjacent to Pile Supported Building

Slope inclinometer casing installed at three (3) locations along the west side of the site was monitored during the compaction grouting program. At two of the locations, monitoring began after the first four (4) rows of grout closest to the subway tunnel had been installed. Lateral ground deflections at these locations did not exceed 0.5 inches. Deflections at the third location, shown in Figure 15, approached 6 inches at a depth of 29 feet. Deflections were less than 2 inches above a depth of 20 feet, this depth corresponding to the approximate elevation of the bottom of the subway tunnel. The inclinometer was located approximately 4 feet from the closest grout hole and the deflections opposite less than the maximum deflection permitted by the MTA.

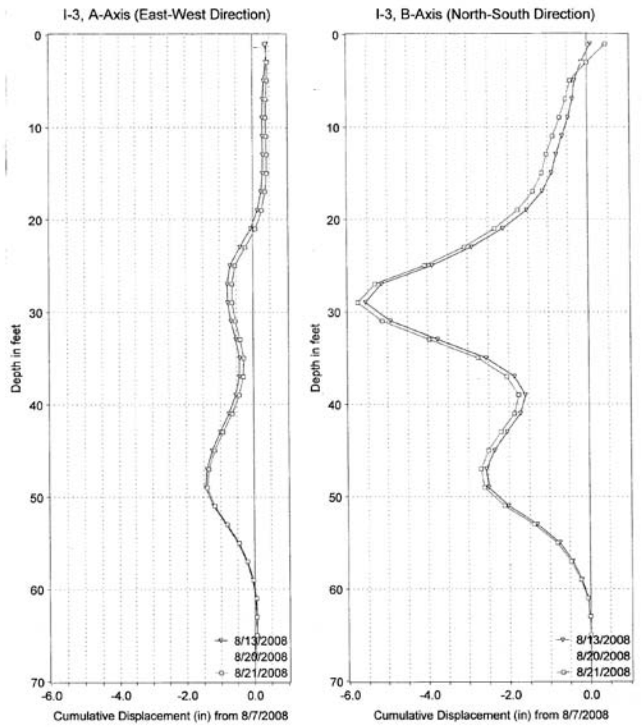

Fig. 15. Slope Inclinometer Profile

CONCLUSION

A compaction grouting ground improvement program was successfully implemented on an urban site to mitigate seismic hazards for construction of a new hospital founded on shallow foundations. Low-mobility grout was injected throughout a 40 foot layer of loose sand, in 2 foot stages, to densify the sand. The ground improvement program included a field test program to ensure sufficient ground improvement, and automated data acquisition and 3D visualization systems to ensure quality control and assurance. This program was conducted with minimal disturbance to adjacent structures and was fully verifiable through post-treatment testing.

Densification of the loose sand layer resulted in an increase in cone tip resistance over 100%, which raised the factor of safety against liquefaction from approximately 1 to over 1.5. Also, the anticipated liquefaction-induced settlement was reduced from 2 to 5 inches to negligible amounts.

The response of adjacent structures was monitored with vibration and deformation sensors, which indicated minimal impact to the adjacent structures. Conventional means of monitoring were employed along with real-time methods which provided for quick reaction to conditions judged potentially damaging to structures or operating equipment. Changes to the compaction grouting program included reduction of injection rates and pressures to lessen lateral ground displacements adjacent to a pile supported building and heave and lateral movement of underground utilities. Close interaction between engineering staff of the Geotechnical Engineer-of-Record and the Contractor resulted in the project's successful performance and end result.

Successful application of compaction grouting for improvement of the seismic and static response of the site soils eliminated the requirement for a deep foundation. In this case, the elimination of piles or drilled shafts provided a significant cost savings to the owner.

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