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ABSTRACT

To improve our understanding of the lateral load behavior of deep foundations in liquefied soil, a series of full-scale lateral load tests have been performed at the National Geotechnical Experimentation Site (NGES) at Treasure Island in San Francisco, California. The ground around the test piles was liquefied using explosives prior to lateral load testing. The goal of the project is to develop load-displacement relationships for bored and driven piles and pile groups in liquefied sand under full-scale conditions for improved and non-improved ground. The results of this investigation confirmed that controlled blasting techniques could successfully be used to induce liquefaction in a well-defined, limited area for field-testing purposes. Excess pore pressure ratios greater than 0.8 were typically maintained for 4 to 10minutes after blasting. Data were collected showing the behavior of laterally loaded piles before and after liquefaction in non-improved ground. Following liquefaction, the stiffness of the soil-foundation system typically decreased by 70 to 80% of its pre-liquefaction value non-improved ground. Ground improvement with stone columns was then performed prior to an additional series of tests. Lateral load tests were again conducted before and after blasting to induce liquefaction. Cone penetration testing following the installation of stone columns found that the density was improved significantly. As a result, the stiffness of the foundation system following blasting was 2.9 to 3.6 times that in the liquefied soil. Subsequent tests involving more than twice as many piles or 50% larger piles provided less than 50% of the increased resistance produced by stone column treatment alone. This study provides some of the first full-scale quantitative results on the improvement of foundation performance due to ground improvement in a liquefiable deposit.

INTRODUCTION

The results presented in this paper are part of a larger series of tests on the full-scale behavior of laterally loaded piles in liquefied sand. This project, known as the Treasure Island Liquefaction Test (TILT), was a joint venture between Brigham Young University and the University of California, San Diego.

As part of the TILT project, pilot liquefaction studies along with a series of full-scale tests were performed on deep foundations in liquefiable sand. The full-scale tests were conducted on a 4- and 9-pile group of 324-mm diameter pipe piles that were loaded laterally against a 0.6-m and 0.9-m Cast-In-Steel-Shell (CISS) pile, respectively in 15- by 21-m excavations, approximately 1.5-m deep. A high-speed hydraulic actuator was used to apply the lateral loads.

The site at Treasure Island was selected for a number of reasons. Approvals for the use of explosives were relatively easy to obtain for the portion of the island still operated by the U.S. Navy. In addition, the site is only 300 meters away from the Treasure Island Fire Station, the location of a National Geotechnical Experimentation Site (NGES). The NGES status of the site, as well as numerous other geotechnical investigations on the island, provides a wealth of geotechnical data to draw from for the TILT project. Furthermore, there is a known liquefaction hazard at the site due to the high groundwater level and loose nature of the hydraulic fill. In fact, liquefaction was observed across the island during the 1989 Loma Prieta earthquake (Andrus *et al.*, 1998).

The first step in the testing program was to evaluate the ability of controlled blasting to produce a liquefied soil layer suitable for the testing program. Although blast densification has been successfully performed for over 50 years in many different soil and site conditions, site-specific studies are generally recommended (Narin van Court and Mitchell, 1995). A pilot liquefaction study was designed to determine the optimal charge size, pattern, and delays required to liquefy the soil to a depth of about 5 meters and a radius of 5 meters surrounding the foundations.

The first lateral load test of the 4-pile group and 0.6-m CISS pile was conducted before the installation of stone columns and before blasting. Liquefaction was then induced using controlled blasting, and the piles were tested again. Stone columns were then installed, and the tests were repeated for both pre-blast and post-blast behavior. Load-displacement and pore pressure information was gathered for all tests. Similar testing was carried out at an adjacent

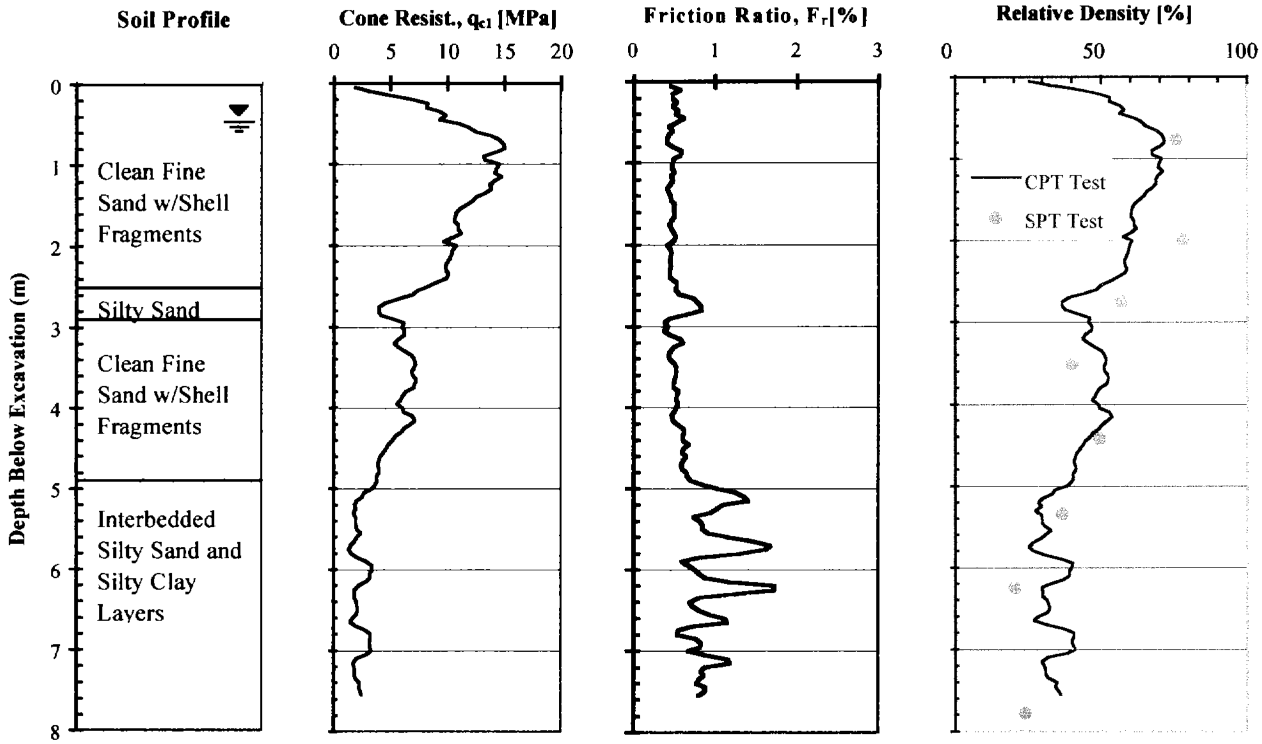

Fig. 1. Soil profile, CPT profile and interpreted relative density at pilot liquefaction test site.

excavation on the 9-pile group and 0.9-m CISS pile without the installation of stone columns.

This paper contains a summary of the pilot liquefaction experimentation and the results of the full-scale tests performed on deep foundations in liquefiable sand, before and after ground improvement. Also contained herein is a comparison between the performance deep foundations in ground improved by stone columns to that of the same and larger (i.e. larger diameter or more piles) foundation systems in unimproved ground, both before and during liquefaction

SITE CHARACTERISTICS

Treasure Island is a 400-acre manmade island immediately northwest of the rock outcrop on Yerba Buena Island in San Francisco Bay. It was constructed in 1936-37 for activities celebrating the construction of the Golden Gate Bridge and the San Francisco-Oakland Bay Bridge. Treasure Island was constructed by hydraulic and clamshell dredging. A perimeter rock dike was built in two to four stages on a bed of coarse sand placed over bay mud. This dike acted as a retaining system for the sand that was pumped or placed inside. The structure is thus essentially an upstream-constructed hydraulic fill. Treasure Island has served as a naval installation since World War II, but was recently decommissioned as part of a nation-wide base closure. Site-specific geotechnical investigations were carried out as part of this study. The generalized soil profile at the pilot liquefaction test area is shown in Fig. 1 after excavation to a depth of 1.2 m. The soil profile typically consists of hydraulically placed fill and native shoal sands to a depth 4.5 to 6 m. The hydraulic fill generally consists of loose uniformly graded fine sands or sandy silts with thin interbeds of clay. The sand is underlain by sandy silts and young bay mud

The water table is typically 1.2 to 1.8 m below the original ground surface and the average horizontal hydraulic conductivity of the sand is 3.5×10^{-3} cm/sec (10ft/day) (Faris, U.S. Navy, Personal communication). The sand typically classifies as SP-SM material according to the Unified Soil Classification system and generally has a D_{50} between 0.2 and 0.3 mm. Both standard penetration (SPT) testing and cone penetration (CPT) testing was performed at the test site. The $(N_1)_{60}$ values in the sand typically ranged from 28 to 7 while the normalized cone resistance, q_{c1} ranged from 14 to 6 MPa as shown in Fig. 1. At the pilot liquefaction site a denser layer appears to exist around a depth of 1 m but this layer was not present at all test sites. The relative density (D_r) was estimated using two independent correlations with $(N_1)_{60}$ and q_{c1} developed by Kulhawy and Mayne (1990) and is plotted as a function of depth in Fig. 1. The estimated D_r was typically between 40 and 60% in the clean sand layers.

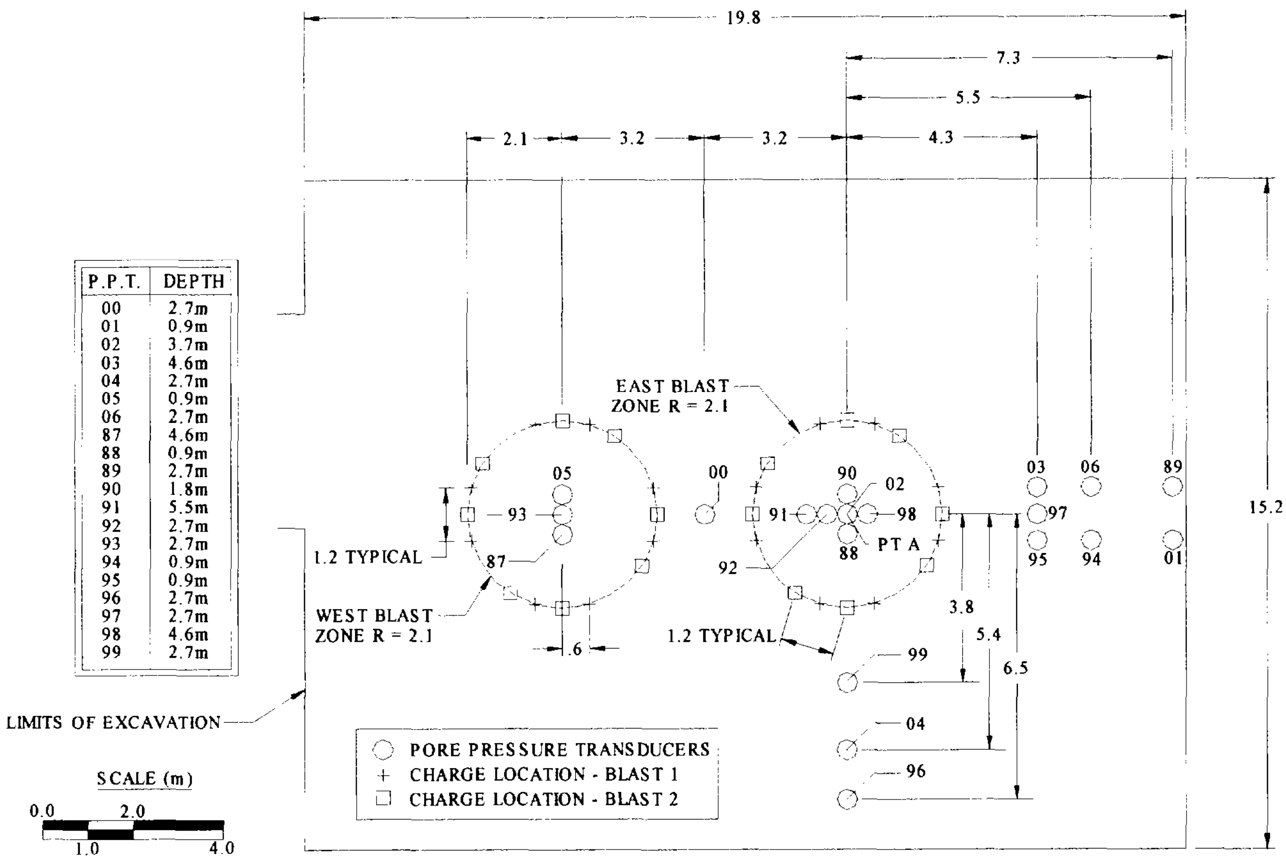

Fig. 2. Layout of blast points and pore pressure transducers (PPT) at pilot liquefaction test site (depth below water table).

PILOT LIQUEFACTION TESTING

In order to determine the blasting procedure to be used at the site, a pilot liquefaction study was performed on an adjacent site. The pilot liquefaction test area was located about 100 m from the test blast areas. In addition, some preliminary test blasts were carried out using a single charge to evaluate transducer viability (Rollins et al, 2000). A plan view of the layout of blast holes and pore pressure transducers at the pilot liquefaction test area is shown in Fig. 2. The objective of the pilot liquefaction test blasts was to simulate the sequence of blasting to be used around the bored pile and driven pile groups in future testing. Prior to testing, an area 15 m x 20 m was excavated to a depth of about 1.5 m so that the water table was about 0.5 m below the excavated surface. This minimized the thickness of non-liquefiable sand at the surface but still allowed drill rigs and CPT trucks to traverse the site.

Two sets of blasts were carried out to determine whether it would be possible to liquefy the site a second time. For each blast, 16 0.5 kg charges were detonated. For safety reasons, two-part explosives were used on the project. When mixed, the nitro methane and ammonium nitrate had the equivalent explosive power of 0.5 kg of TNT (trinitrotoluene) per charge. The charges were placed around the periphery of two circles each having a radius of 2.1 m. Deep foundation elements were placed at the center of these circles in future tests. Pore pressure transducers were positioned to provide an indication of the distribution of pressure as a function of depth and distance from the blast points as shown in Fig. 2. Pore pressure readings from the 20 transducers were obtained at 0.1-second intervals using a laptop computer data acquisition system.

Charges were detonated two at a time with a 250-millisecond delay between explosions. Although the pore pressure transducers indicated that liquefaction occurred within one second of the blast, there was no surface manifestation of liquefaction for a period of 3 to 5 minutes. At this point, sand boils began to form at several of the transducer boreholes as well as at some blast hole locations. Water continued to flow for 10 to 15 minutes following the blast and soil boils reached heights of about 0.3 m. Because liquefaction filled the boreholes above the transducers with sand, the transducers had to be retrieved by jetting following the testing. Three days after the first blast, additional charges were placed as shown in Fig. 2 and a second set of 16 0.5 kg charges were detonated as before.

Vibration Attenuation

Particle velocity was measured at the ground surface during each blast three-component seismographs. A plot of peak particle velocity (PPV) versus square root scaled distance from the blast location for the pilot liquefaction tests is presented in Fig. 3. For the pilot liquefaction testing, the charge mass was taken as 1 kg (the mass of two charges detonated simultaneously) rather than the total 8 kg charge because the delay between detonations caused the velocity to be similar to that from independent blasts.
An upper bound based on blast densification vibration data tabulated by Narin van Court and Mitchell (1995) is also shown in Fig 3 for comparison. In general, the peak velocities fall below the upper bound line; however, a few points fall slightly above the line. The trend line for the Treasure Island data is also shown in Fig 3. The particle velocity attenuates more rapidly with scaled distance than would be expected based on the upper bound relationship developed from previous case histories.

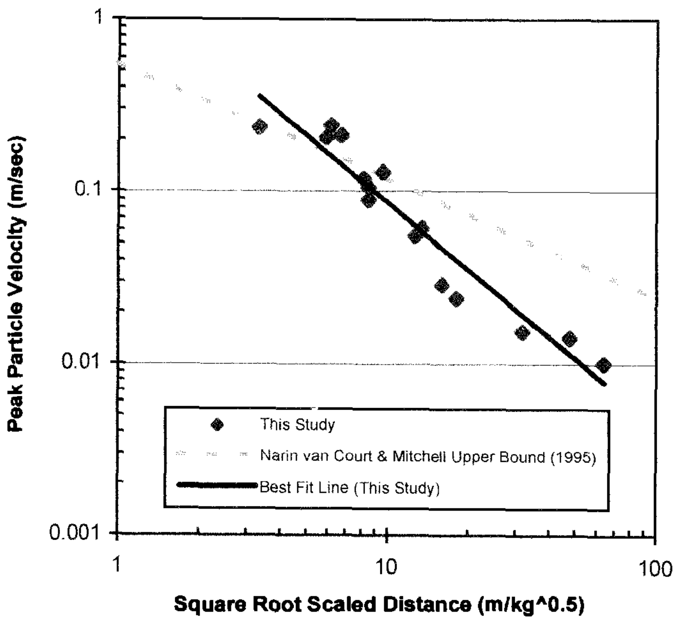

Fig. 3. Measured peak particle velocity as a function of scaled distance relative to upper bound limit from previous investigations.

Excess Pore Pressure

Piezoresistive transducers were used to measure the pore water pressure generation and dissipation. These transducers had the ability to survive a blast pressure of up to 41.4 MPa (6000 psi), yet they could also measure pressure with an accuracy of 0.7 kPa (0.1 psi). The transducers were placed within plastic cone tips with 8 ports open to the groundwater and pushed to the desired depth after saturation. The measured residual excess pore pressure (Δu) at each transducer depth was divided by the effective vertical stress (σ'_o) at that depth to define the excess pore pressure ratio (R_u). An R_u of 1.0 indicates liquefaction.

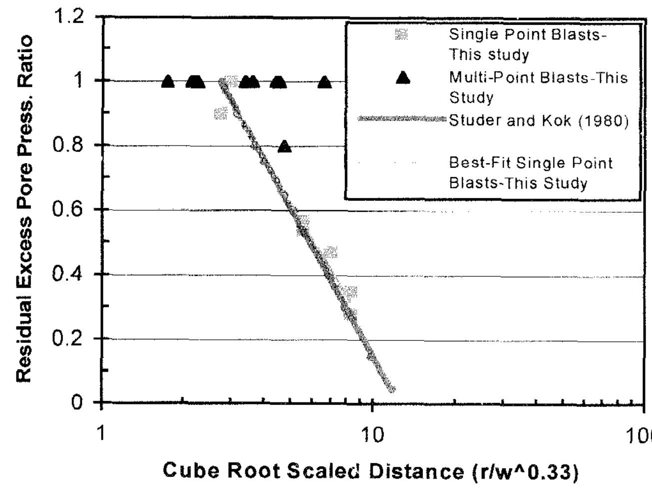

Fig. 4. Measured excess pore pressure ratio vs scaled distances for single and multiple blasts relative to predicted ratios using the Studer and Kok (1980) relationship.

A plot of measured peak R_u as a function of scaled distance from the blast point is shown in Fig. 4. A best-fit line for the single point blast data from this study is also shown in Fig. 4 along with a similar line developed by Studer and Kok (1980) for much larger charge weights. The agreement between the two lines is very good when single blast charges were employed. However, when two or more charges were employed, the measured R_u values were significantly higher than expected at larger scaled distances. For example, the Studer and Kok relationship predicts a R_u of 1.0 for scaled distances less than 2.8, but R_u values of 1.0 were achieved for scaled distances as high as 6.6. These results suggest that multiple blast points may be more effective in generating excess pore pressures than a single blast point with the same charge weight. This may result from small variations in arrival times that could lead to multiple stress pulses or longer pulse duration.

For the first pilot liquefaction blast, plots of R_u versus time are shown for one vertical and one horizontal transducer array in Figs. 5 and 6. Transducers for the vertical array were located in the center of a ring of eight blast charges shown as Point A in Fig. 2. In subsequent tests at other sites, a pile foundation was located in this position. Transducers were spaced at about 0.9 m intervals below the water table. The results from the vertical array in Fig. 5 indicate that a peak R_u between 0.9 and 1.0 was produced at each of the transducers with the exception of that at 1.2 m depth. At the 1.2 m depth, the R_u peaked at 0.76 but them rapidly dropped to around 0.40. The lower R_u at this level could be because the sand is densest at this level (see Fig. 1) or to the lower confining pressure near the surface. For all other transducer depths, the R_u value remained above 0.8 for at least 4 minutes and above 0.6 for at least 8 minutes after the blast.

The transducer at 5.9 m depth maintained a R_u above 0.94 for 6 minutes and remained higher than all the other transducers thereafter. This indicates that the transducer was likely within

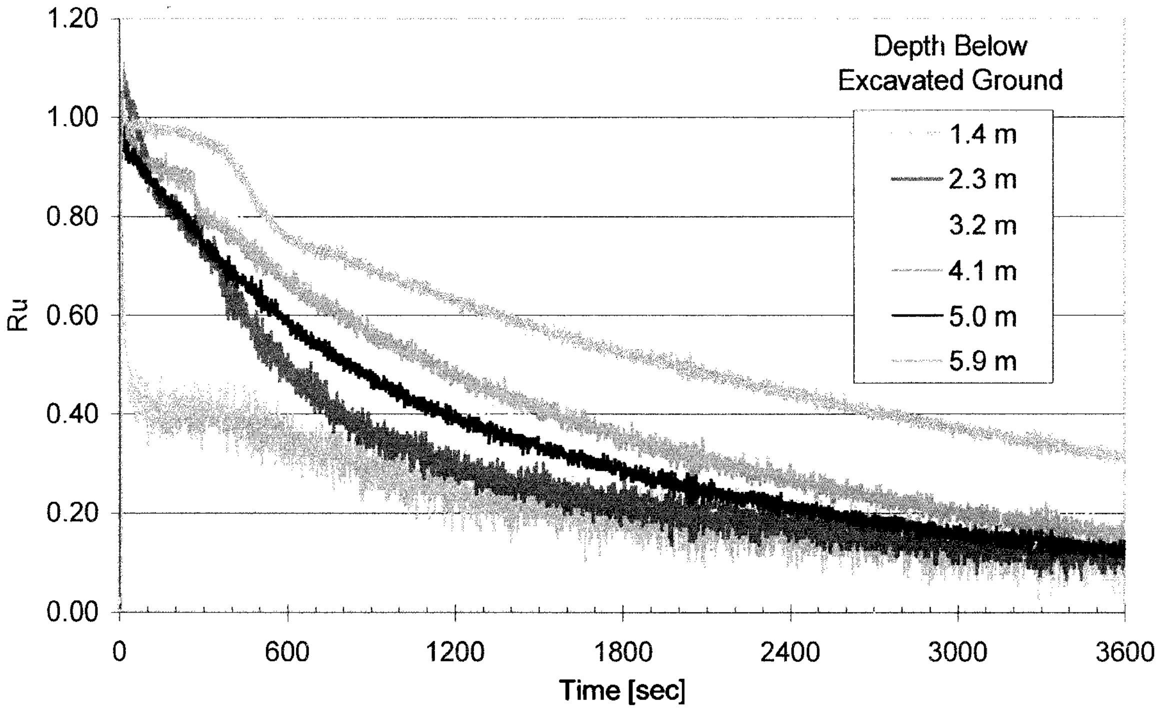

Fig. 5. Measured excess pore pressure ratio vs time for a vertical array at the center of the east blast zone (Point A.)

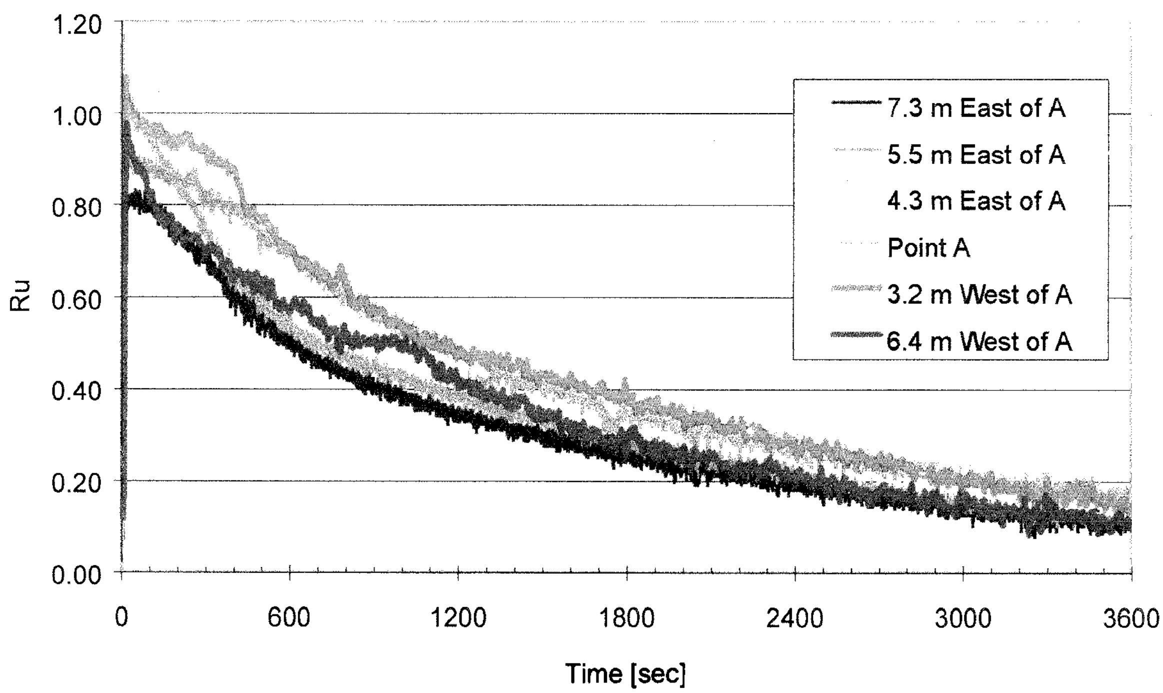

Fig. 6. Measured excess pore pressure ratio vs time for a horizontal array along an east-west axis through the blast zone at a depth of 3.2 m.

one of the more fine-grained layers located around that depth or was bounded by fine-grained layers. One hour after the blast, excess pore pressure ratios in the sand were typically down to between 0.1 and 0.2. Transducers for the horizontal array were placed at 3.2 m below the ground surface (2.7 m below the water table) at various distances east and west of point A as shown in Fig. 2.

The results from the horizontal array are shown in Fig. 6. These results and those from another horizontal array perpendicular to that shown indicate that liquefaction (R_u = 1.0) extended to a distance at least 6.4 m from Point A (4.3 m from the blast points). The transducer at 7.3 m from point A (5.2 m from the blast points) still recorded an R_u of 0.8. In the zone where liquefaction occurred, the R_u typically stayed above 0.8 for at least 4 minutes and above 0.6 for at least 8 minutes.

Results from the second blast at the pilot liquefaction site were very similar to those for the first blast and confirmed that liquefaction could be induced at least twice if the time interval between blasts was less than a few days. In most cases, the pore pressure dissipation rate was only slightly faster for the second blast.

Settlement

Ground surface settlement was monitored using lines of survey stakes spaced at approximately 0.6 m intervals through the blast area. Settlement was calculated as the change in the stake elevation after the blast. Maximum ground surface settlements ranged from 25 mm for the single blast charges to almost 100 mm for the 16 blast points. About 85% of the settlement occurred within about 30 minutes of the detonation. A plot of the settlement in the east-west direction through the pilot liquefaction test area is shown in Fig. 7 for both blasts.

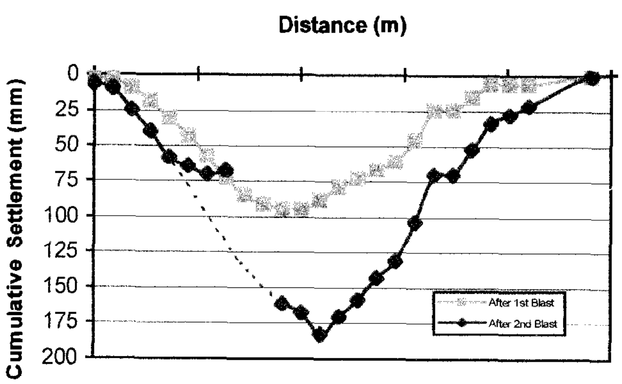

Fig. 7. Cumulative settlement along an east-west section through the blast zone for the two blasts at the pilot liquefaction test site.

The maximum settlement for the second blast was about the same as that for the first blast. During the second blast, a 3-m square area (between 6 and 9 m markers in Fig. 7) was excavated down to the water table for observation purposes. Following the blast, this excavation filled up with a large sand boil making it impossible to locate some survey stakes in this area. In addition, the reduction in overburden pressure allowed the ground to heave following the blasting. The dashed line in Fig. 7 represents our approximation of the settlement that would have occurred had the excavation not been made based on the behavior of the soil within the other ring of blasts. The maximum settlement produced by each blast amounted to about 2.5% of the thickness of the liquefied zone.

LATERAL PILE LOAD TESTING

Test Set-up

Figure 8 presents a plan view of the test set-up. The 4-and 9-pile groups consisted of 342-mm O.D. steel pipe piles with a 10-mm wall thickness, connected by a load frame that allowed for the free rotation of the pile heads while maintaining the same lateral displacement for all four piles in the group. The CISS piles were 0.6 m and 0.9 m in diameter, with nominal wall thickness of 13 mm and 11 mm, respectively. The hydraulic actuator used had a double swivel connection to both the CISS pile and pile group load frame thus allowing free rotation at the CISS load stub. All of the piles were driven to depths between 12 and 14 meters below the excavated surface. For the CISS pile, the steel shell was driven into place, drilled out, and then filled with steel reinforcement and concrete. No water was observed in the steel shell prior to placement of the concrete.

The sites were excavated to a depth of approximately 1.5 meters. The objective of this was to conduct the lateral load test primarily in the loose saturated sand by removing the medium dense sand and lowering the excavated surface closer to the ground water table. Prior to excavation, SPTs and CPTs were performed. Following completion of the pre-treatment load tests, the 4-pile group/0.6-m CISS pile site was backfilled to the original elevation, and the stone columns were installed. A second set of CPTs was then performed. The site was then re-excavated to the same level as before, and the post-treatment series of tests were performed.

A 1500-kN high-speed hydraulic actuator was used to laterally load the piles, with the loading point approximately 0.8 meters above the excavated surface. The speed of the actuator was approximately 10 mm/second. For each case, the actuator was connected between the load frame of the pile group and the load stub of the CISS pile, such that load-displacement information for the pile group and the CISS pile was obtained simultaneously. The applied load was measured in the actuator using an array of three 500-kN load cells. Relative displacement between the pile group and CISS pile was measured in the actuator as well, using a linear variable displacement transducer (LVDT). In addition, absolute displacement measurements of the piles were obtained

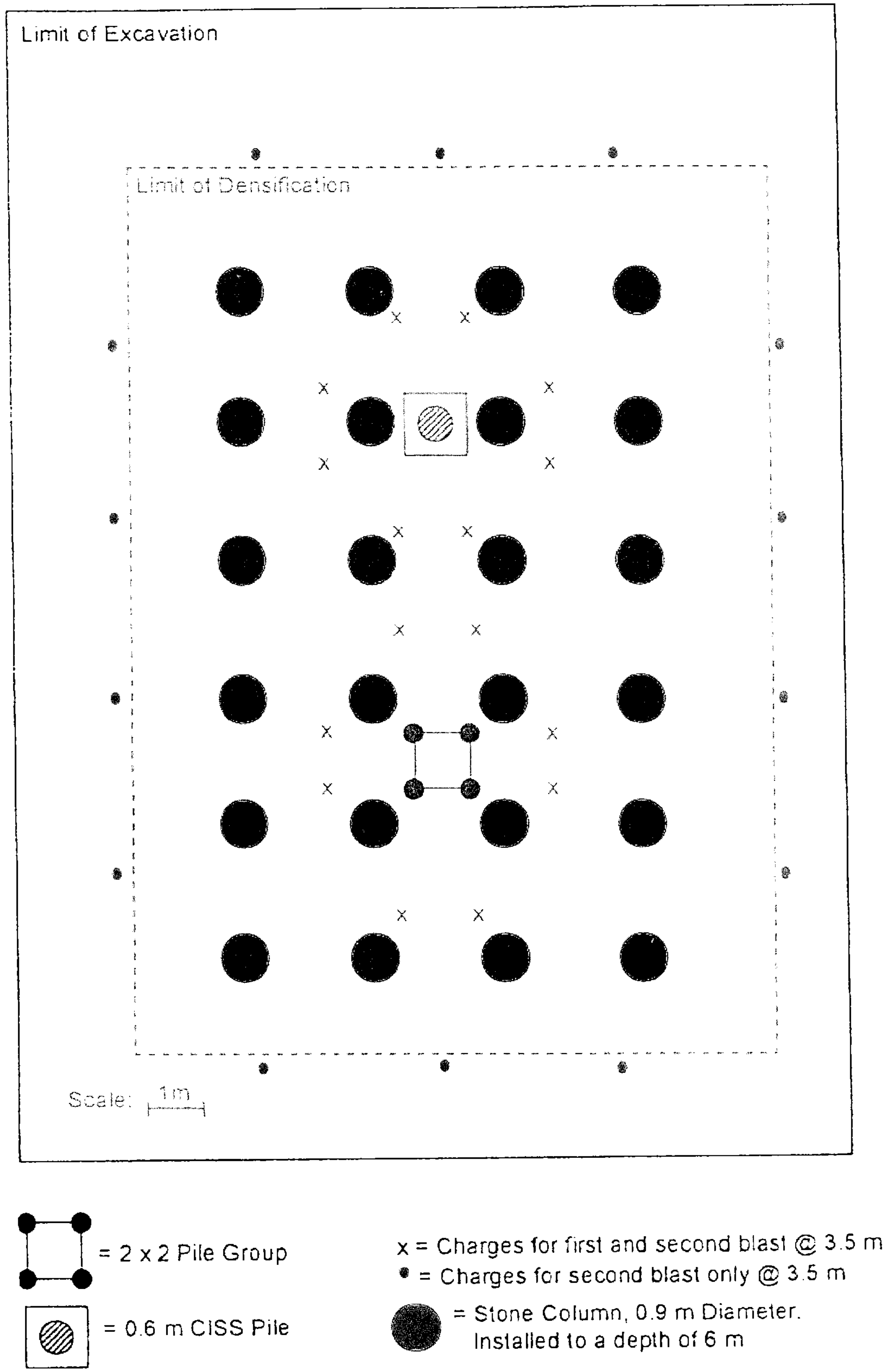

Fig. 8. Plan view of 4-pile group showing layout of stone columns around piles.

using string activated linear potentiometers fixed to a reference post outside of the excavation. These were attached to the foundations in such a way as to allow for monitoring of displacement, tilt, and rotation. In order to monitor the effectiveness of the controlled blasting, pore pressures were measured using pore pressure transducers arranged at various depths immediately adjacent to the piles.

STONE COLUMN INSTALLATION

Installation of stone columns, also referred to as vibro-replacement and vibro-displacement, is a ground improvement technique often used to mitigate liquefaction hazards in saturated loose granular soils. Stone columns can improve the performance of these deposits in four main ways. First, the installation of the stone columns densifies the deposit by vibration and replacement. Second, this technique increases the lateral stresses in the surrounding soil. Third, stone columns provide reinforcement, as the stone columns are stiffer, stronger and denser than the surrounding soils. Finally, stone columns provide drainage, reducing the potential for build-up of excess pore water pressures. Though the effect of each of these factors will vary between deposits, combined they make stone columns an efficient and popular liquefaction hazard mitigation technique (Kramer, 1996).

There is considerable qualitative data available showing that stone column installation is an effective means of ground improvement for mitigating liquefaction hazards during earthquakes (Mitchell *et al.*, 1995; Priebe, 1990). While qualitative information from past earthquakes is valuable in confirming that stone columns can be an effective means of ground improvement, quantitative data is also needed for design purposes.

After the first series of tests, the site was backfilled and stone columns were installed around the piles. Twenty-four 0.9-m diameter stone columns were installed in a 4 by 6 grid around the piles, with a spacing of approximately 2.4 meters on center, as shown in Figure 2. The stone columns extended through the surficial sand layer at the site, to depths of approximately 5.5 to 6.0 meters. After the installation, the site was re-excavated to the same depth as for the pre-treatment tests.

The stone columns used were installed using the dry bottom feed method. "Dry" in this context refers to the fact that the vibratory probe was driven into the ground using compressed air instead of water. The term "bottom feed" is in reference to the way gravel is fed through the tip of the probe rather than being placed into the soil from the ground surface. Compressed air, vibration, and the weight of the probe itself drove the probe into the ground. Once the probe reached full depth, it was lifted up and the hole was backfilled with gravel from the probe tip. The probe was approximately 0.5 m in diameter, and required multiple passes to create a column 0.9 m in diameter. The probe was raised in 0.9-m lifts, and gravel was placed into the soil. The probe was then re-lowered into the gravel that had just been placed, forcing it outward and further densifying the surrounding soil. Lifting the probe multiple times inserts more gravel into the column. To determine the number of passes required for complete site treatment, the operator monitored the amperage of the vibrating probe. As the soil was dandified, the probe required more power to maintain its vibration. Once a set level of increase in amperage had been reached, the operator proceeded to the next 0.9-m lift.

Loading Sequence

All foundations were loaded prior to blasting in order to obtain baseline information in the non-liquefied state. In the case of the 4-pile group/0.6-m CISS pile test, a complete series of tests were conducted prior to installation of stone columns. For the pre-liquefaction tests, the piles were pulled towards each other until one pile was displaced 38 mm. The load was reduced until one of the piles returned to its original position. After this test, the charges were set off. Ten seconds after the blast, the piles were loaded again, cycled under displacement control to 75 mm, 150 mm, and 225 mm of absolute displacement, then cycled at 225 mm of displacement nine times. For these tests, the load level was approximately 1 meter above the excavated surface.

The procedure for the post-treatment tests was essentially the same as for the pre-treatment testing. After the first tests were completed, stone columns were installed and the post treatment testing took place. For the post-treatment testing, the same loading sequence as the pre-treatment tests was attempted. However, the capacity of individual load cells within the pile group was exceeded before the piles had reached 150 mm of absolute displacement, so the piles were cycled under load control instead of displacement control.

RESULTS

The improvement to the upper sand layer is apparent from review of Fig. 9, which shows the CPT tip resistance values (q_c) for the upper sand layer, both before and after treatment with stone columns. Excluding the top 1.5 meters that was excavated prior to testing, a substantial increase in the tip resistance can be seen throughout the sand layer. Prior to installation of the stone columns, the average tip resistance in the upper sand was approximately 4 MPa. After installation of the stone columns, the average tip resistance in the upper sand ranged between 10 and 50 MPa, and below a depth of 2 meters (i.e. 0.5 meters below the excavated surface) the average is well above 20 MPa. This amount of improvement can be expected from the installation of stone columns (e.g. Priebe, 1991; Soydemir, 1997). Clearly, this substantial increase in tip resistance corresponds to a substantial decrease in the susceptibility of the upper sand to liquefaction.

This increased resistance to liquefaction was observed in comparison of the pre- and post-treatment excess pore pressures as shown in Figs. 10 and 11. Testing found an immediate increase in pore pressure at all depths at the time of the blast and these were maintained generally in excess of R_u equal to 80

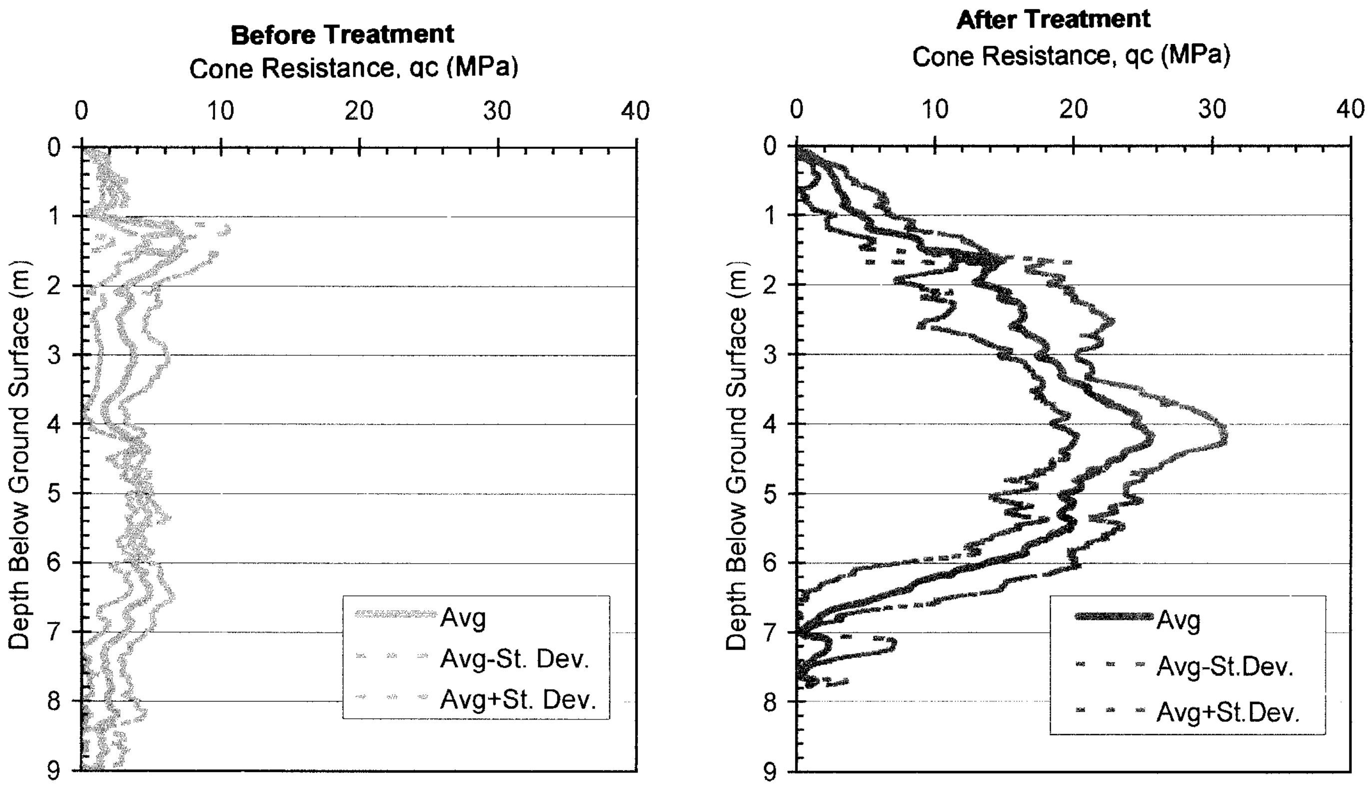

Fig. 9. Cone penetration test results before and after stone column treatment

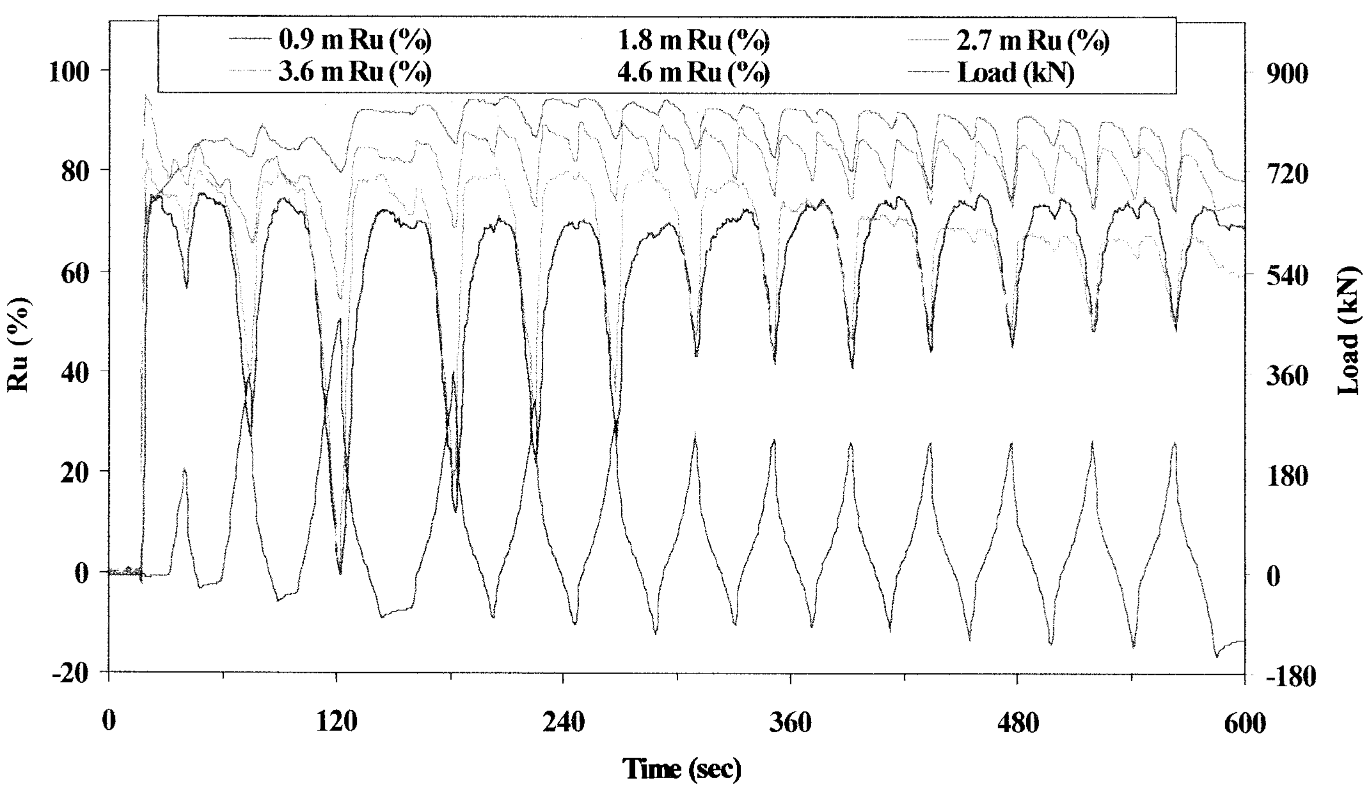

Fig. 11. Excess pore pressure ratio versus time near 0.6 m CISS for lateral load test prior to stone column treatment.

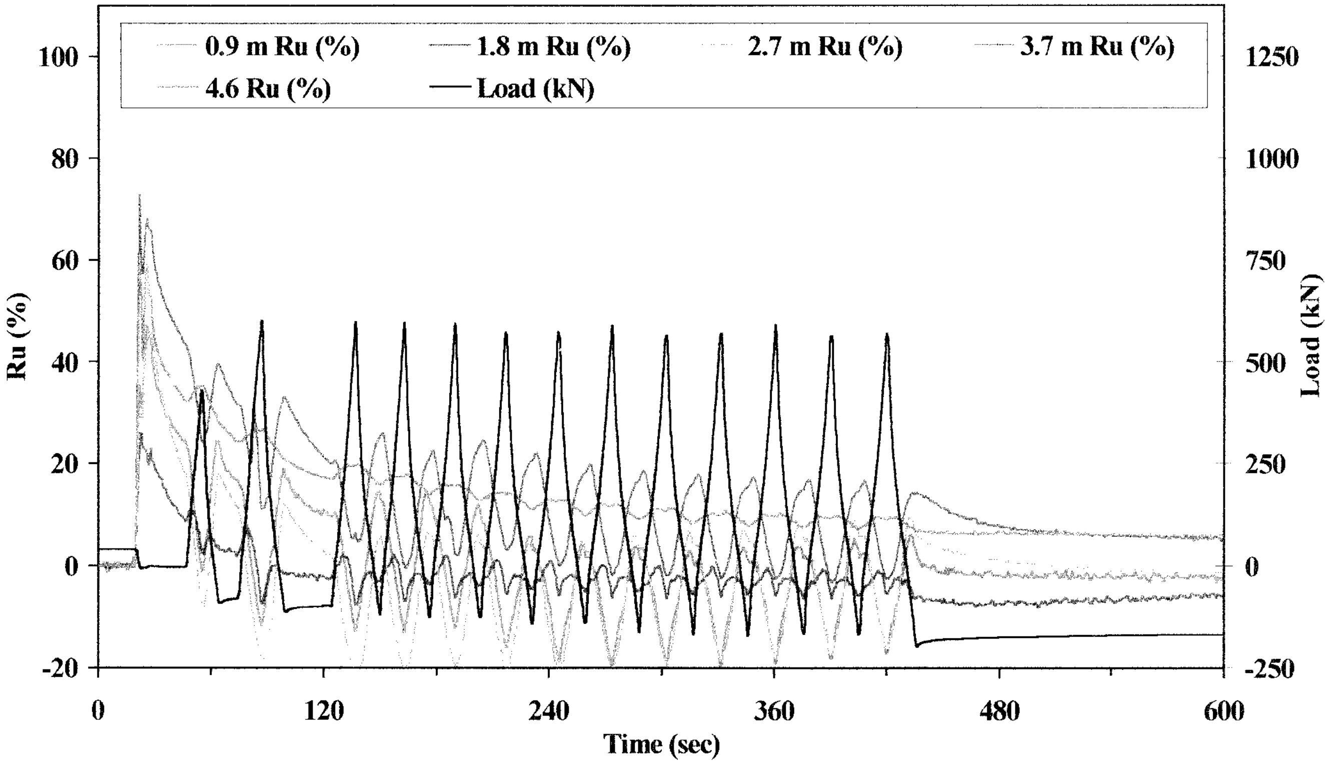

Fig. 12. Excess pore pressure ratio versus time near 0.6 m CISS for lateral load test following stone column treatment.

percent for depths greater than 2.7 meters for the first 10 minutes of loading. Though R_u was not found to be 100 percent throughout the profile, observations confirmed that the site was essentially liquefied. These observations included numerous sand boils, water flowing freely from the ground, and considerable surface settlement. Results found that the pore pressure response for the post-treatment tests in sharp contrast to those recorded before installation of the stone columns. However, a sudden increase in pore pressure was apparent at the beginning of the record following the blast, the increase was much less than in the pre-treatment case ($R_u = 60\%$).

Furthermore, rapid dissipation of excess pore pressures was observed. For example, at the end of 10 minutes, pore pressures were nearly hydrostatic, and in fact were slightly negative in some cases. Observations were consistent with these measurements, in that no visible signs of liquefaction were apparent. No sand boils, surface settlement, or flowing water was observed, and there was actually significant gapping around the piles during the cyclic loading.

Perhaps the most dramatic indicator of soil improvement because of stone column installation, and of direct importance to this study, is the load-displacement curves. Reviewing first the pre-treatment plots for both the 4-pile group and the 0.6-m CISS pile, shown respectively in Figures 13 and 14, the pre-blast secant stiffness from the plots is approximately 7.5 kN/mm.

These values are immediately reduced by over 60 percent due to the increased pore pressure from the blast (Table 1). As the number of cycles increase and the soil structure is broken down, the secant stiffness is further reduced a total of nearly 70 percent for the 4-pile group and 80 percent for the CISS pile. Higher excess pore pressures observed surrounding the CISS pile might explain the slightly lower stiffness values.

Table 1. Summary of secant stiffness values for various foundation systems before and after blast induced liquefaction.

Foundation System	Pre-Blast Secant Stiffness (kN/mm)	Post-Blast Secant Stiffness (kN/mm)
4 Pile Group Untreated	7.5	1.8
4 Pile Group Treated	9.3	7.0
9 Pile Group Untreated	14.3	3.7
0.6m CISS Untreated	7.5	1.5
0.6 m CISS Treated	10.8	7.0
0.9 m CISS Untreated	20.0	3.8

Similar results are observed for the non-treated soil for the 9-pile group and 0.9-m CISS pile shown in Fig 15 and 16, respectively. The pre-blast secant stiffness is approximately 14.3 kN/mm for the 9-pile group and 20 kN/mm for the 0.9-m CISS pile. After several cycles of post-blast loading, both of these values reduce to approximately 3.5 kN/mm, a decrease of over 75 to 80 percent from the pre-blast case.

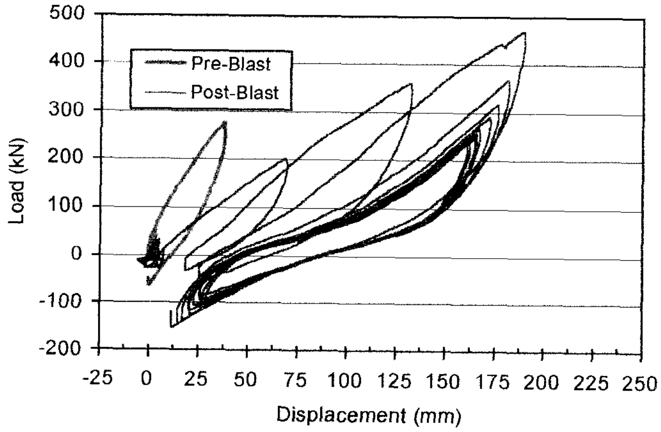

Fig. 13. Load-displacement curve for 4-pile group prior to stone column treatment.

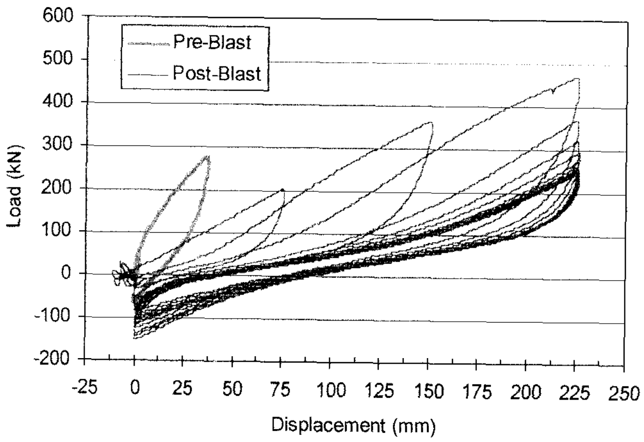

Fig. 14. Load-displacement curve for 0.6-m CISS prior to stone column treatment.

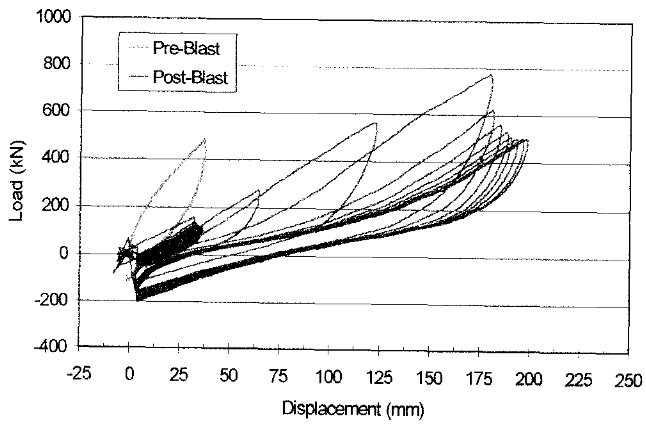

Fig. 15. Load-displacement curve for 9-pile group prior to stone column treatment.

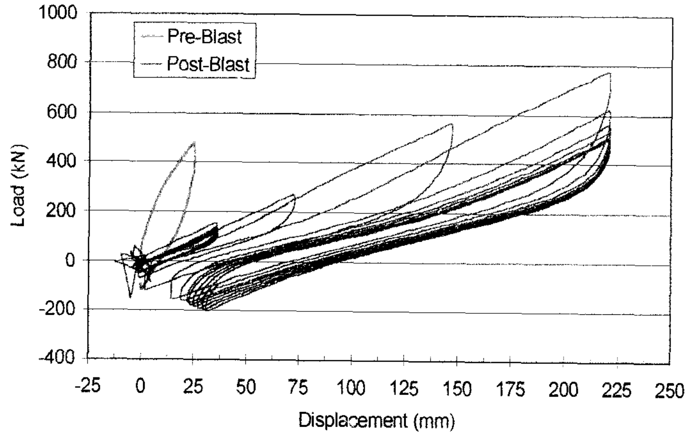

Fig. 16. Load-displacement curve for 0.9-m CISS prior to stone column treatment.

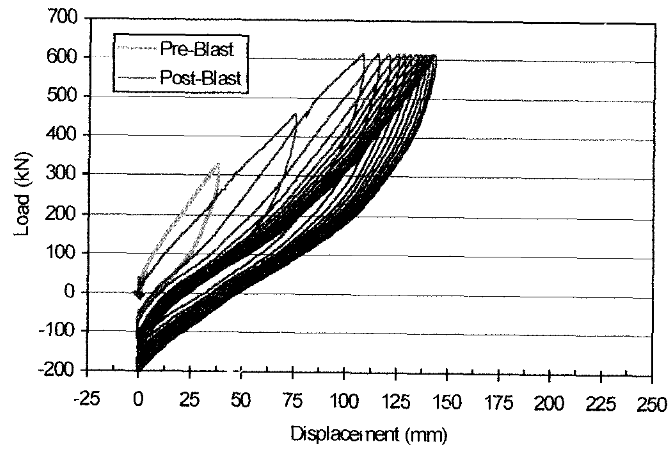

Fig. 17. Load-displacement curve for 4-pile group after stone column treatment.

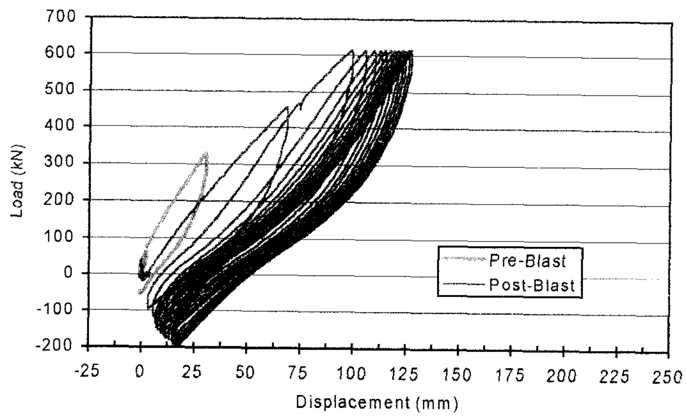

Fig. 18. Load-displacement curve for 0.6-m CISS after stone column treatment.

In sharp contrast are the post-treatment load-displacement curves for the 4-pile group and the 0.6-m CISS pile, shown in Fig 17and *18*, respectively. The initial secant stiffness prior to blasting is approximately 9.3 kN/mm for the 4-pile group and 10.8 kN/mm for the CISS pile. This is an increase of 25 to 45 percent over the pre-treatment, pre-blast test results. For both foundation types, however, the post-blast secant stiffness is approximately 7 kN/mm. This is only a 25 to 35 percent decrease from the pre-blast values.

It is of interest to compare the test results in the improved ground to both cases of non-improved ground. In a direct comparison of the pre- and post-treatment tests for the 4-pile group, it can be seen that the pre-blast secant stiffness is increased by 25 percent because of ground improvement. Similarly, a 45 percent increase is observed for the 0.6-m CISS pile. A much more dramatic improvement is observed post-blast, where the improved ground yields a secant stiffness 2.9 times greater for the 4-pile group and 3.6 times greater for the 0.6-m CISS pile.

A comparison of the test results of the post-treatment 4-pile group and 0.6-m CISS pile to those for the non-treated 9-pile group and 0.9-m CISS pile gives an indication of the effectiveness of increasing the pile size or number of piles in lieu of ground treatment. This comparison shows a more substantial foundation may be worthwhile in the non-liquefied case. The secant stiffness for the non-treated 9-pile group is over 50 percent higher than the treated 4-pile group. The increase is over 80 percent when comparing the non-treated 0.9-m CISS pile to the treated 0.6-m CISS pile. However, the post-blast comparison is more favorable to the improved ground case. When comparing the treated 4-pile group to the non-treated 9-pile group and the treated 0.6-m CISS pile to the 0.9-m CISS pile, the treated ground in both cases yields secant stiffness twice that of the unimproved case.

It is understood that many factors influence the comparison between a more substantial foundation and ground improvement. These factors are not limited to the details of the foundation system, the soil profile, and construction considerations. However, in this study of full-scale test results in liquefied ground, it was shown that more than doubling the number of piles (from 4 to 9) or increasing the shaft diameter by 50 percent (from 0.6 to 0.9 m) does very little for foundation performance during liquefaction in comparison to ground improvement by stone columns.

CONCLUSIONS

This paper presents the results of full-scale lateral load tests in liquefiable sands before and after ground improvement with stone columns. In each case, controlled blasting was used to elevate pore pressures in an attempt to liquefy the soil surrounding the deep foundations. Based on these results, several conclusions can be made regarding blast induced liquefaction and the effectiveness of stone columns in improving the performance of the foundation system under lateral loading in liquefiable soils.

- Controlled blasting techniques can be successfully used to induce liquefaction in a well-defined volume of soil in the field for full-scale experimentation. In this case, excess pore pressures ratios (R_u) of 90 to 100% were typically generated within a depth range of 1 to 6 m and over a 13 m x 19 m surface area. R_u values greater than 0.8 were typically maintained for 4 to 10 minutes.

- The excess pore pressures generated by the blasts were predicted with reasonable accuracy using the Studer and Kok (1980) relationship when single blast charges were used. However, for multiple blast charges, measured excess pressures were significantly higher than would have been predicted for a single blast with the same charge weight.

- Peak particle velocity attenuated rapidly and was generally below the upper-bound limit based on data summarized by Narin van Court and Mitchell (1995). PPV attenuation correlated reasonably well with the square root scaled distance.

- Settlement ranged from 25 mm using a 0.5 kg charge at one point to 100 mm using 0.5 kg charges at 16 points. Settlement was typically about 2.5% of the liquefied thickness and about 85% of the settlement occurred within 30 minutes after the blast.

- Following liquefaction, the lateral foundation stiffness was typically reduced by 70 to 80% in comparison with the pre-liquefaction value.

- As has been observed in previous studies, the installation of stone columns significantly increased the density of the ground surrounding the foundations as indicated by the cone penetrometer test.

- The installation of stone columns significantly increased the stiffness of an identical foundation system before and after blasting. This increase was 2.9 to 3.6 times that of the system in the liquefied soil.

- Increasing the number of piles in a group from 4 to 9 or increasing the diameter of CISS piles from 0.6 to 0.9 m more than compensated for the ground improvement in the non-liquefied case.

- Increasing the number of piles in a group from 4 to 9 or increasing the diameter of CISS piles from 0.6 to 0.9 m resulted in much lower foundation stiffness than was achieved with stone column treatment for the post-blast (liquefied) case.

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