CONTROLLED BLASTING TO SIMULATE LIQUEFACTION FOR FULL-SCALE LATERAL LOAD TESTING

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SUMMARY

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. 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. This paper describes a pilot liquefaction test program that was carried out to determine appropriate charge weight, charge spacing and instrumentation. 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 at least 4 minutes after blasting. The test results indicate that excess pore pressure ratios produced by blasting can be 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 was measured at the ground surface during each blast using seismographs. The particle velocity attenuated more rapidly with scaled distance than would be expected based on the upper bound relationship developed from previous case histories. 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

INTRODUCTION

The lateral load capacity of deep foundations is critically important in the design of bridge structures in seismically active regions. Although fairly reliable methods have been developed for predicting the lateral capacity of piles in non-liquefied soils, there is very little information to guide engineers in the design of piles that are surrounded by liquefiable soils. Certainly, ongoing centrifuge studies using small-scale models (e.g. Wilson et al., 1996; Dobry et al., 1996) are providing valuable insight on soil-pile interaction in liquefied soil. However, full-scale tests are necessary to verify/calibrate these models and provide ground truth information. 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 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. The tests were carried out using a high-speed hydraulic loading system after the sand surrounding the piles was liquefied using blasting techniques.

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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. In order to accomplish this, a series of controlled blasts were carried out at three separate locations at the Treasure Island NGES. The site, consisting of loose saturated sand placed by hydraulic filling, was known to have liquefied in the 1989 Loma Prieta Earthquake.

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.

Since Treasure Island is a National Geotechnical Experimentation Site, a substantial amount of geotechnical data is available in the vicinity. In addition, site-specific geotechnical investigations were carried out as part of this study. 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 fine sands or sandy silts with thin interbeds of lean clay. The sand is underlain by sandy silts and young Bay mud. The soil profile at the pilot liquefaction test area is shown in Fig. 1 after excavation to a depth of 1.2 m.

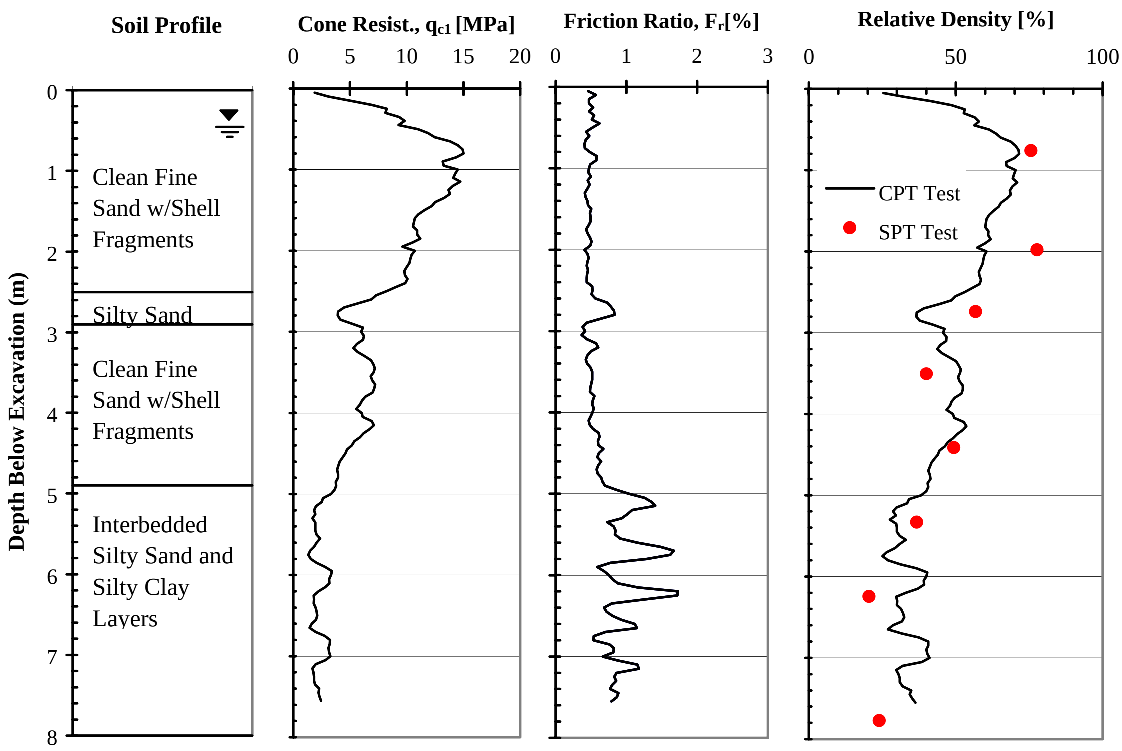

Fig. 1 Soil Profile, CPT profile and Interpreted Relative Density at Pilot Liquefaction Test Site.

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 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. A denser layer appears to exist around a depth of 1 m where the higher values were recorded. The relative density (D_r) was estimated using two independent correlations with $(N_1)_{60}$ and q_{c1} developed by 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.

TEST BLAST LAYOUT AND INSTRUMENTATION

Initially, a series of tests were performed using a single charge with a mass between 0.25 and 0.75 kg. These tests were performed to define the relationship between induced excess pore pressure ratio and scaled distance. In addition, the tests evaluated the capability of various pore pressure transducers to survive the transient blast pressures generated by the blasting yet still provide enough resolution to monitor changes as small as 0.7 kPa. Subsequently, test blasts were carried out with two charges and finally with 16 charges. The effects of the blasts were also measured by settlement monuments and portable seismographs

Test Blast Area 1

Prior to testing, a 7.3 m x 15.2 m area was excavated to depth of 0.6 m. Ammonium nitrate charges were placed at a depth of 3.66 m below the excavated surface (3.05 m below the water table) and the borehole was back-filled with pea gravel. Three pore pressure transducers were placed at the same depth but at radial distances of 2.13, 4.27 and 6.40 m from the blast point. The pore pressure transducers were installed by first drilling a borehole to a depth about 300 mm above the desired installation level. The transducers, which were mounted inside a cylinder with a conical tip, were then pushed the final 300 mm into the undisturbed sand. The transducers and conical tips were saturated with de-aired water prior to placement in the ground. Pore pressure readings were obtained at 0.1 sec. intervals using a laptop computer based data acquisition system.

Vibrating wire transducers with a pressure limit of 1.4 Mpa (200 psi) were used in the first test blast area. Although the transient blast pressures were expected to exceed the pressure limit, previous experience with the transducer suggested that much higher pressures could be tolerated by orienting the three portholes leading into the transducer away from the blast. (Hachey et al., 1994). Since the vibrating wire transducer could be purchased at one-third the cost of a more robust piezoresistive transducer with a rated overpressure limit of 41.4 MPa (6000 psi), considerable cost savings could be realized if the vibrating wire transducer proved successful. Unfortunately, the first blast destroyed a transducer at a distance of 2.13 m from the 0.5 kg charge and the third blast destroyed a transducer at a distance of 2.13 m from a 0.25 kg charge. These results disqualified the vibrating wire sensor from further use in the testing program.

Test Blast Area 2

Test area 2 was located adjacent to test area 1. The blast holes and transducer locations were identical to those in test area 1, except that that in one case, two separate blast charges were placed rather than one single charge. In the second blast area, 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 as described previously for test area 1. During the testing program, these transducers survived blasts with 0.5 and 1.0 kg of explosives at a distance of 2.13 m and were successfully retrieved for used in subsequent testing.

Pilot Liquefaction Test Area

The pilot liquefaction test area was located about 100 m from the test blast areas. 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 15.2 m x 19.8 m was excavated to a depth of 1.2 m so that the water table was about 0.46 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, a total of 16 0.5 kg charges were detonated. The charges were placed around the periphery of two circles each having a radius of 2.1 m. Deep foundation elements were to be 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 1-second intervals using a laptop computer data acquisition system

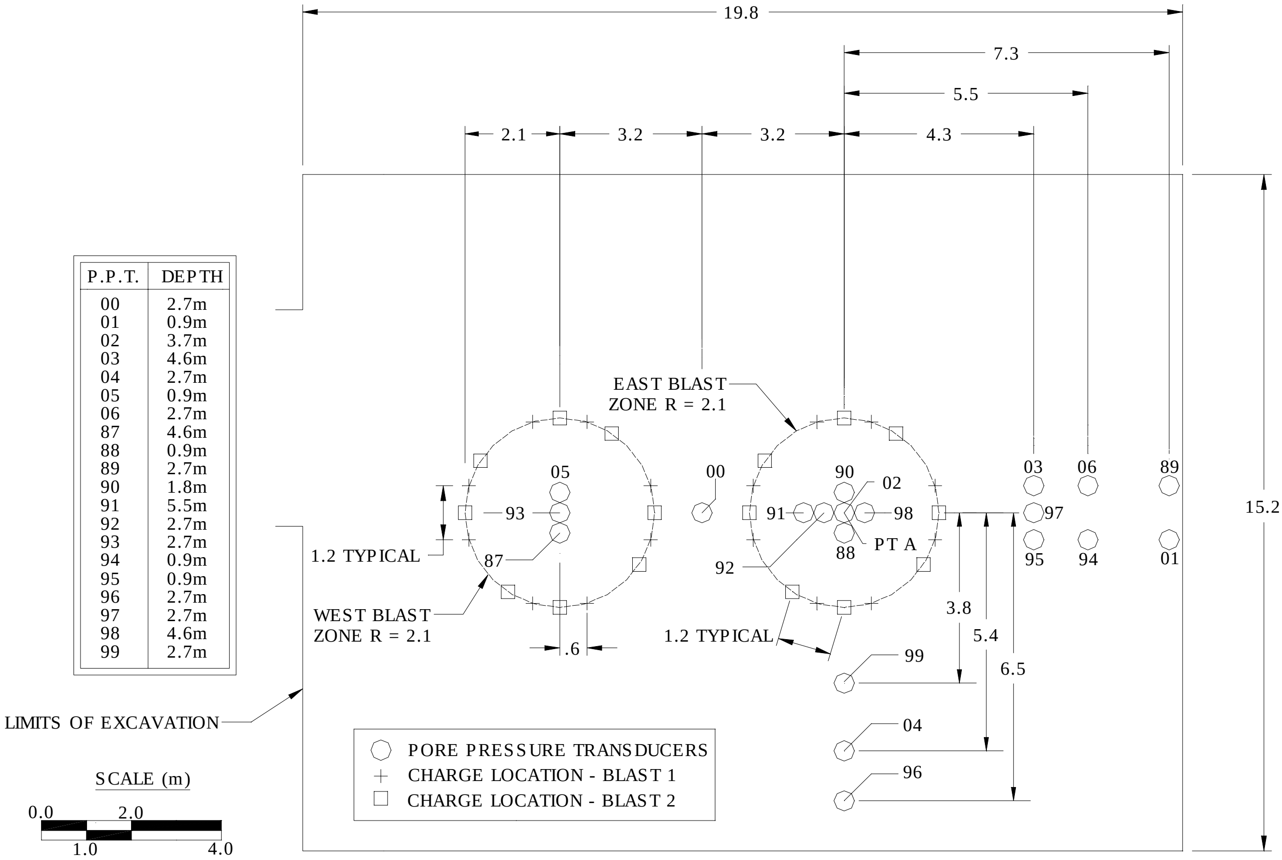

Fig. 2. Layout of Blast Points and Pore Pressure Transducers (PPT) at Pilot Liquefaction Test Site (Depth Below Water Table).

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.

TEST BLAST RESULTS

Vibration Attenuation

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 test blast and pilot liquefaction data 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. This may result from the fact that the charge weights are small and the measurements were relatively close to the blast points, therefore, the PPV would likely be produced by body waves rather than surface wave. Body waves would be expected to attenuate more rapidly than surface waves.

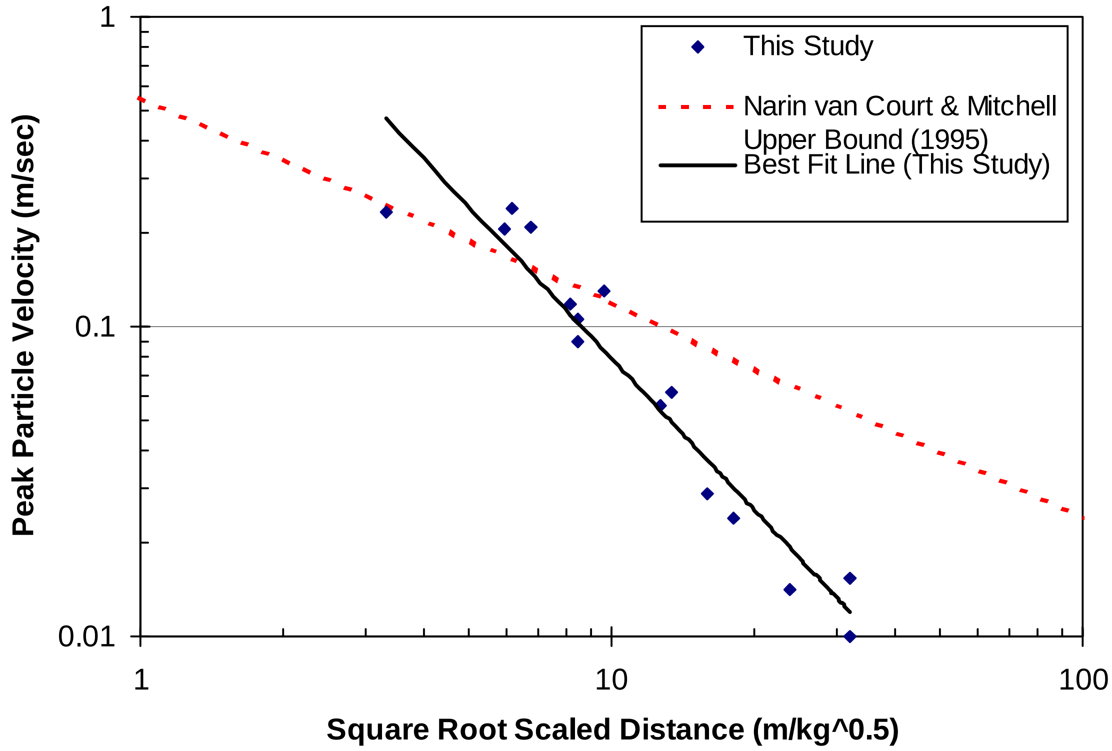

Fig. 3 Measured Peak Particle Velocity as a Function of Scaled Distance Relative to Upper Bound Limit from Previous Investigations.

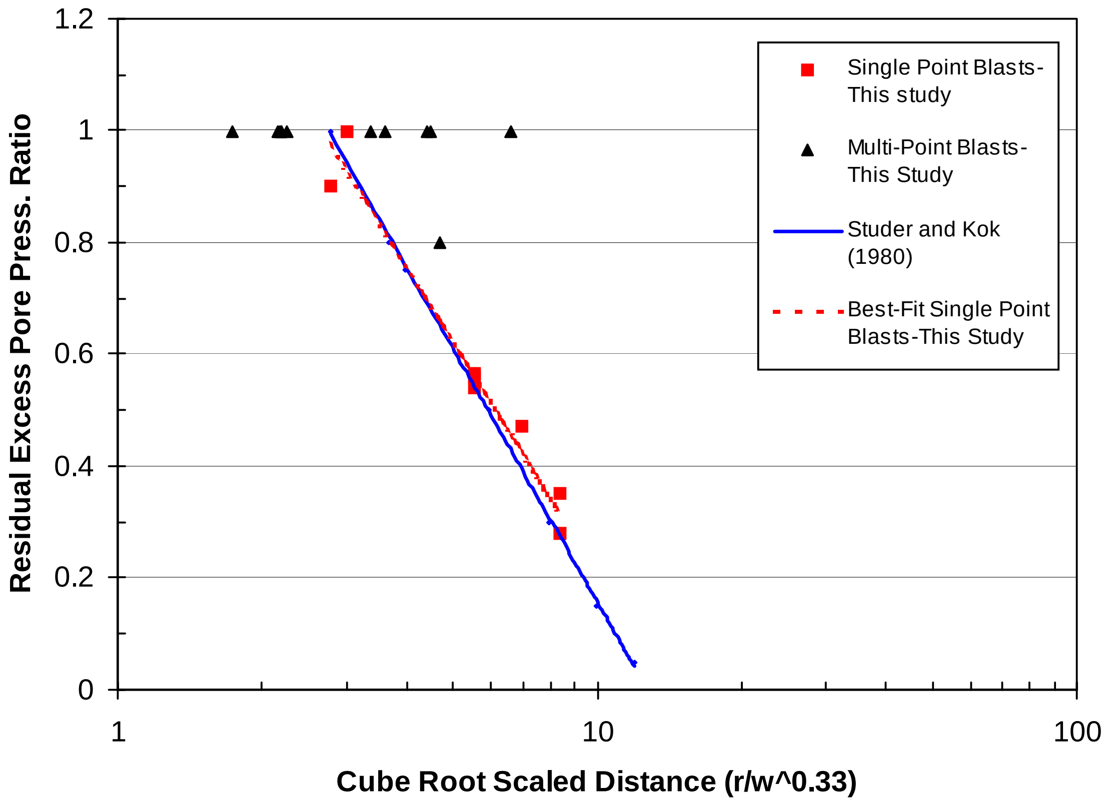

Fig. 4 Measured Excess Pore Pressure Ratio vs Scaled Distances for Single and Multiple Blasts Relative to Predicted Ratios Using the Studer and Kok (1980) Relationship.

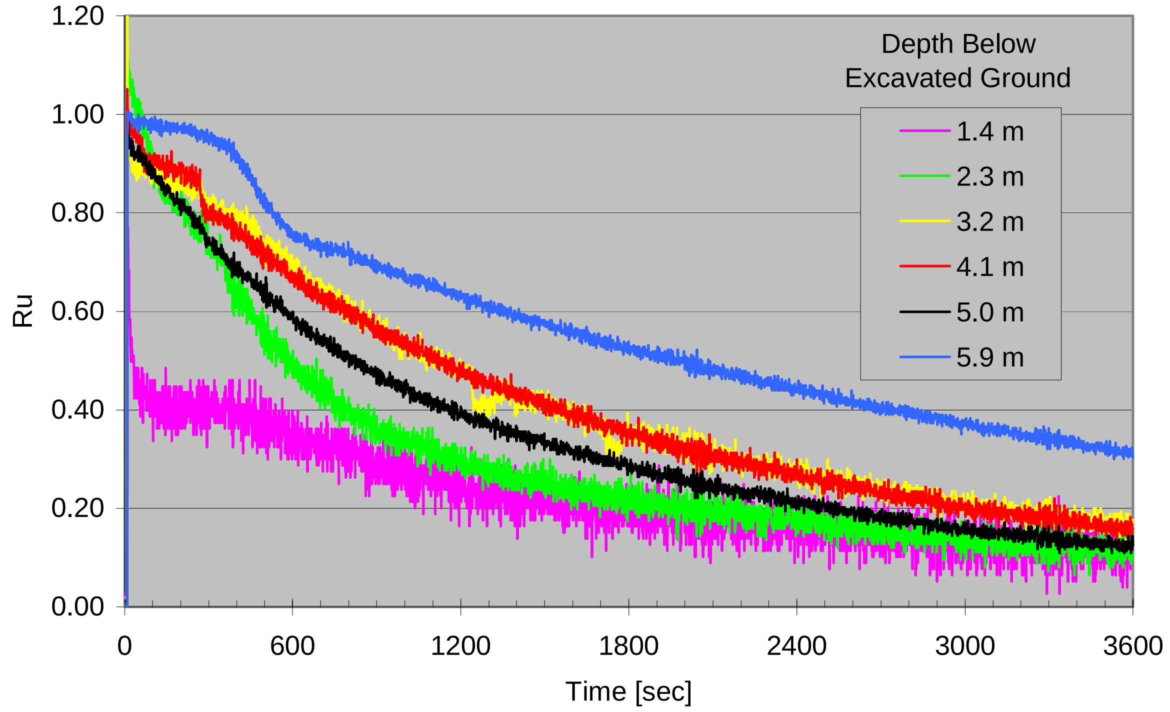

Fig. 5 Measured Excess Pore Pressure Ratio vs Time for a Vertical Array at the Center of the East Blast Zone (Point A.)

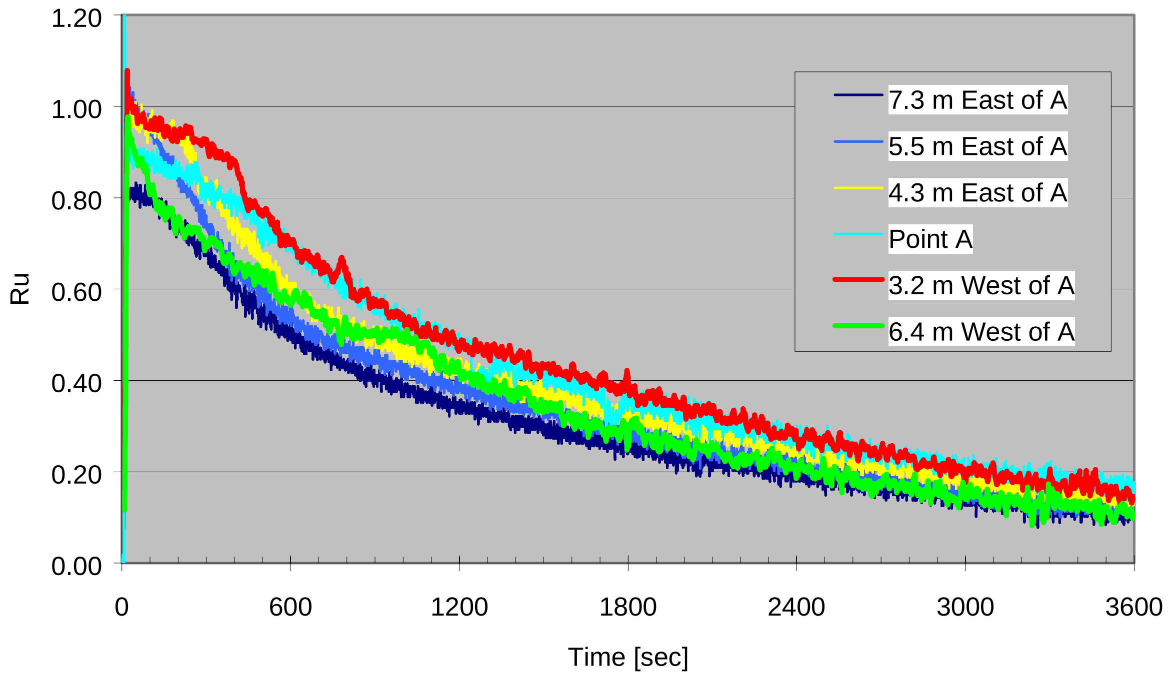

Fig. 6 Measured Excess Pore Pressure Ratio vs Time for a Horizontal Array Along and East-West Axis Through the Blast Zone at a Depth of 3.2 m.

Excess Pore Pressure

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. 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 an 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 8 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 due to the fact that 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 an 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 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 small. 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 blast area is shown in Fig. 7 for both blasts. 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.

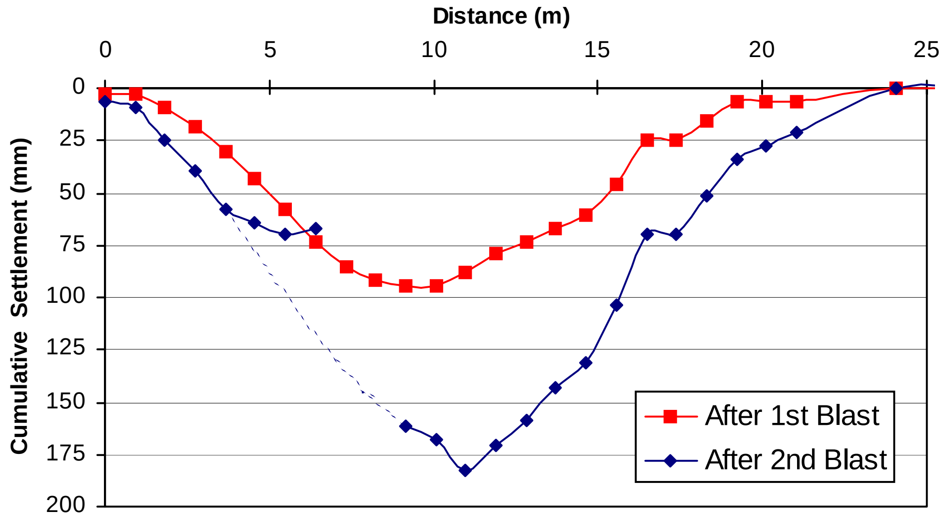

Fig. 7 Cumulative Settlement Along an East-West Section Through the Blast Zone for the Two Blasts at the Pilot Liquefaction Test Site.

CONCLUSIONS

Based on the field testing conducted in connection with this study the following conclusions can be made:

1. 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 generated within a depth range of 1.8 m to 5.9 m and over a 12.8 m x 19.2 m surface area. R_u values greater than 0.8 were typically maintained for 4 minutes and values greater than 0.6 for 8 minutes.

2. 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.

3. 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.

4. 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.

REFERENCES

Dobry, R., Abdoun, T., and O'Rourke, T.D. (1996). "Evaluation of pile response due to liquefaction induced lateral spreading of the ground" *Procs. Fourth Caltrans Seismic Design* Workshop, 10 p.

Studer, J. and Kok, L. (1980). "Blast-induced excess porewater pressure and liquefaction experience and application, *Intl. Symp. on Soils Under Cyclic and Transient Loadings*, Swansea, Wales, p. 581-593.

Hachey, J.E., Plum, R.L., Byrne, R.J., Kilian, A.P., Jenkins, D.V. (1994). "Blast-Densification of thick, loose debris flow at Mt. St. Helen's, Washington." *Vertical and Horizontal Deformations of Foundations and Embankments, ASCE Geotech. Spec. Pub. 40*, pp. 502-512.

Kulhawy, F.H. and Mayne, P.W. (1990). "Manual on Estimating Soil Properties for Foundation Design", Electric Power Research Institute, EL-6800 Research Project 1493-6 Final Report, p. 2-24 and 2-33.

Narin van Court, W.A. and Mitchell, J.K. (1995). "New insights into explosive compaction of loose, saturated, cohesionless soils" *Soil Improvement for Earthquake Hazard Mitigation, ASCE, Geotech. Spec. Pub. 49*, p. 51-65.

Wilson, D.W., Boulanger, R.W., Kutter, B.L. and Abghari, A. (1996). "Soil-Pile-Superstructure interaction experiments with liquefiable sand in the centrifuge" *Procs., Fourth Caltrans Seismic Design Workshop*, 12 p