16 Apr 2004, 1:30pm - 3:30pm

Liquefaction Hazard Mitigation by Prefabricated Vertical Drains

Kyle M. Rollins
Brigham Young University, Provo, Utah

R. Robert Goughnour
Nilex, Inc., Leesburg, Virginia

J. K. S. Anderson
Mendenhall-Smith, Inc., Las Vegas, Nevada

Stacey F. Wade
Brigham Young University, Provo, Utah

Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

LIQUEFACTION HAZARD MITIGATION BY PREFABRICATED VERTICAL DRAINS

Kyle M. Rollins
Brigham Young University
Provo, Utah-USA

R. Robert Goughnour
Nilex, Inc.
Leesburg, Virginia-USA

J. K. S. Anderson
Mendenhall-Smith, Inc.
Las Vegas, Nevada-USA

Stacey F. Wade
Brigham Young University
Provo, Utah-USA

ABSTRACT

Liquefaction has typically been mitigated by in-situ densification; however vertical composite drains offer the possibility of preventing liquefaction and associated settlement while reducing the cost and time required for treatment. Three case histories are presented which describe the use of vertical drains to mitigate liquefaction hazard and techniques to control the flow of water exiting the drains. In addition, results from a test case are presented where controlled blasting techniques were used to evaluate drain performance in-situ. Blasting was successful in liquefying loose sand in an untreated test site. Similar blast charges were then detonated at adjacent sites treated with drains. Measurements demonstrated that the drains significantly increased the rate of pore pressure dissipation. In addition, the installation process typically densified the surrounding soil, thereby decreasing the liquefaction potential. Computer analyses successfully matched the measured response and suggest that the drains could be effective for earthquake events.

INTRODUCTION

Liquefaction and the resulting loss of shear strength can lead to landslides, lateral spreading of bridge abutments and wharfs, loss of vertical and lateral bearing support for foundations, and excessive foundation settlement and rotation. Liquefaction resulted in nearly $1 billion in damage during the 1964 Niigata Japan earthquake (NRC, 1985), $99 million damage in the 1989 Loma Prieta earthquake (Holzer, 1998), and over $11.8 billion in damage just to ports and wharf facilities in the 1995 Kobe earthquake (EQE, 1995). The loss of these major port facilities subsequently led to significant indirect economic losses.

Typically, liquefaction hazards have been mitigated by densifying the soil in-situ using techniques such as vibro-compaction, deep soil mixing, dynamic compaction, or explosive compaction. Although these techniques have generally proven effective in clean sands, they are not generally successful for sands with higher fines contents. An alternative to densifying the sand is to provide drainage so that the excess pore water pressures generated by the earthquake shaking are rapidly dissipated thereby preventing liquefaction from occurring. The concept of using vertical gravel drains for liquefaction mitigation was pioneered by Seed and Booker (1977). They developed design charts that could be used to determine drain diameter and spacing. Improved curves which account for head losses were developed by Onoue (1988).

Although gravel drains or stone columns have been utilized at many sites for liquefaction mitigation, most designers have relied on the densification provided by the stone column installation rather than the drainage. Some investigators suspect that significant settlement might still occur even if drainage prevents liquefaction. In addition, investigators have found that sand infiltration can reduce the hydraulic conductivity and flow capacity of gravel drains in practice relative to lab values (Boulanger et al, 1997).

One recent innovation for providing drainage is the use of vertical, slotted plastic drain pipes "Earthquake Drains", 75 to 150 mm in diameter. These drains are installed with a vibrating steel mandrel in much the same way that pre-fabricated vertical drains (PVDs) are installed for consolidation of clays. The drains are typically placed in a triangular grid pattern at center-to-center spacings of 1 to 2 m depending on the permeability of the treated soil. In contrast to conventional PVDs, which have limited flow capacity (2.83×10^{-5} m^3/sec, for a gradient of 0.25), a 100 mm diameter drain can carry very large flow volumes (0.093 m^3/sec) sufficient to relieve water pressure in sands. This flow volume is more than 10 times greater than that provided by a 1 m diameter stone column (6.51×10^{-3} m^3/sec). Filter fabric tubes are placed around the drains to prevent infiltration of silt and sand. These vertical drains can be installed more rapidly and at a fraction of the cost of stone columns. For example, for a 12 m-thick layer, treatment with stone columns would typically cost $\$107/m^2$ of surface area and vibro-compaction would cost $\$75/m^2$, while the drains only cost $\$48/m^2$. In addition, the drains can be installed in about one-third to one-half of the time required to treat a profile using conventional means.

Vertical drains have been used at several sites throughout the US and abroad to mitigate liquefaction hazard. Case histories describing the installations at three sites are provided in this paper. An important part of the design of an Earthquake Drain system is the location and form of the reservoir. Pressure must be provided to lift expelled water from the ground water table elevation to the reservoir. Since this pressure appears as back pressure to water entering the drain, it is important to keep this distance as small as possible. The following three case histories illustrate different methods of providing the requisite reservoir space.

To this point, none of the earthquake drain installations has experienced a seismic event large enough to produce liquefaction. This lack of field performance data is an impediment to expanding the use of this technique. In addition, there is little data available to indicate what degree of densification would be produced during drain installation and how this would improve overall performance. Rather than instrumenting a field site and waiting for an earthquake to test the drain behavior, we have used controlled blasting techniques to produce liquefaction under field conditions and compared behavior with and without vertical drains. This paper also reports the results of a blast liquefaction test carried out at a site near Vancouver, BC.

EARTHQUAKE DRAIN CASE HISTORIES

San Francisco-Oakland Bay Bridge

As part of a seismic upgrade of the San Francisco-Oakland Bay Bridge, Caltrans determined that it would be more cost-effective in the long run to replace the East Span than to attempt its retrofit. The replacement of the East Span is scheduled for completion in 2007.

The design of the new span incorporates upgraded traffic safety issues. To accommodate these upgrades it was necessary to widen the roadway connecting the Skyway section to the existing freeway lanes west of the Bay Bridge Toll Plaza. This roadway is called the Oakland Touchdown and will be built atop what is called the "Geofill" area (California Alliance for Jobs, 2003). This fill was to be built over existing mole fill material, which overlays soft Bay Mud (see Fig. 1).

Calculations indicated that excessive consolidation settlement in the Bay Mud would occur under the added load. It was determined that installation of vertical prefabricated drains (wick drains) with surcharging would be the most cost-effective treatment. However, in addition to the consolidation problem, calculations also indicated that the existing mole fill material would liquefy under shaking from the design 8.1 magnitude earthquake. A scheme to address both of these problems is illustrated on the typical cross section shown on Fig. 2.

As shown, this scheme included both wick drains and Earthquake Drains. The construction sequence was to first excavate as indicated to elevation -0.44 m. Approximately 0.47 m of Class 3 permeable aggregate was then placed over a geotextile, bringing the surface to about elevation +0.03. Approximately 6000 wick drains were then installed through the class 3 stone to depths ranging from about 10 m to 25m. The wick drains were placed at a triangular spacing of 1.8 m and were cut off at the surface of the stone.

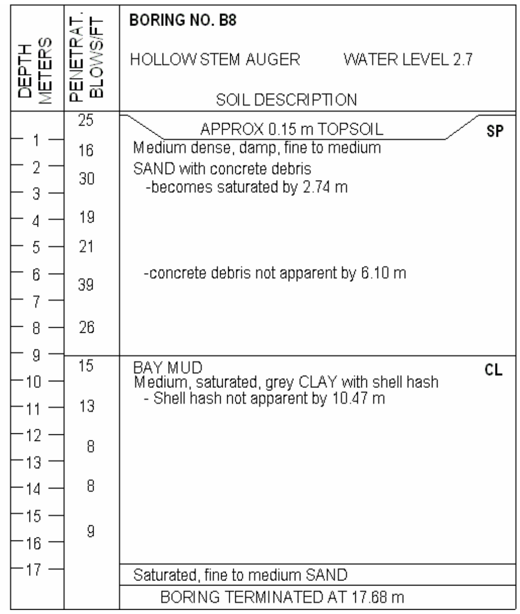

Fig. 1. Typical boring log from "Geofill" area.

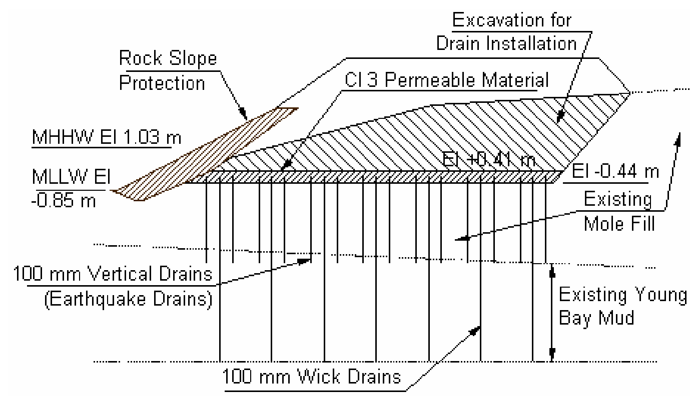

Fig. 2. Typical cross section Bay Bridge, Oakland Touchdown.

Fig. 3. Installing Earthquake Drains at Bay Bridge.

As shown in Fig. 3, approximately 17,000 100 mm Earthquake Drains were then installed through the rock to the bottom of the original mole fill, with depths ranging from about 3.5 m to 6.5 m.

These drains were installed within a vibrating mandrel to achieve densification of the mole fill simultaneously with installation. Three symmetrically spaced fins attached to the mandrel helped to transmit vibration to the soil. Approximately 0.6 m of settlement occurred during installation of the 6.5 m Earthquake Drains.

The tops of the drains were then trimmed close to the top of rock and elbows were placed atop each drain. Another approximately 0.38 m of stone was bladed over the drains. The stone was bladed over the drains from the closed side of the elbows, to avoid stone falling into the drains, thus leaving a clear path for discharge of water from the drains into the stone. After placing a geotextile over the stone the rock slope protection and embankment were built as shown. In addition a surcharge load was added which is anticipated to remain in place for about 9 months.

Since the rock drainage layer is open to the rock slope protection, efficient drainage is provided to the bay for water expelled during consolidation. During low tide the drainage stone is above water table and reservoir space is available for water expelled during a seismic event. At high tide water can move freely through the drainage stone to and through the rock slope protection with very little resistance.

<u>Barnard Elementary School Library, San Diego, CA</u>

Recent construction at the Barnard Elementary School in San Diego, California included construction of a new 2000 square foot library. The site, located in the coastal plain portion of the Peninsular Ranges, Geomorphic Province of California, is underlain by the Bay Point Formation, unconsolidated Bay Deposits, and fill soils as shown in Fig. 4.

The Bay Deposits below groundwater level were estimated to have a high potential for liquefaction when subjected to the Design Basis ground motion. The estimated peak ground acceleration resulting from the Design Basis Earthquake was 0.4g. Resulting dynamic settlement was estimated to be on the order of 6 to 9 inches with differential settlement on the order of 1-1/2 inches.

The new library was to be constructed within about 6 feet of an existing building founded on spread footings. Earthquake Drains were determined to be the most cost-effective solution. Calculations indicated that a 3.5-foot triangular spacing with a maximum lift of about 5 feet from the ground water table to the reservoir would limit the maximum pore pressure ratios to lessthan 0.6 during the design earthquake, and reduce estimated seismic settlements to less than 1 inch.

The construction sequence, illustrated in Fig. 5, first required excavation over the building area to a depth of elevation +5 feet, and 10 feet outside of the building footprint, except near the existing K2 building. Geotextile was placed over the bottom of this excavation and 6 inches of crushed rock was placed on the geotextile as a working mat. Approximately 400 drains were installed through the rock and geotextile to the top of the Bay Point Formation.

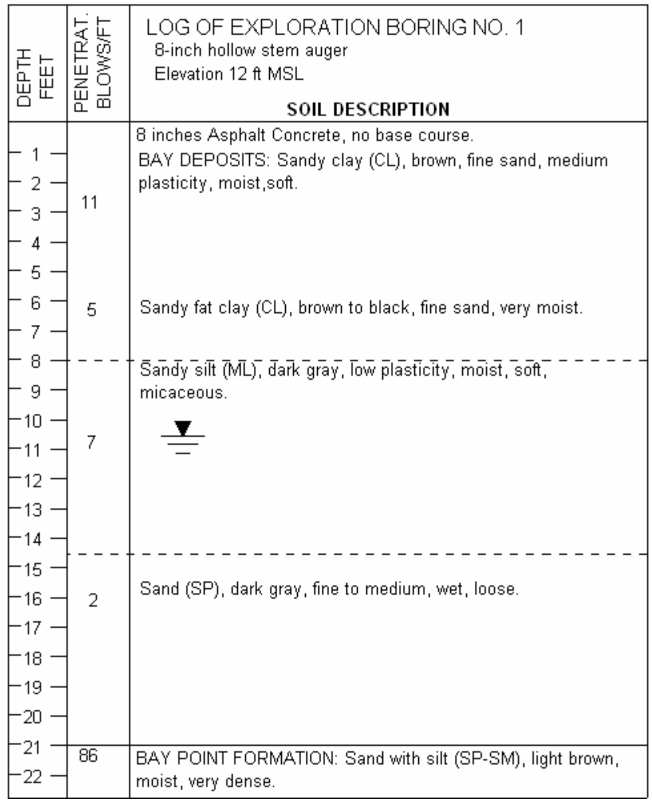

Fig. 4. Typical boring log, Barnard Elementary School library.

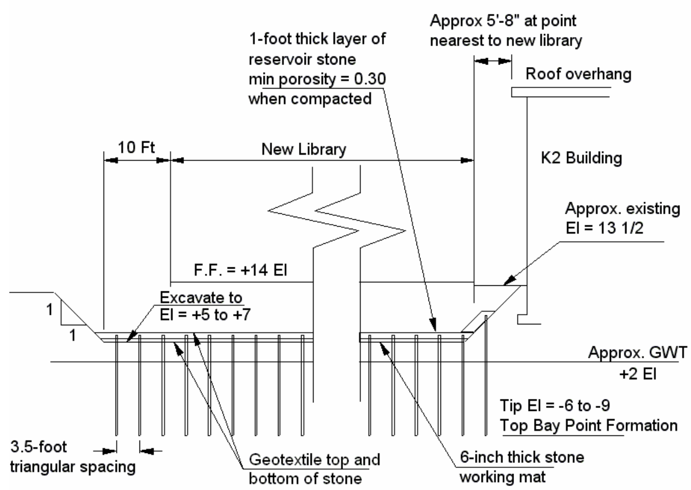

Fig. 5. Cross section of ground treatment at Barnard Elementary School Library.

The drains were all trimmed to a uniform height. High-permeability geotextile was attached over the tops of the drains and a 1-foot layer of open graded stone was placed over the drains. Another geotextile was placed, and the area was filled with structural fill and building of the library proceeded. The library was founded on spread footings constructed in the structural fill. Figure 6 shows a photograph of the drains being trimmed preparatory to placing the reservoir stone. Although drains were installed within about 6 feet of the K2 building, no damage to the building was observed.

Fig.6. Trimming drains to uniform height at Barnard Elementary School Library.

Hyatt Regency Hotel and Casino

To take advantage of beautiful vistas of the Atlantic Ocean and the Caribbean Sea, the causeway connecting the main island of St. Lucia, British West Indies with Pigeon Point was chosen as the site for a new Hyatt Regency Hotel and Casino.

The relatively flat causeway was man-made between 1969 and 1973 by hydraulically placing fill excavated from the bay bottom. This fill, approximately 11 m deep, consists of coral sand (varying in size from fine sand to fine gravel), and contains thin silt layers or clay layers, and occasional lenses of organic material. SPT N-values were typically greater than 20 blows/0.3 m in the 0 to 2 m depth range. However, N-values were typically less than 5 blows/0.3 m from about 1.5 to 4.5 m below the ground surface. In the 4.5 to 9 m depth range N-values usually varied from about 10 to 15, with significant amounts of data less than or exceeding this rough average range.

Natural coral sand was encountered below the hydraulic fill. The natural deposits varied from fine to medium sand to sandy gravel composed of coral gypsum. Below 9 m the N-values were generally in the 10 to 15 range. Weathered rock was encountered below the coral sand at depths of about 12.2 to 13.7 m, consisting typically of cemented clayey or silty sand. Groundwater was encountered at depths of 1.5 to 1.8 m.

Review of historical seismologic data indicated several earthquakes with magnitude 7.0 to 7.7 within 100 kilometers of St. Lucia. The design earthquake, magnitude 7 to 7-1/2 at a distance of 25 to 100 kilometers, would produce a maximum ground acceleration of about 0.33g. Although the four-storey structure was to be founded on piling, there was concern that the loose sands might liquefy, leaving the piling without lateral support.

Calculations indicated that 100 mm Earthquake Drains spaced at 1.22 m and installed to a depth of 13.7 m would limit excess pore pressure ratios to less than 0.6 during the design earthquake. If the drains extend nearly to the ground surface there would be sufficient reservoir space within the drain itself to contain the expelled pore water. The scheme adopted is shown in Fig. 7 and a photo of the drains is presented in Fig. 8. The drains at the lower level inside the building were allowed to discharge directly onto the ground surface. Two rows of drains were installed outside the building perimeter.

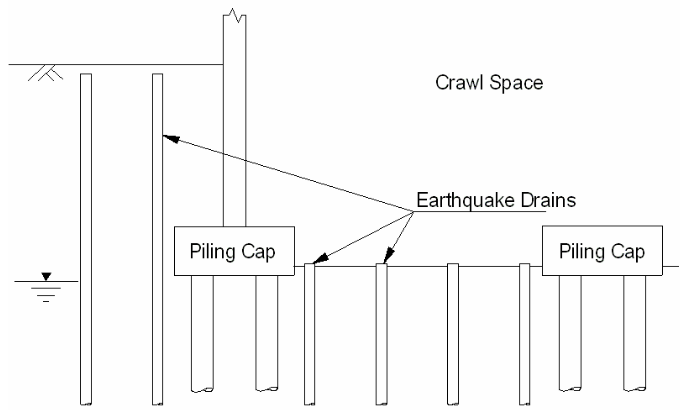

Fig. 7. Conceptual treatment scheme for Hyatt Regency Hotel and Casino.

Fig. 8. Photograph of drain installation at Hyatt Regency Hotel and Casino prior to installation of piles.

On June 8, 1999 a magnitude 5.4 earthquake did occur with epicenter about 100 km northeast of St. Lucia's capital city, Castries. Estimated ground accelerations across the northern portion of the island were on the order of 0.12g. It is interesting to note that although no evidence of liquefaction was apparent on the island, excess pore pressures were generated from the ground shaking, raising the water level in the drains and in some cases spilling small amounts of water onto the ground surface as intended.

Vancouver, British Columbia Blast Test

Drain behavior at the Vancouver test site was evaluated by installing 35 Earthquake Drains at one site and comparing the pore pressure and settlement behavior with an adjacent untreated site after blasting. Figure 9 shows the layout of the drains, blast charges and piezometers at the drain test area. The same blast charges and layout were used for the adjacent untreated test site. The Vancouver Earthquake Drain tests were conducted at a test site near the south portal of the Massey tunnel which runs under the Fraser River. This site is within about 200 m of a test site thoroughly characterized in connection with the Canadian Liquefaction Experiment (CANALEX) (Wride et al, 2000).

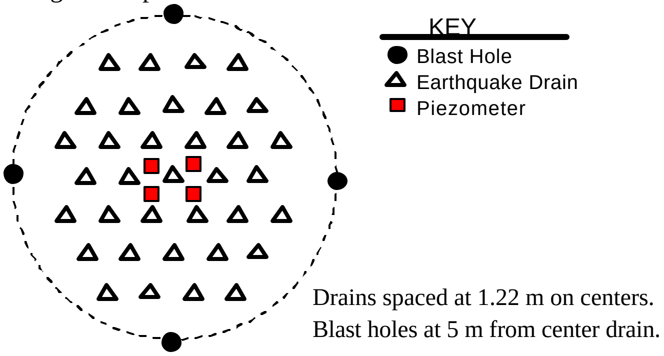

Fig. 9 Layout of Earthquake Drains, blast charges, and piezometers for Vancouver, BC drain test.

Drain and Soil Properties. The corrugated ADS drain pipes used at this site had an inside diameter of 100 cm, an outside diameter of 120.7 mm, and a flow area of 81.7 cm^2. Each drain was wrapped in a filter fabric (Synthetic Industries SB252) with an AOS of 50 microns. Additionally, the lower end of the fabric tube was tied to prevent infiltration.

Two CPT soundings were performed to compare the soil properties at the drain test site with the untreated site. The soil profiles were very similar at each site and consisted of silt and clay to a depth of 6 m which was underlain by loose clean sand to a depth of about 15 m. The clean sand typically classified as SP material according to the USCS and generally had a D_{50} between 0.2 and 0.3 mm. The water table was approximately 2.8 m below the ground surface during testing. The average cone resistance was between 5 and 7 MPa in the clean sand layer. Based on correlations with the CPT cone resistance, the relative density (D_r) was generally between 40 and 45% (Kulhawy and Mayne, 1990).

Drain Installation and Layout. Figure.9 shows the layout of the drains and piezometers as well as the location of the blast charges. The drains were installed in a triangular pattern with a spacing of 1.22 m center to center and extended to the ground surface. Therefore, the only reservoir provided was the volume of the drain pipe above the water table. The drain pipes were attached to 150 mm square steel anchor plates and pushed to the target depth of 12.8 m using a pipe mandrel with three radial fins. The mandrel was installed using an ICE Model 44 vibratory hammer suspended from a 70 tonne mobile crane.

Instrumentation. Four piezometers were pushed into the ground around the center drain as shown in Fig. 9. The piezometers were located at depths of 6.7, 9.1, 11.6, and 14.0 m below the ground surface. In addition, pore pressure piezometers were lowered to depths of 6.7 and 11.6 in the center blast hole to provide a comparison between the pressure inside the drain and that in the surrounding soil. In the untreated test area, two piezometers were installed at a depth of 8.2 m and two piezometers were installed at a depth of 12.5 m. The pore pressure data was recorded using a laptop-based data acquisition system which recorded at a rate of 10 Hz.

Settlement was monitored using survey points along eight rays spaced at 45° angles extending from the center of each test area. The change in elevation of these points was used to determine settlement due to drain installation and the settlement due to blast-induced liquefaction. Blast-induced settlement was also recorded using string potentiometers anchored to the ground and attached to a cable which was stretched across the test site.

Installation Induced Settlement. A plot of the drain installation induced settlement is presented in Fig. 10. Nearly 350 mm of settlement occurred at the center of the test area which decreased to about 50 mm at the periphery of the drain cluster. This differential settlement is likely due to arching against the surrounding untreated soil. The settlement trough produced by the drain installation was left in place prior to the blast testing.

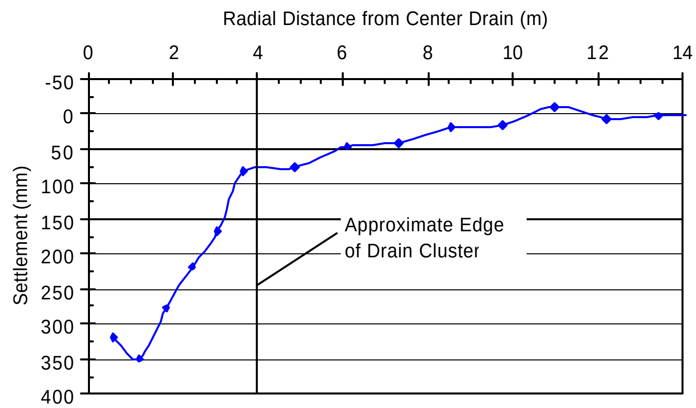

Fig.10 Drain installation induced settlement profile as a function of distance from the center of the drain test site.

Although the drain installation clearly compacted the sand in the profile, CPT tests performed a few days after the insertion of the drains actually indicated that the cone tip resistance had dropped to about half of its original value.

Blast Testing. A total of 16 explosive charges were detonated to produce a liquefied state. Four charges were placed in each of four holes around the periphery of a 5 m radius circle as shown in Fig. 9. In each hole, charges of 1.8, 1.8, 1.8 and 2.7 kg were placed at depths of 5, 8, 11, and 14 m, respectively. The charges were detonated one at a time with a delay of approximately 500 milliseconds.

Blast-Induced Pore Pressure. Shortly after the first four charges were detonated around the drain test site, water began rapidly flowing out of the drains indicating that high pore pressures had been produced. The measured pore pressure time histories also indicated that liquefaction was produced in about three or four stress cycles produced by the blasting. The large blast weights and the low liquefaction resistance of the loose sand combined to produce the rapid liquefaction.

Plots of R_u vs. time for piezometers in the untreated test site and drain test site are presented in Fig. 11 for two depths. In addition, R_u vs. time plots are provided for the piezometers positioned in the drains themselves. Although the drains were insufficient to prevent initial liquefaction, the rate of dissipation at both depths was significantly greater in the drain test area than in the untreated area. This clearly indicates that the drains were performing their function. The R_u value in the drains themselves also rose following blasting due to water flowing out of the drain and ponding on the ground surface. Once the R_u in the ground dropped below the R_u in the drain, the drains no longer provided any benefit and the dissipation rate became equal to that of the untreated soil. Eventually the ponded surface water flowed back down the drains and the static water level was re-established.

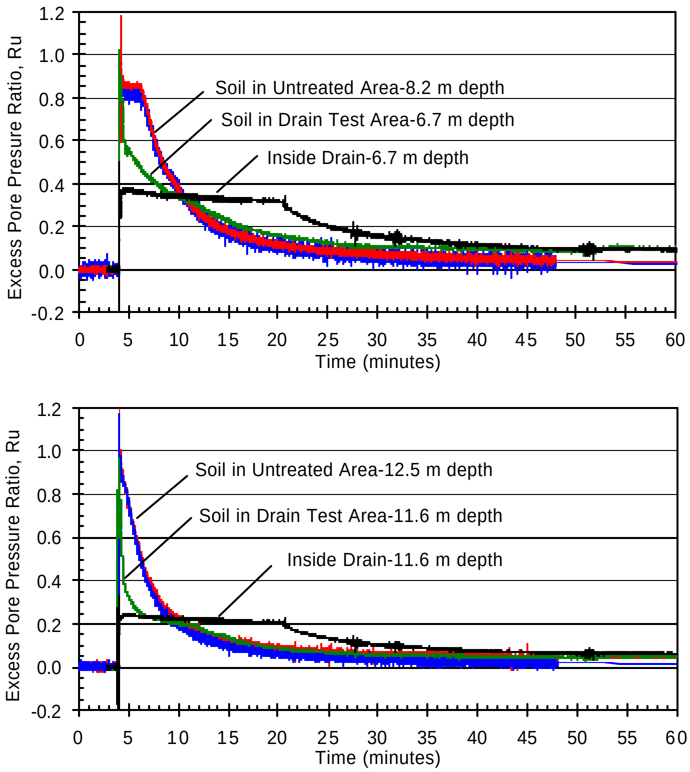

Fig. 11 Comparison of R_u vs. time curves for the piezometers in the drain test and untreated test sites in Vancouver at two depth levels.

Blast Induced Settlement. Plots of liquefaction induced settlement vs. distance from the center of the drain test and untreated test areas are shown in Fig. 12. Despite the fact that initial liquefaction occurred in both test areas, the maximum settlement in the untreated test area was 30 to 65% higher than that in the drain test area. In addition, the settlement within the drain test area was much more uniform than what was observed for the untreated test area. Part of the reduction in settlement is likely due to the densification produced by the installation of the drains; however, as shown by similar tests conducted at Treasure Island (Rollins et al, 2002), some of the reduction is likely due to the increased rate of dissipation.

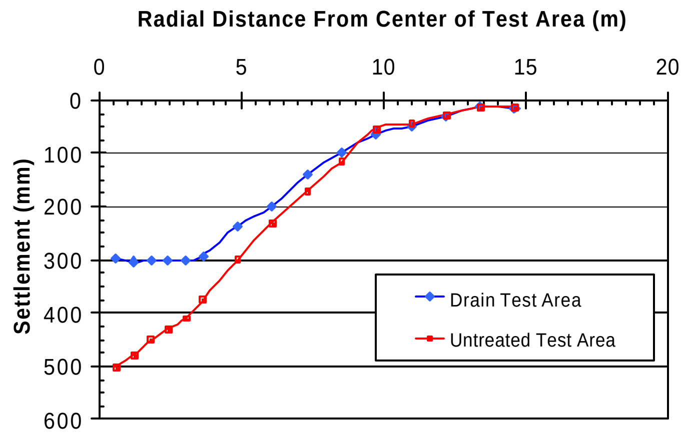

Fig. 12 Liquefaction induced settlement versus radial distance from the center of the drain test and untreated test areas.

Analysis of Test Results. Because the blast testing approach produces liquefaction much more rapidly than an earthquake, there is less time for pore pressure dissipation and the effectiveness of drains in an earthquake may be obscured. The blast sequence at the Vancouver test site took only 2 or 3 seconds to produce liquefaction while destructive earthquakes might take 10 to 60 seconds to produce liquefaction. The longer time for pore pressure buildup allows the Earthquake Drains to operate more effectively in limiting pore pressure generation.

To provide increased understanding of the behavior of the drains in an earthquake, analyses were performed using the computer program FEQDrain (Pestana et al, 1997). The computer model was first calibrated using the measured settlement and pore pressure time histories from the blast test. Then, the calibrated soil properties were held constant while the duration of shaking was increased to match typical earthquake durations. The soil layering used in the model was based on the CPT soundings. The initial estimate of permeability (k_x and k_y) for each layer was based on borehole permeability testing that was performed with a double packer inside several of the Earthquake Drains prior to the blast testing. The modulus of compressibility and duration of earthquake shaking were estimated using guidelines provided by Pestana et al (1997). Relatively small variations in these parameters were generally sufficient to obtain a reasonable match with the measured pore pressure dissipation and settlement time histories. Fig. 13 presents a plot showing the computed and measured R_u vs time curves, while Fig. 14 provides a plot of computed and measured settlement versus time curves. In both cases the agreement is relatively good.

Analyses were then performed using the same soil profile and properties but with durations typical of various earthquakes. The ratio of equivalent earthquake stress cycles to cycles producing liquefaction (N_q/N_l) was estimated. Table 1 provides a summary of the maximum computed R_u and settlement for various earthquake events and drain spacings. Table 1 suggests that appropriately designed drains can significantly reduce excess pore pressure and settlement.

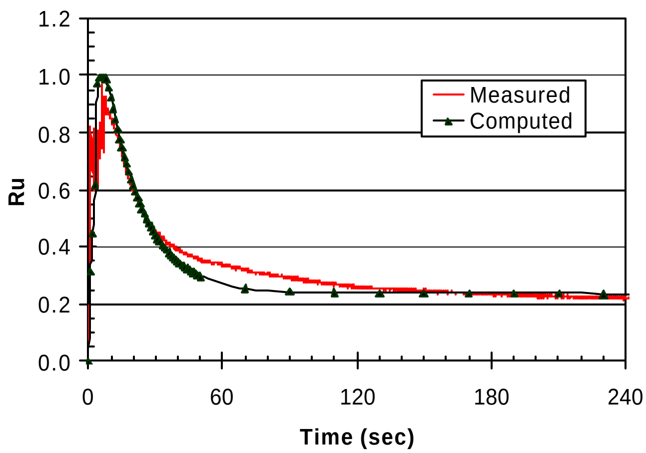

Fig. 13 Comparison of measured and computed excess pore pressure ratio (R_u) versus time at a depth of 11.8 m for the Vancouver test site.

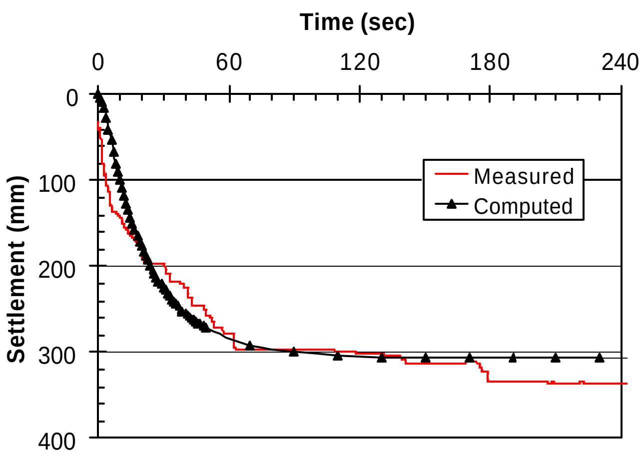

Fig. 14 Comparison of measured and computed settlement versus time curves for the Vancouver test site.

Finally, analyses were performed with FEQDrain to evaluate the effect of the inclusion of a horizontal gravel drain to serve as a reservoir. The gravel drain was 1 m-thick and the base was located 0.3 m above the water table in the model. Analyses were performed for the blast liquefaction test as described previously. Based on the analyses, placement of the reservoir prevented the water from rising above the ground surface and reducing the effectiveness of the drains as was the case without the gravel reservoir (see Fig. 11).

Table 1 Summary of computed maximum R_u and settlement for various earthquake events and drain spacings at the Vancouver site.

M	Duration (sec)	N_q/N_l	Drain Spacing (m)	Max. R_u	Settlement (mm)
Blast	8	4.0	1.22	1.0	310
6.0	8	2.0	0.91	0.40	31
6.75	17	2.0	0.91	0.47	35
6.75	17	3.0	0.91	0.61	48
7.5	35	2.0	0.91	0.65	53

Time histories of the computed excess pore pressure ratio with and without the gravel reservoir are plotted at three depths for comparison in Fig. 15. Although liquefaction (R_u=1) occurs at all depths without the reservoir, with the reservoir, peak R_u values are reduced to 0.6 and 0.4 at 9.1 and 11.6 m depths, respectively. In addition, for all depths, the rate of pore pressure dissipation is significantly greater with the gravel reservoir than without it. This is particularly evident for depths where initial liquefaction was prevented. These calculations clearly indicate the value of having a reservoir located close to the groundwater.

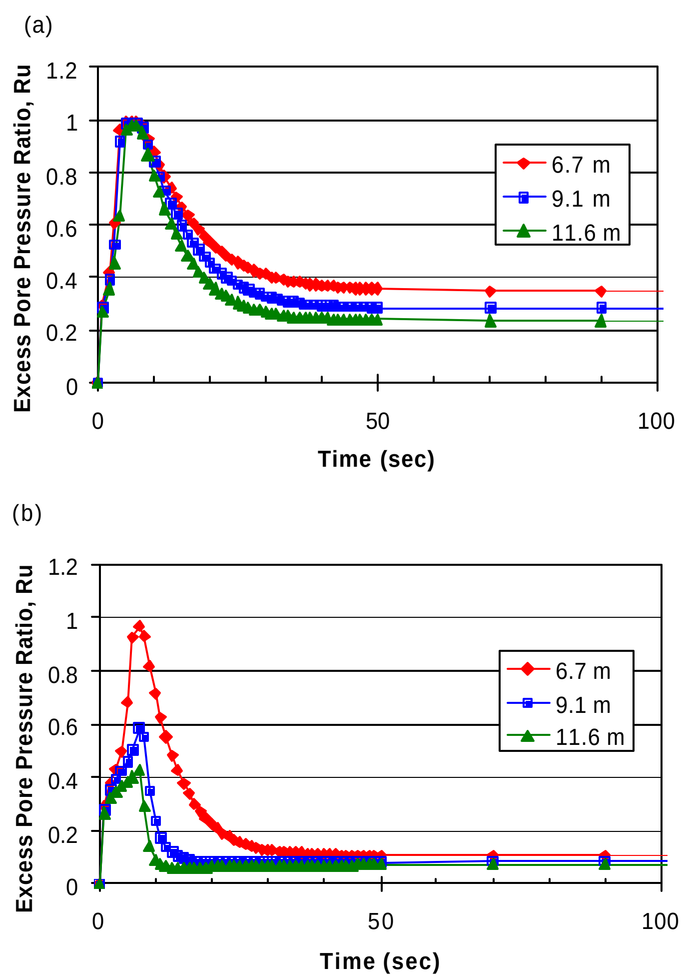

Fig. 15 Computed excess pore pressure ratio time histories at three depths (a) without gravel reservoir and (b) with gravel reservoir for the conditions during the Vancouver blast test.

CONCLUSIONS

1. Significant settlement may be achieved in the process of installing Earthquake Drains. This settlement leads to increased density and a lower compressibility which should reduce the amount of settlement and increase the rate of pore pressure dissipation relative to untreated sites in earthquakes.

2. Drains can be installed within 2 m of an existing building without causing any damage to the structure.

3. Horizontal drain blankets with geotextile filters can serve as a reservoir for flow exiting Earthquake Drains. Placement of the reservoir closer to the liquefiable zone improves the computed performance of vertical drains due to lower backpressure.

4. The presence of Earthquake Drains significantly increased the rate of excess pore water pressure dissipation relative to untreated areas in the test blasts. Some of this increase can be attributed to increased density but the increase was also observed when densification was less significant.

5. Settlement in areas treated with drains was reduced to only 60% of the settlement measured in untreated sites even after liquefaction.

6. Reasonable estimates of pore pressure dissipation rates and settlement can be obtained for the blast tests using FEQDrain. Further computer analyses, using soil properties calibrated with the blast test data, suggest that vertical drains can successfully limit pore pressure buildup and associated settlement for earthquake motions where stress cycles are applied more slowly than during a blasting event.

ACKNOWLEDGEMENTS

Funding for the Vancouver blast test was provided by the NCHRP-IDEA program and this support is gratefully acknowledged. The results do not necessarily reflect the opinions of the sponsors. We are grateful to the BC Columbia Ministry of Transportation for use of the Vancouver site. Dr. Blair Gohl of Pacific Geodynamics, Inc. provided data for the untreated test area at the Vancouver test site. David Anderson of BYU provided technical support for the blast tests.

REFERENCES

Booker, JR, Rahman, MS and Seed, HB [1976]. “GADFLEA: A Computer Program for the Analysis of Pore Pressure Generation and Dissipation During Cyclic or Earthquake Loading” *Report No. EERC* 76-24, Univ. of California, Berkeley.

Boulanger, DP, Hashish, Y and Schmidt, B [1997]. Drainage Capacity of Stone Columns or Gravel Drains for Mitigating Liquefaction, *2nd Geotechnical Earthquake Engineering and Soil Dynamics Conference*, Seattle, Vol. I, 678-690.

California Alliance for Jobs [2003], NewBayBridge.org
The Geofill Project,
www.newbaybridge.org/the_projects/geofill_project.html
Accessed June 24, 2003

EQE [1995]. “The January 17, 1995 Kobe Earthquake” Summary Report,
www.eqe.com/publications/kobe/economic.htm

Kulhawy, FH and Mayne, PW [1990]. *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute, Palo Alto, California, Research Report EERI EL-6800

Onoue, A [1988]. “Diagrams Considering Well Resistance for Designing Spacing Ratio of Gravel Drains,” *Soils and Foundations*, Japanese Soc. of Soil Mechanics and Foundation Engineering, 28(No. 3), 160-168.

Pestana, JM, Hunt, CE and Goughnour, RR [1997]. “FEQDrain: A Finite Element Computer Program for the Analysis of the Earthquake Generation and Dissipation of Pore Water Pressure in Layered Sand Deposits with Vertical Drains,” *Report No. EERC 97-17*, Earthquake Engineering Res. Ctr., UC-Berkeley, CA.

Rollins, KM, Anderson, JKS, Goughnour, RR, and McCain, AK [2003], “Vertical Composite Drains for Mitigating Liquefaction Hazard”, 13th Intl. Conference on Offshore and Polar Engineering, Intl. Society for Offshore and Polar Engineering, Paper 2003-SAK-01, 8 pp.

Seed, HB, and Booker, JR [1977]. “Stabilization of Potentially Liquefiable Sand Deposits Using Gravel Drains,” *J. Geotechnical Engrg. Div.*, ASCE, 103(GT7), 757-768.

Wride, CE, Robertson, PK, Biggar, KW, Campanella, RG, Hofmann, BA, Hughes, JMG, Kuppper, A, and Woeller, DJ [2000]. “Interpretation of In-Situ Test Results from the CANLEX sites”, *Canadian Geotechnical Journal*, NRC, 37: 505-529.