March 26, 2014
TPF-5(244) Shaking Table Vertical Drains
Task 1 Revised Literature Review

Literature Review

for

TPF-5(244)

Shaking Table Testing to Evaluate Effectiveness of Vertical Drains for Liquefaction Mitigation

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Introduction

Liquefaction of loose saturated sand results in significant damage to transportation systems in nearly every major earthquake event. 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 worth of 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. Port facilities in Oakland, Los Angeles and Seattle are vulnerable to similar losses.

Typically, liquefaction hazards have been mitigated by densifying the soil in-situ using techniques such as vibrocompaction, stone columns, compaction grouting, dynamic compaction, or explosives. 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. The excess pore pressure ratio (r_u = excess pore pressure divided by the vertical effective stress) must normally be kept below 0.4 to prevent excessive settlement due to increases in compressibility (Albaisa and Lee 1974, Seed and Booker, 1977) Vertical drains allow for pore pressure dissipation through horizontal flow which significantly decreases the drainage path length. This feature becomes particularly important when drainage is impeded by a horizontal silt or clay layer and a water interlayer forms further increasing the potential for sliding (Kulasingam et al. 2004). As shown in Fig. 1 vertical drains can relieve these pressures, prevent the formation of a water interlayer, and reduce the potential for lateral spreading and slope instability.

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).

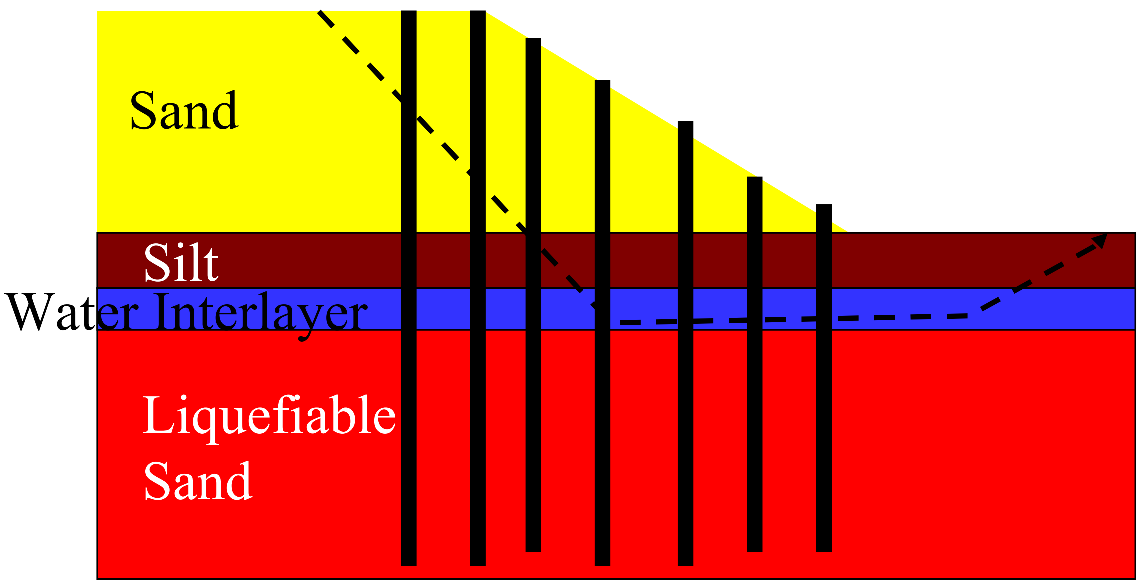


Fig. 1. Schematic drawing showing the potential for vertical drains to relieve pore pressures and intercept water interlayers which may form below a low permeability silt layer thereby reducing the potential for slope instability and lateral spreading.

One recent innovation for providing drainage is the geo-composite drain (Rollins 2003). As shown in Fig. 2, geo-composite drains are vertical, slotted plastic drain pipes also known as "EQ drains" which are typically 75 to 150 mm in diameter. These drains are installed with a vibrating steel mandrel in much the same way that smaller pre-fabricated vertical drains (PVDs) are installed for consolidation of clays. The geocomposite 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 x 10^{-5} m^3/sec, for a gradient of 0.25), a 100 mm diameter drain can theoretically carry very large flow volumes (0.093 m^3/sec) with the potential to relieve water pressure in sands. This flow volume is more than 10 times greater than that provided by a typical 1 m diameter stone column (6.51×10^{-3} m^3/sec). Filter fabric sleeves are placed around the drains to prevent infiltration of sand.

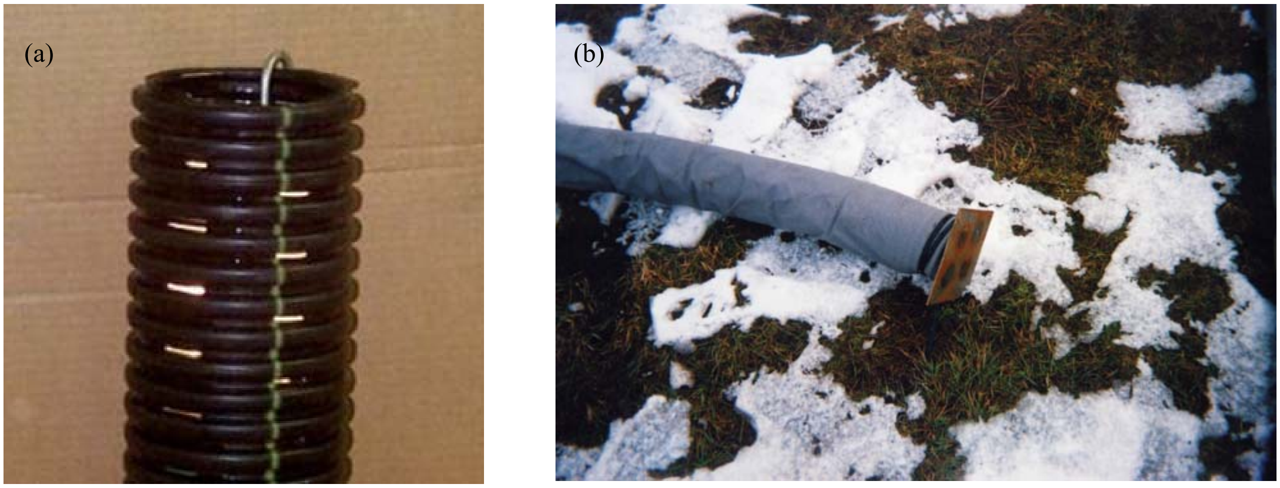


Fig. 2. (a) EQ Drain without filter fabric showing slots illuminated by light inside pipe and (b) EQ Drain with filter fabric and anchor plate at the end (Rollins et al, 2004).

Unfortunately, no field performance data is available to show how vertical drains actually perform when subjected to earthquake motions. In the absence of earthquake performance data, investigators have used a number of methods to investigate the effectiveness of vertical geo-composite drains. These methods include: (a) field tests involving controlled blasting or vibrations to induced liquefaction, (b) centrifuge testing with scaled models which are accelerated to simulate the stress levels existing under field conditions and (c) numerical methods.

Field Testing to Evaluate Vertical Drains for Liquefaction Remediation

Rollins et al (2003) employed controlled blasting techniques to generate excess pore pressures to test full-scale EQ drains at a test site on Treasure Island in San Francisco Bay. These tests investigated the pore pressure dissipation properties of EQ drains and the densification produced during drain installation. The test site consisted of two rings of blast holes with several test regions surrounding the blast holes. Each test region contained a cluster of seven EQ drains installed in a triangular grid pattern, incorporating various combinations of drain spacing, use of a filter sock, and amount of vibration used during installation. Installation settlement was dependent on the vibration energy and reached as much as 0.3 m. This densification increased the cone penetration resistance by about 25%.

Due to the rapid loading rate from the explosive changes, the EQ drains were unable to prevent liquefaction. However, dissipation rates were substantially increased as shown in Fig. 3.

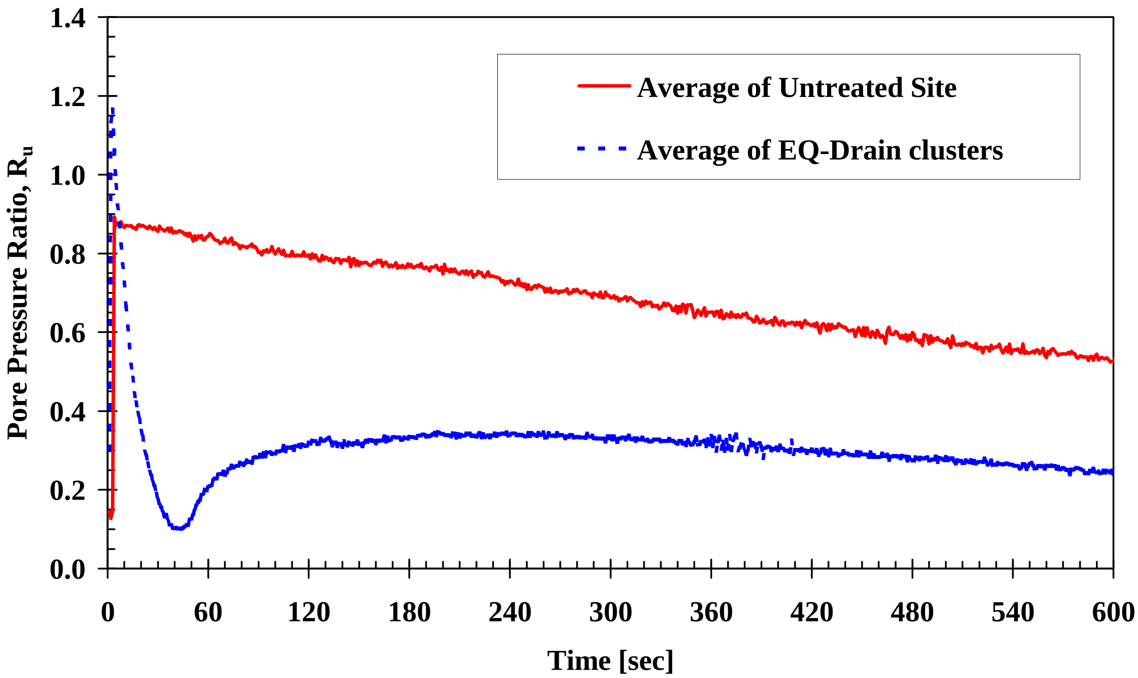


Fig. 3. Comparison of excess pore pressure ratio as a function of time at sites treated with EQ drains relative to an untreated test site (Rollins et al. 2003).

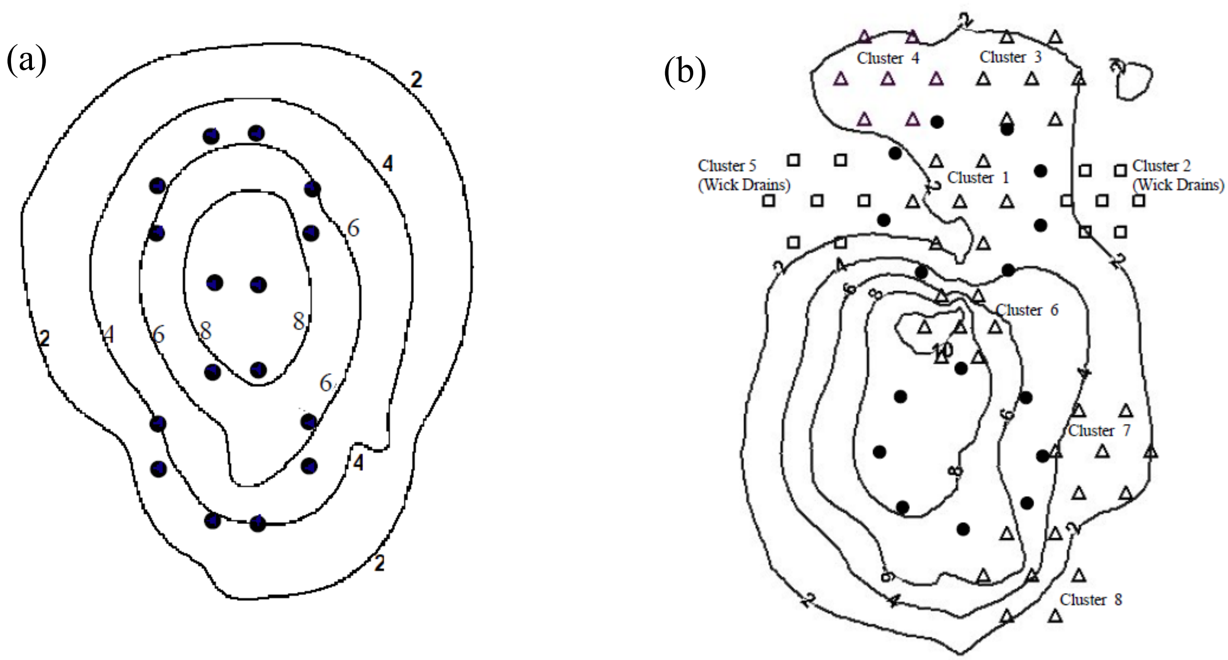


Fig. 4. Contours of measured settlement (in cm) for (a) untreated site and (b) site treated with clusters of EQ drains after detonation of 16 explosive charges around two 4.3 m diameter rings. Rollins et al, (2003).

Furthermore, post-liquefaction settlements were reduced from about 100 mm in the untreated region to less than 25 mm in several of the regions treated with drains (see Fig. 4). The increase in the pore pressure ratio after initial dissipation for the drain test areas in Fig. 3 appears to result from sand infiltration due to inadequate filter fabric. Several of the drains filled with sand.

Subsequently, blast liquefaction experiments with EQ drains were reported by Rollins et al. (2004) at a site south of Vancouver, BC, Canada. EQ drain performance was evaluated by installing a cluster of 35 EQ drains at one test site as shown in Fig. 5 and comparing the pore pressure and settlement behavior with an adjacent, untreated control site. The drains were installed using a vibratory mandrel in a triangular grid pattern with a center-to-center spacing of 1.22 m. Drain installation caused the soil within the boundaries of the cluster to settle with a maximum settlement of over 350 mm. The relative density of the treated sand was increased from an initial value of 40% to a final value of about 50% by the drain installation.

Sixteen explosive charges in four blast holes were used to induce liquefaction. Although a 0.5 second delay was used between blasts, the charges were very large (1.8 kg to 2.7 kg) and induced liquefaction within 2 seconds. Nevertheless, pore pressure dissipation rates were much faster with the drains than without as shown in Fig. 6. The drains were also able to reduce the total amount of settlement by about 40% when compared to the untreated site.

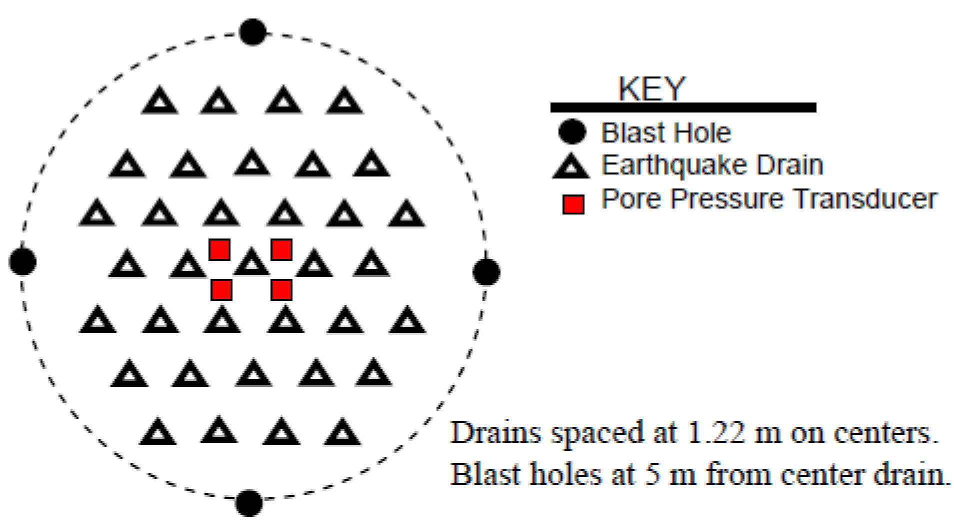


Fig. 5. Layout of EQ drains, blast holes and pore pressure transducers to monitor effectiveness of drains for liquefaction remediation.

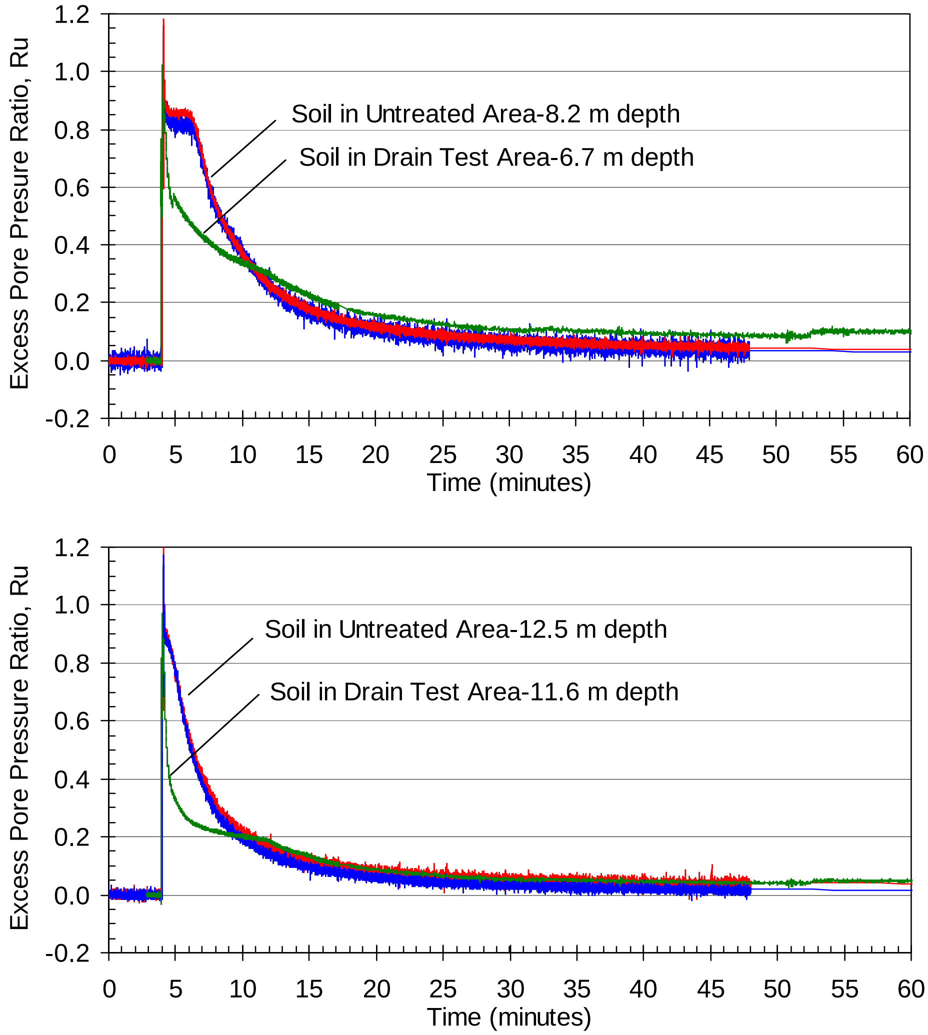


Figure 6. Comparison of measured excess pore pressure ratio vs. time following blasting at two depths with and without drains in place (Rollins et al. 2004).

Chang et al. (2004) performed field tests on a volume of reconstituted, saturated sand measuring 1.2 m x 1.2 m x 1.2 m, surrounded by an impervious membrane. Tests were conducted with and without an EQ drain in the center of the test volume. The relative density of the sand for both tests was approximately 35%. Stress cycles were applied using a large vibrosesis truck from the NEES-Univ. of Texas site and pore pressures and accelerations were measured at several points within the test volume.

Plots of the measured excess pore pressure ratio with and without a drain from this test are presented in Fig. 7. Without a drain, liquefaction was produced during the application of 60 stress cycles (3 second total duration), while the excess pore pressure ratio did not exceed 0.25

(25%) for the test volume with a drain subjected to the same vibrations.

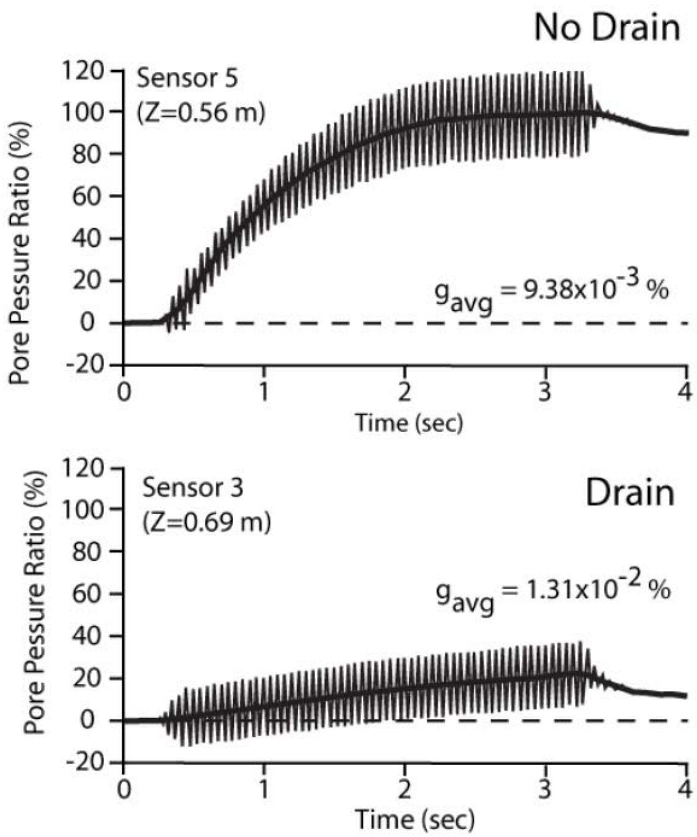


Figure 7. Comparison of excess pore pressure ratio at test sites with and without a drain while subject to cyclic strain from the NEES@UT-Austin Vibroseis Truck (Chang et al, 2004).

Volumetric strain decreased from 2.1% without a drain to less than 0.5% with a drain in place. While the EQ drain successfully prevented liquefaction for this shallow soil layer, drainage of a thicker layer would be more difficult. In addition, the applied strain amplitude was relatively small and a more severe motion could produce different results.

In 2010, full scale dynamic testing was performed using a vibratory hammer excitation source in an attempt to evaluate how effective EQ drains were in dissipating excess pore water pressures (Marinucci et al. 2010). The subsurface profile beneath the topsoil consisted of relatively clean loose-to-medium dense sand underlain by silt and clay, though it was interbedded and highly variable with a water table at approximately 2 feet below ground surface (bgs). The liquefaction sensors consisted of both miniature pore water pressure transducers and tri-axial accelerometers. Crosshole seismic testing was performed to assess the saturation of the soil. The average shear wave velocities indicated that the soil was not liquefiable, but the average stress-corrected CPT tip stress values indicated the soil was highly liquefiable. The discrepancy between the two in situ test parameters was attributed to the age and cementation of the Pleistocene era sand.

The vertical EQ drains were installed using a vibratory mandrel, followed by dynamic testing that vibrated on opposite sides of the test area at various distances from the centerline. The layout of test is shown in Fig. 8. The vibratory installation of one drain was used as a test of the untreated condition. Shear wave velocity decreased after the installation and testing of the drains. Significant settlement during installation of the drains indicated considerable densification of the sand, which contributed to the reduced dynamic and pore pressure responses. This densification was presumably due to breaking of cementation bonds within the sand.

Although lower excess pore pressure and settlements were generated in the treated ground relative to the untreated ground, the comparisons are not definitive. Unfortunately, the vibratory hammer also produced significantly lower accelerations in the treated ground so that it was not possible to say conclusively whether the improved performance came from improved drainage or the reduced acceleration levels.

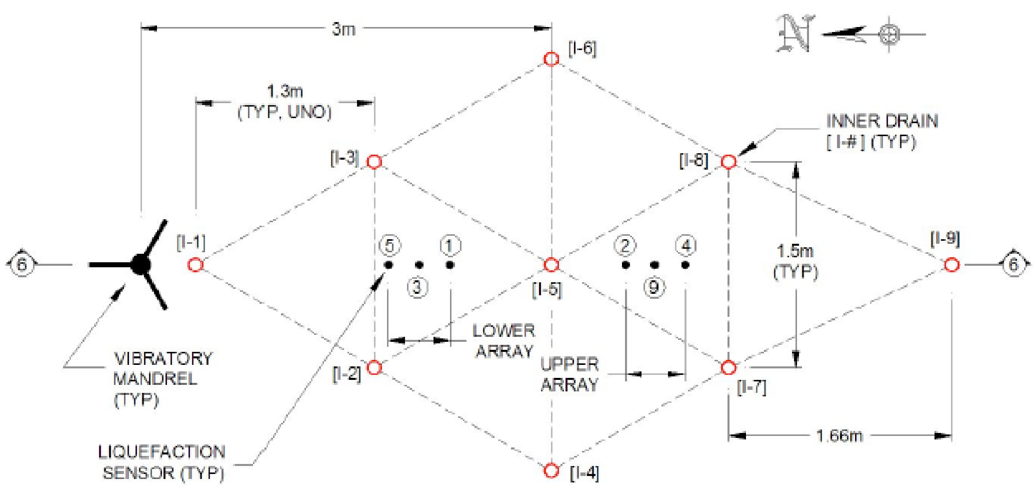


Figure 8. Plan view of the instrumentation and vertical drain geometry. The vibratory mandrel source is shown at only one position for clarity (Marinucci et al. 2010).

Centrifuge Testing to Evaluate Vertical Drains for Liquefaction Remediation

As part of a NEESR grand challenge study, three dynamic centrifuge tests were performed to evaluate EQ drain performance through time histories of acceleration, displacement, excess pore water pressure (Δu), and excess pore water pressure ratio (r_u). Results from the first test were discussed by Kamai et al. (2007), Marinucci et al. (2008), Howell et al. (2009a), and Marinucci (2010), and results from the second test were discussed by Kamai et al. (2008) and Marinucci (2008). The first centrifuge test was used to investigate the ability of vertical drains to prevent lateral spreading. Testing was performed to compare performance of two 3° slopes, one with and one without vertical drains. At prototype scale, the soil profile consisted of a 5.5-m thick liquefiable sand overlain by a 0.5-m thick silt layer. At acceleration levels between 0.11g and 0.15g full liquefaction and some soil deformations occurred on the untreated slope while smaller pore pressures and less deformation occurred on the slope with the vertical drains.

The second centrifuge test also involved the effect of prefabricated drains on lateral spreading with a 3° slope. At prototype scale, the profile consisted of a 4.8-m thick liquefiable zone (D_r=40%), but with a 1.0-m thick clay layer overlying it. The slopes were subjected to three significant earthquake motions, with peak ground accelerations of 0.06 g, 0.11 g and 0.28g. Fig. 9 presents plots of the excess pore pressure as a function of depth at various times during the 0.28 g shaking event for (a) the treated and (b) the untreated slopes. While liquefaction (excess pore pressure equal to the initial vertical effective stress line) was produced in the untreated slope, excess pore pressures were reduced by the presence of the drains. However, the drainage appears to have been more effective restricting excess pore pressures in the lower half of the profile than in the upper half.

Fig. 10 provides plots of (a) the horizontal settlement and (b) the vertical settlement of the treated and untreated slopes for the series of tests with various peak ground acceleration levels. Although the vertical drains were not successful in eliminating all movement, they were effective in reducing horizontal displacements to about 20% of those for the untreated slope and vertical settlements to about 50% of the untreated slope for the acceleration levels involved.

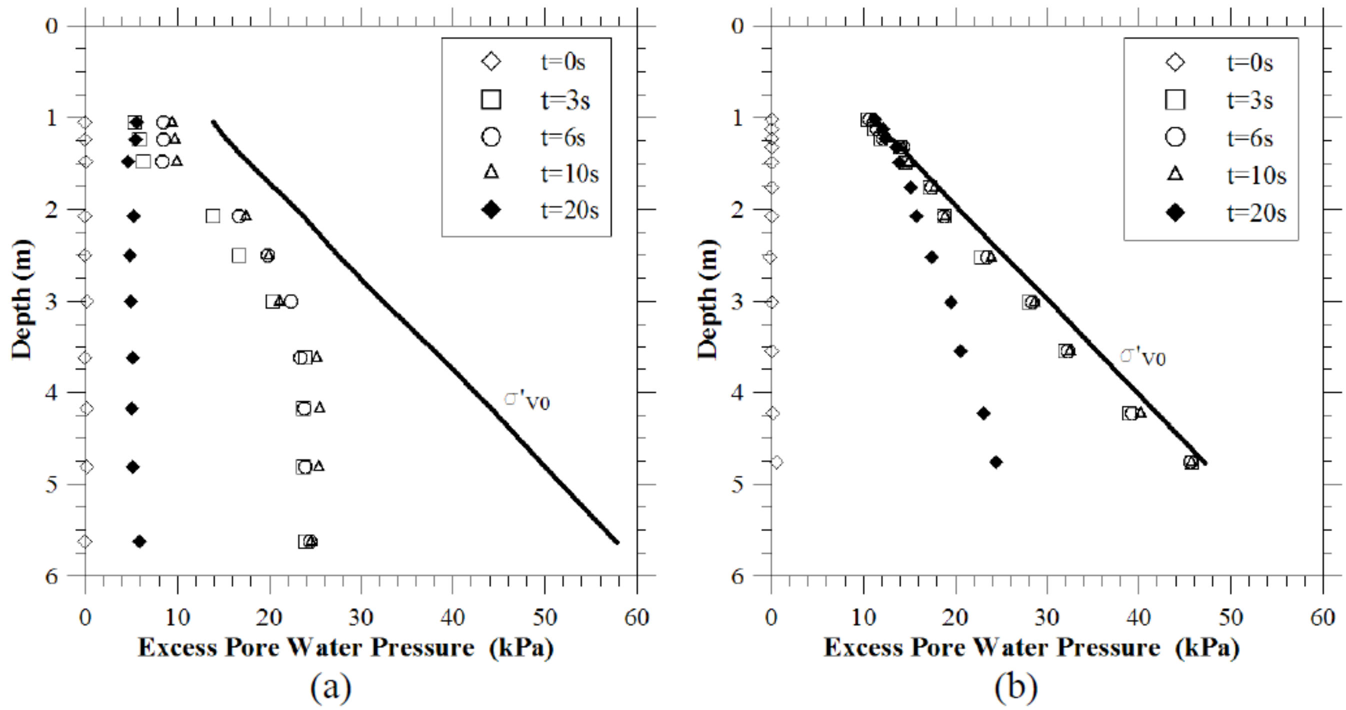


Figure 9. Excess pore water pressure profile for varying times for PGA=0.28g event for (a) treated and (b) untreated sides. (Note: "t=0s" corresponds to the start of shaking.) (Marinucci et al 2008)

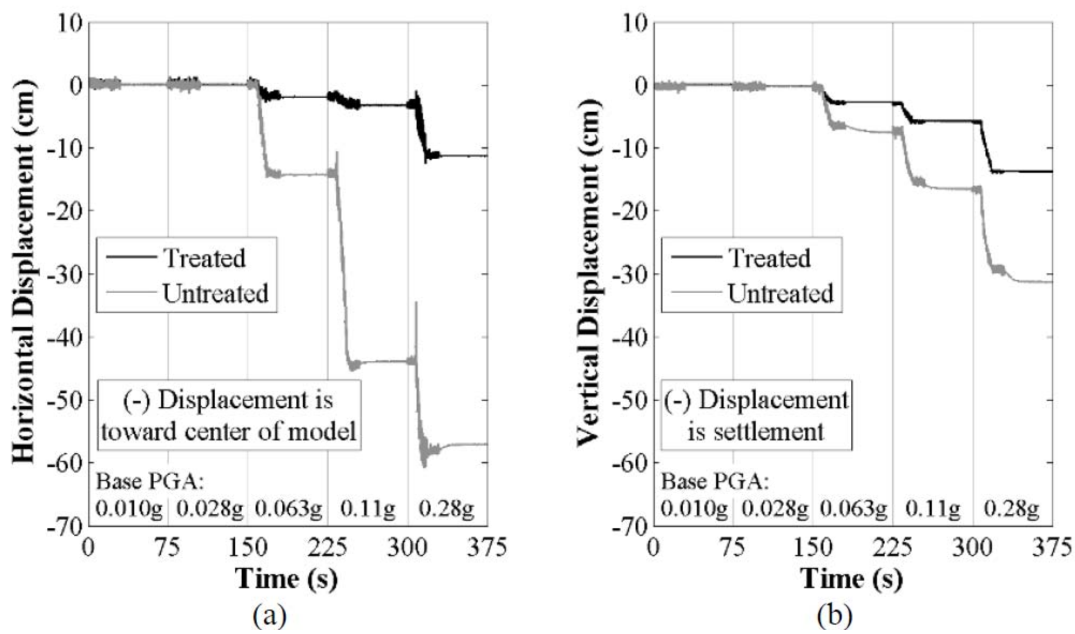


Figure 10. Shaking-induced deformation: (a) horizontal and (b) vertical directions for untreated and treated slopes. (Marinucci et al, 2008).

The third centrifuge test reported by Howell, et al. (2012) consisted of three treatment areas: one untreated, one untreated but containing non-draining tubes (to confirm soil pinning was not an issue), and one drain treated. Fig. 11 shows the plan view and the half of the cross section that contained the vertical drains, although the other half would mirror it to the right minus the drains. This test had a steeper slope (10° rather than 3°) and a thicker clay layer (1.5 m) over the liquefiable sand zone which was 5.5-m thick with a relative density of 40%.

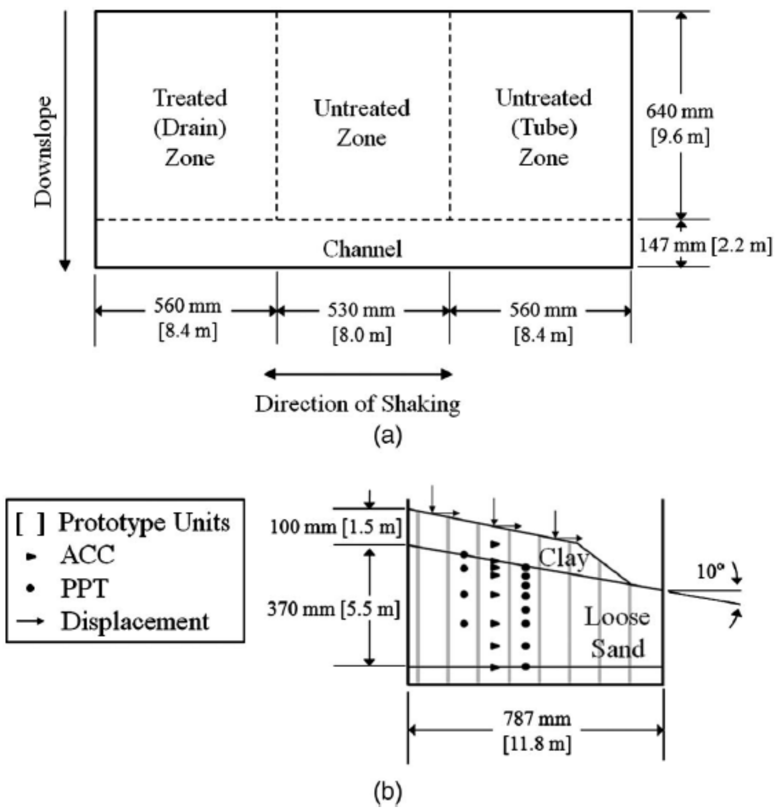


Fig. 11. Layout for centrifuge test comparing behavior of slopes in liquefiable sand with and without vertical drains to mitigate liquefaction hazard: (a) plan; (b) cross section (Howell et al. 2012).

Water was used as the pore fluid for these tests out of concern about how well a more viscous fluid would flow through the model drains, which also meant that scaling laws for diffusion and dynamic response were not simultaneously satisfied (Kutter 1995). Scaling laws still apply for dynamics, and the hydraulic conductivity of the fine Nevada sand can be scaled upward by a factor of 15 to correspond to values typical of medium to coarse sands. The vertical drains were spaced at 1.5 m center to center. The slopes were subjected to progressively higher accelerations levels ranging from 0.10g to 0.95g. No appreciable difference was observed between the performance of the untreated slope and the slope with non-draining tubes so the effect of pinning was considered to be inconsequential.

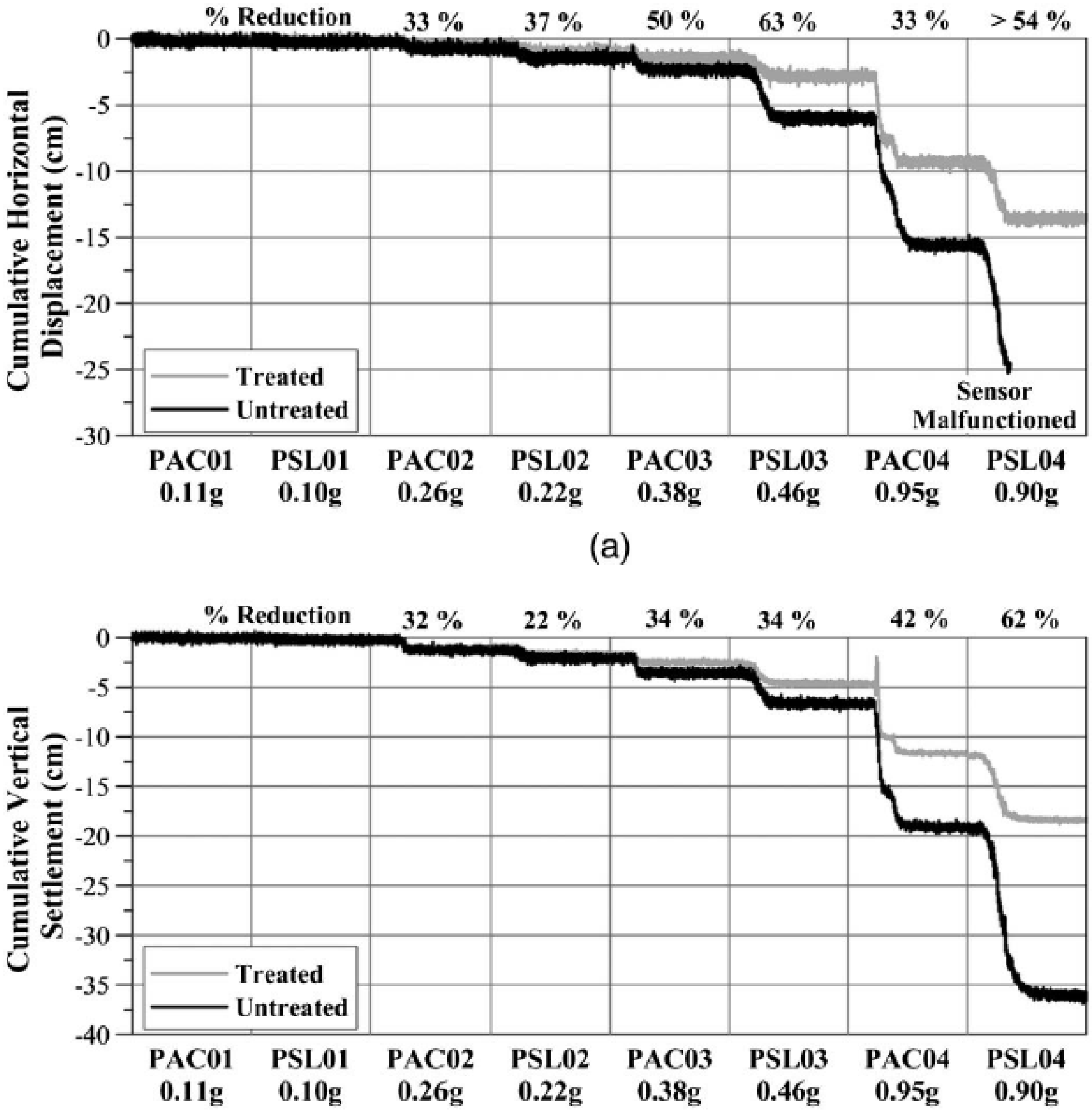


Fig. 12. Cumulative (a) horizontal and (b) vertical displacements at mid-slope in the untreated and treated areas for all shaking events (After Howell et al, 2012).

The vertical drains were effective in reducing the measured deformations during shaking by dissipating the excess pore water pressures both during and after the shaking event. Plots showing the cumulative (a) horizontal and (b) vertical displacements for the treated and untreated slopes for the various events are shown in Fig. 12. The percent reduction in settlement is summarized in Fig. 12 along with the peak ground acceleration (PGA) of the earthquake event. The reduction in settlement for the treated slopes with drains was typically 30% to 60% of that for the untreated slopes without drains.

As indicated in Fig. 12, there was often a significant variation in the reduction in deformation obtained for various records and acceleration levels. As shown in Fig. 13, Howell et al (2012) found that much of this variation could be explained by plotting the displacement as a function of the elapsed by between the first and last exceedance of r_u=0.5. Therefore, the longer the soil remained in a quasi-fluid state the greater the horizontal and vertical settlement for a given soil profile. This find demonstrates the importance of vertical drains in reducing the potential for settlement and lateral spreading.

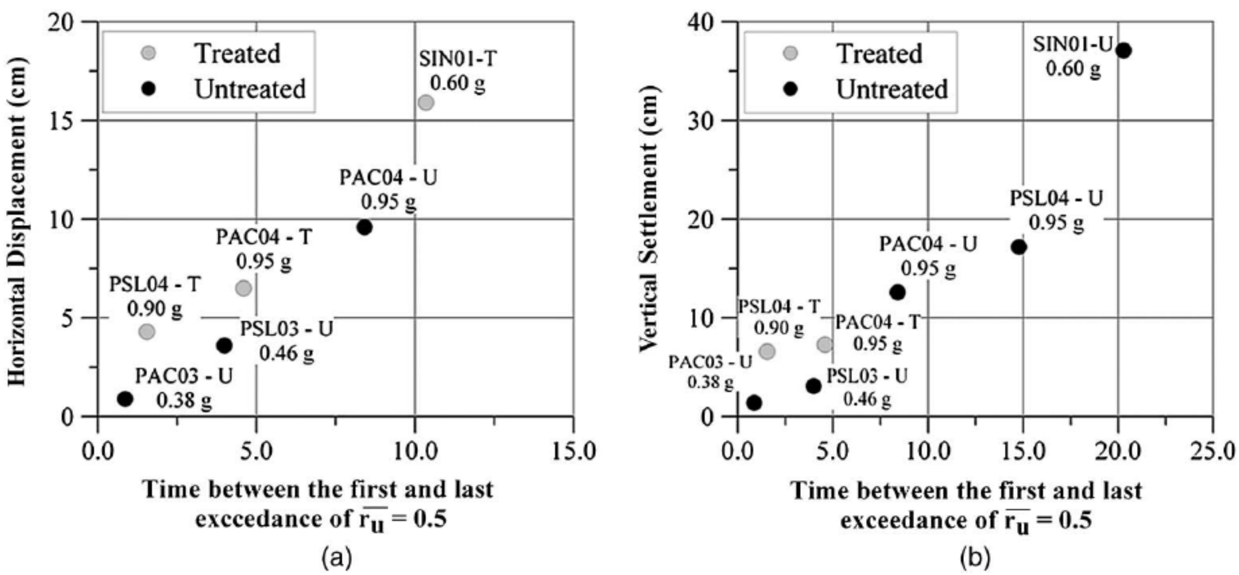


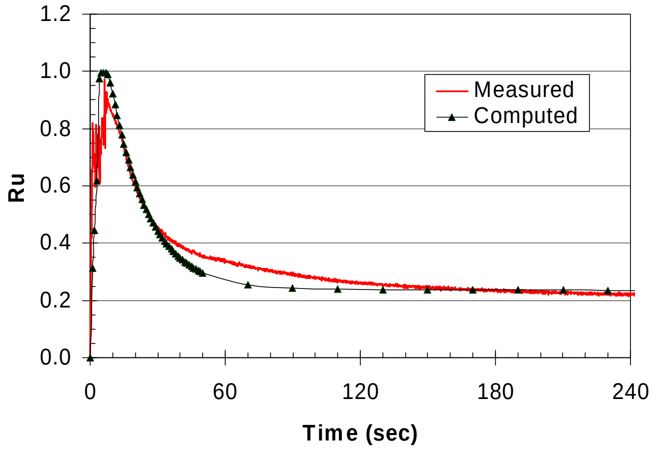
Fig. 13. Horizontal and vertical deformations at midslope in the untreated and treated areas: (a) horizontal displacement, (b) vertical dispalcements as a function of time between the first and last exceedance of r_u=0.5. (Howell et al. 2012).

Numerical Analyses Conducted to Evaluate Liquefaction Remediation with Drains

Because the blast testing approach produced liquefaction much more rapidly than an earthquake, there was less time for pore pressure dissipation and the effectiveness of drains in an earthquake might be obscured. For example, 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, Rollins et al (2004) performed numerical analyses using the computer program FEQDrain (Pestana et al, 1997). The computer model was first calibrated using the measured settlement and pore pressure response 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. In addition, calibrated parameters were within the range of measured values. Fig. 14 presents a plots showing (a) the computed and measured R_u vs time curves and (b) the 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 based on magnitude and guidelines suggested by Youd et al (2001). 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.



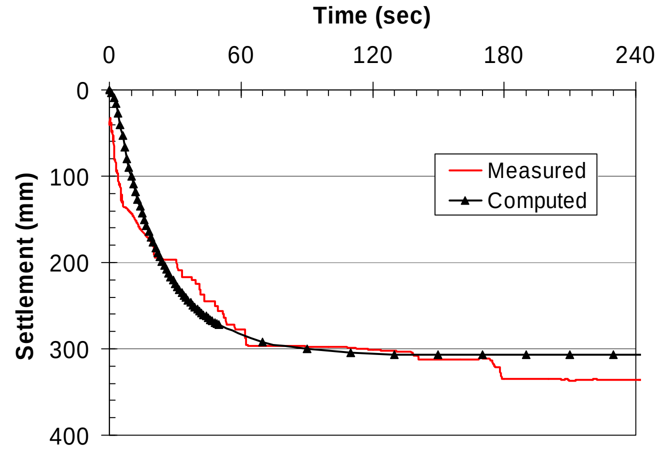


Fig. 14 Comparison of (a) measured and computed excess pore pressure ratio (R_u) versus time at a depth of 11.8 m and (b) measured and computed settlement versus time curves for the Vancouver test site. (Rollins et al, 2004).

Table 3 Summary of computed maximum r_u and settlement for various earthquake events and drain spacings at the Vancouver site.

Magnitude	Duration (sec)	N_q/N_l	Drain Spacing (m)	Maximum. 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

Recent numerical simulations by Vytiniotis et al. (2013) compared slope deformations with and without EQ drains for 58 reference seismic ground motions. Using finite element software a model was created to simulate boundary conditions and ground motions to evaluate EQ drain effectiveness in reducing earthquake-induced permanent slope deformations for a partially submerged saturated sandy slope. The geometry of the model is shown in Figure 15. One of the key findings is that EQ drains show no correlation to the Arias Intensity, meaning that EQ drains will be similarly effective under different acceleration time-histories. The numerical simulations also demonstrated that EQ drains are effective in reducing earthquake-induced permanent slope deformations for sloped, loose granular, liquefiable soils such as are commonly found in U.S. ports. Though the EQ drains are behind the crest of the partially submerged slope, they reduce slope deformations by prohibiting the diffusion of excess pore pressures from the far field to the slope.

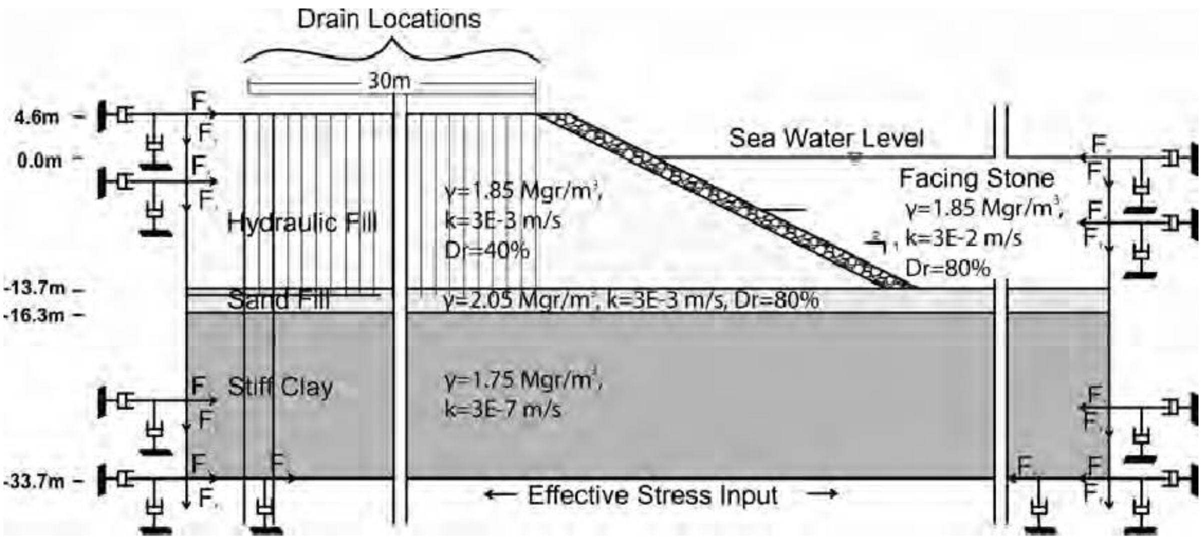


Figure 15. Analyzed section with details of the properties of the finite element numerical model (Vytiniotis et al. 2013).

Limitations of Previous Studies

While the previous studies clearly highlight the potential effectiveness of earthquake drains, they are all limited in one way or another. Moreover, these limitations represent a significant impediment to the implementation of drainage as a more routine mitigation strategy. For example, the blast liquefaction testing involves native sand under full-scale conditions, but the blast charges produce a very intense dynamic load that is applied much more rapidly than an earthquake and it is difficult to translate the observed performance during blasting to a magnitude and peak acceleration for earthquake conditions.

Centrifuge testing can simulate realistic earthquake shaking conditions; however, similitude issues are always a concern and it is difficult to reproduce the aging and natural “structure” of sands in the field. As a result, flow failures which have been observed in nature have not been observed in centrifuge tests, even with very loose sand, without placing low permeability layers. In contrast, very dense sands have experienced liquefaction and exhibited significant settlement in centrifuge tests while this poor performance has not been observed in nature (Knappett and Madabhushi, 2008). These departures from field performance make it difficult to directly apply results from centrifuge tests to design practice. Furthermore, in the centrifuge tests involving drains reported by Marinucci et al. (2010) and Howell et al (2012), water was used as the pore fluid so that the permeability of the sand under prototype conditions was equivalent to that of coarser sand. Performance could be considerably different for sand with a permeability 50 to 100 times lower.

The numerical simulations by Vytiniotis et al. (2013) and Rollins et al (2004) demonstrate the effectiveness of EQ drains in reducing deformations for partially submerged saturated sandy slopes, but further field testing is needed to validate the models. While the tests with the Vibroseis trucks involved full-scale conditions, the sand thickness was limited to 1.5 m and induced shear strains were so low that 40 strain cycles were required to induce liquefaction. For higher strain levels, more typical of earthquake shaking, and thicker zones of potentially liquefiable sand typical of many field sites, the drain performance would be expected to be less robust. In the full-scale field tests by Marinucci et al. (2010), breaking of cementation bonds within the sand during drain installation resulted in significant settlement, making it hard to isolate the effect of densification from the presence of the EQ drains.

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