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Liquefaction Mitigation Potential of Prefabricated Vertical Drains from Large-Scale Laminar Shear Box Testing

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Abstract. Although small-scale testing suggests that vertical drains can be effective in mitigating liquefaction induced pore pressure and displacements, no full-scale drain installation has been subjected to an earthquake. To address this problem, full-scale tests with vertical drains were conducted in loose liquefiable sand using a 6 m high laminar shear box and high-speed shaking system. Tests involved 75 mm diameter slotted plastic drain piles at 0.9 m center-to-center spacing. The sand was deposited by water pluviation to a relative density between 35 to 45%. Base input motions consisted of 15 sinusoidal cycles with peak accelerations increasing from 0.05g, to 0.10g, to 0.20g. In contrast to tests with untreated sand, which liquefied completely after only a few cycles, the drains were successful in increasing the number of cycles to liquefaction. In addition, pore pressure dissipation at the end of shaking was markedly increased. While liquefaction was not prevented in all cases, the ground surface settlement was reduced by 40 to 60% relative to that for the untreated sand case. This result is in agreement with previous centrifuge and blast liquefaction field tests.

Keywords. Liquefaction, vertical drains, laminar shear box tests, liquefaction hazard mitigation.

1. Introduction

Liquefaction of loose saturated sand results in significant damage to infrastructure 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 over $11.8 billion in damage just to ports and wharf facilities in the 1995 Kobe earthquake [1], The loss of these major port facilities

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subsequently led to significant indirect economic losses. In the Christchhurch New Zealand earthquake sequence liquefaction resulted in about $10 billion damge (Sjoerd van Ballegooy, personal commuication).

Typically, liquefaction hazards have been mitigated by densifying the soil in-situ using techniques such as vibrocompaction, stone columns, compaction grouting, dynamic compaction, or explosive compaction. 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 [2].

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 [3] as illustrated in Figure 1. Vertical drains can relieve these pressures, prevent the formation of a water interlayer, and reduce the potential for lateral spreading and slope instability.

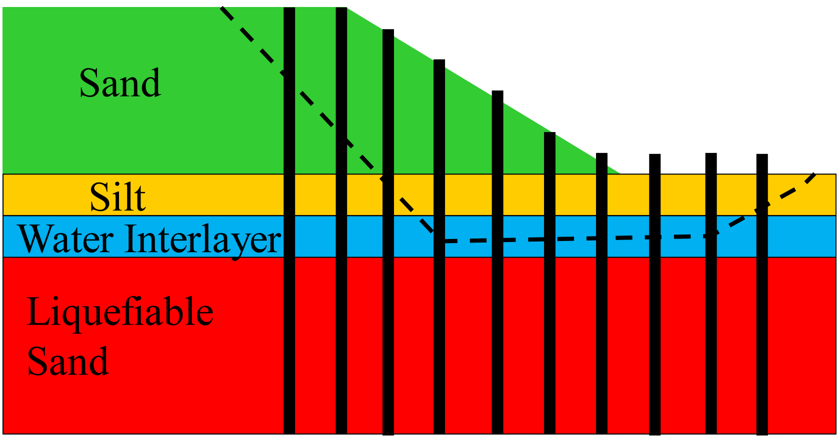

Figure 1. Illustration of the use of pre-fabriacated vertical drains for mitigating development of water interlayer below a low-permeability layer to prevent slope failure.

Unfortunately, no field performance data is available to show how vertical drains actually behave when subjected to earthquake motions. This lack of performance data under full-scale conditions is an impediment to expanding the use of this technique. In the absence of earthquake performance data, investigators have used a number of methods to investigate the effectiveness of vertical drains. These methods include: field tests involving controlled blasting [4] or vibrations [5] to induce pore pressure, centrifuge testing with scaled models that are accelerated to simulate the stress levels existing under field conditions [6, 7], and numerical methods [8].

Rollins et al. (2004) [4] report results from full-scale tests involving blast liquefaction at sites on Treasure Island in San Francisco Bay and at the Massey tunnel portal, south of Vancouver, Canada. Installation of the drains with a finned vibratory mandrel produced about 27 cm of settlement over a 7.6 m length at Treasure Island (3.6% volumetric strain), while installation to a depth of 12.8 m at Vancouver (10.4 m of liquefiable sand) produced 35 cm of settlement (3.4% volumetric strain). The initial relative density at these sites was about 50% and 40%, at Treasure Island and Vancouver, respectively.

Blast tests were subsequently performed at sites with and without drains. At Treasure Island, maximum settlement in the area with drains was reduced to about 3 cm relative to 8 cm at the site without drains, a settlement reduction of about 60%. At Vancouver, the settlement profiles from the center of the test area extending radially outward are compared in Figure 2. Maximum settlement was reduced from 50 cm in the untreated area compared to about 30 cm in the area with drains, a settlement reduction of 40%.

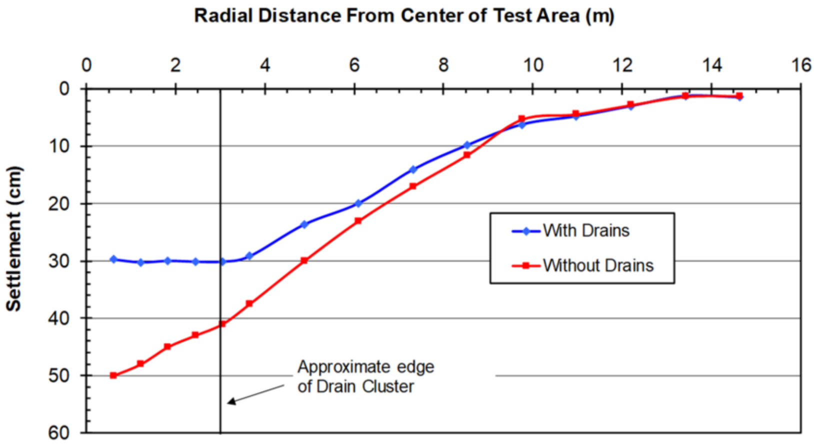

Figure 2. Comparison of liquefaction induced settlement profiles in adacent test areas with and without vertical drains at Vancouver, British Columbia, Canada following blast testing [4].

In centrifuge tests, Marinucci et al. [6] used models with and without drains to evaluate drain perofrmance with peak base accelerations ranging up to 0.28g. The presence of the vertical drains typically reduced the measured settlement by 40 to 50% relative to the models without drains. Marinucci et al. [6] found that the drains were more effective in reducing excess pore pressure ratios at deeper depths (≥ 3 m) than at shallow depths. They also found that the the settlement was correlated with the time the excess pore pressure, R_u remained above 50%. Apparently, the longer the R_u value remains high the sand will have more time to deform and strain.

While each of these methods can be used to obtain useful information, they both have limitations. For example, the blast testing does not simulate the shaking that occurs in earthquakes, while centrifuge testing involves scaling which can sometime lead to uncertainty. Therefore, a full scale test is desirable to validate the results found in the other tests which are analogues of the actual field conditions. To address this problem, full-scale tests with vertical drains were conducted in loose liquefiable sand using the 6-m high laminar shear box and high-speed actuator system at George E. Brown, Jr. NEES facility at the Univ. at Buffalo in New York. This paper describes the testing details, the basic test results, analysis results, and the conclusion obtained in this study.

2. Test Layout and Instrumentation

Plan and profile views of the laminar shear box are presented in Figure 3. The laminar box consists of 40 stacked rectangular rings with dimensions as shown in Figure 3. Each ring is 15 cm tall and is supported by a series of roller bearings which allows each ring to move independently in the horizontal direction so that the movement was largely controlled by the mass of the soil inside the box. Therefore, the soil had the potential to respond as it might in the field during earthquake shaking. Two flexible rubber membrane liners were placed inside the laminar box to allow the sand to be saturated and undrained during the cyclic loading. Acceleration time histories were imposed on the bottom of the laminar box using two high-speed hydraulic actuators with a hydraulic accumulator system.

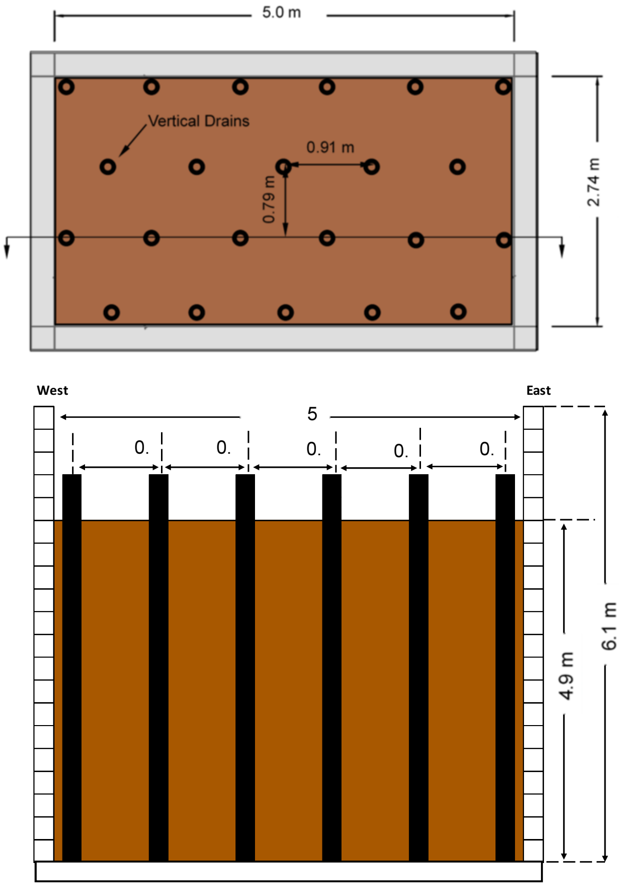

Figure 3. Plan and profile views of the laminar shear box with vertical drains in triangular pattern spaced at 0.9 m on centers.

Prior to sand deposition, vertical drains were hung from a frame above the box and tied to a PVC grid at the bottom. As shown in Figure 3, the drains were placed in a triangular pattern with a spacing of 0.9 m on centers. Therefore, there was no densification associated with drain installation as would be the case for drains in the field that are installed using a vibratory mandrel as discussed previously. The drains were 75 mm diameter corrugated plastic pipes with an outside diameter of approximately 94 mm that was surrounded by a filter fabric "sock" to prevent infiltration of sand. The drains were cut-off about 40 mm above the ground to allow free drainage.

The sand in the laminar shear box was deposited by water pluviation to a depth of about 4.9 m. The sand was pumped from containers in a saturated state and deposited into standing water at a height of about one meter using a spreader. The sand was a poorly graded clean sand (Ottawa F55) with a mean grain size of 0.23 mm and a coefficient uniformity of 1.52. Small buckets placed in the sand during deposition indicated that the relative density was initially between 25 and 30% as shown in Figure 4. Downhole permeability tests indicated that the horizontal hydraulic conductivity in the sand ranged from 0.03 to 0.05 cm/sec with a slight decrease with depth.

Nine shaking tests were performed on the laminar shear box. Tests were performed in three rounds with peak accelerations of 0.05g, 0.10g, and 0.20g for each round. A peak acceleration of 0.20g was the highest acceleration permitted by the NEES@UB lab. Figure 4 provides plots of the planned input base time histories. All motions consisted of 15 cycles of sinusoidal motions with a frequency of 2 Hz. Typically, 15 cycles of motion are associated with a Magnitude 7.5 earthquake [9], which is often used as the base magnitude for liquefaction studies. A ramp-up and ramp-down period was used to be consistent with previous tests at the site that were used for comparisons.
Excess pore pressure was measured with three vertical arrays of pore pressure transducers spaced at approximately 1.5 m depth intervals. These arrays were located at mid-points between drains where the drainage effect would be lowest.

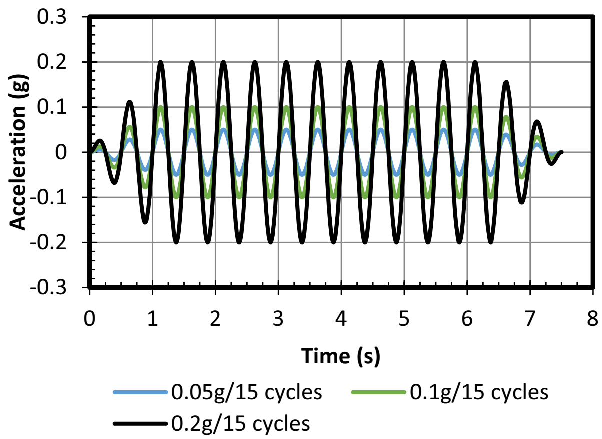

Figure 4. Base input motions for each round of shaking.

Sand settlement was measured by string potentiometers anchored to plates the ground surface along with Sondex "profilometers" which gave settlement versus depth profiles with measurements at 60 cm depth intervals. In addition, total ground settlement was computed by dividing the water volume expelled in each test by the surface area.

3. Test Results

Within seconds after shaking began in each test, water began flowing from each drain pipe and continued to flow for a few seconds after the completion of shaking. Average flow volume per drain with full drainage area was 5.5 to 8.2 liters/s for the first round of testing and decreased by about 40% for each round or testing. In contrast to other tests without drains, there were no sand boils at the surface and the majority of flow appeared to be coming from the drains themselves, although some flow could have come through the soil profile iteself.

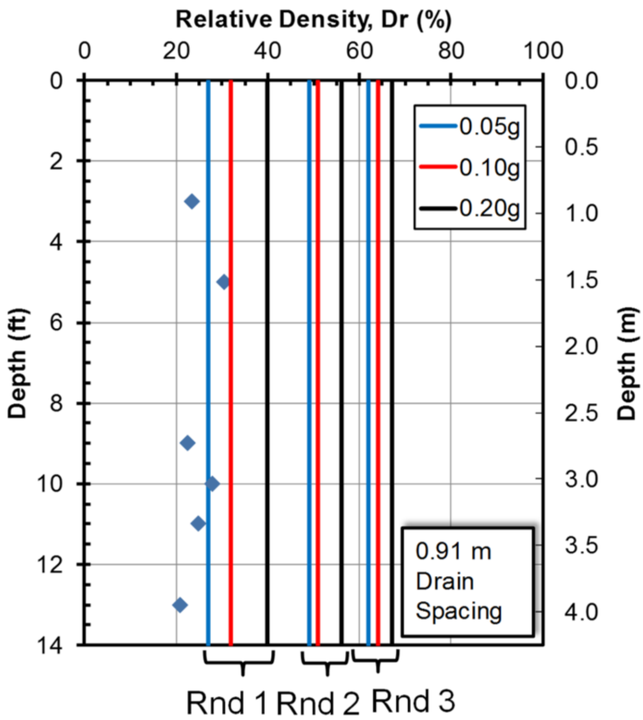

Figure 5. Average relative density (D_r) versus depth for each round of testing.

3.1. Excess pore pressure behavior.

Previous laminar shear box testing, with the sand at a similar relative density but without drains, showed that the sand would liquefy within four cycles of loading with a peak acceleration of 0.05 g [10]. In addition, after shaking, excess pore pressure ratios, required 60 to 120 seconds or more to dissipate below 20%.

Excess pore pressure ratio time histories are provided at six depths for the second round test subjected to a peak acceleration of 0.10g in Figure 6. With drains in place, excess pore pressure ratios did not typically reach 100%, particularly at greater depths. In contrast to the tests without drains, the excess pore pressure ratios immediately decreased at the completion of shaking and excess pore pressure ratios were typically less than 20% within a few seconds after the end of shaking. These results demonstrate that the drains are significantly decreasing the rate of pore pressure generation and increasing the rate of dissipation.

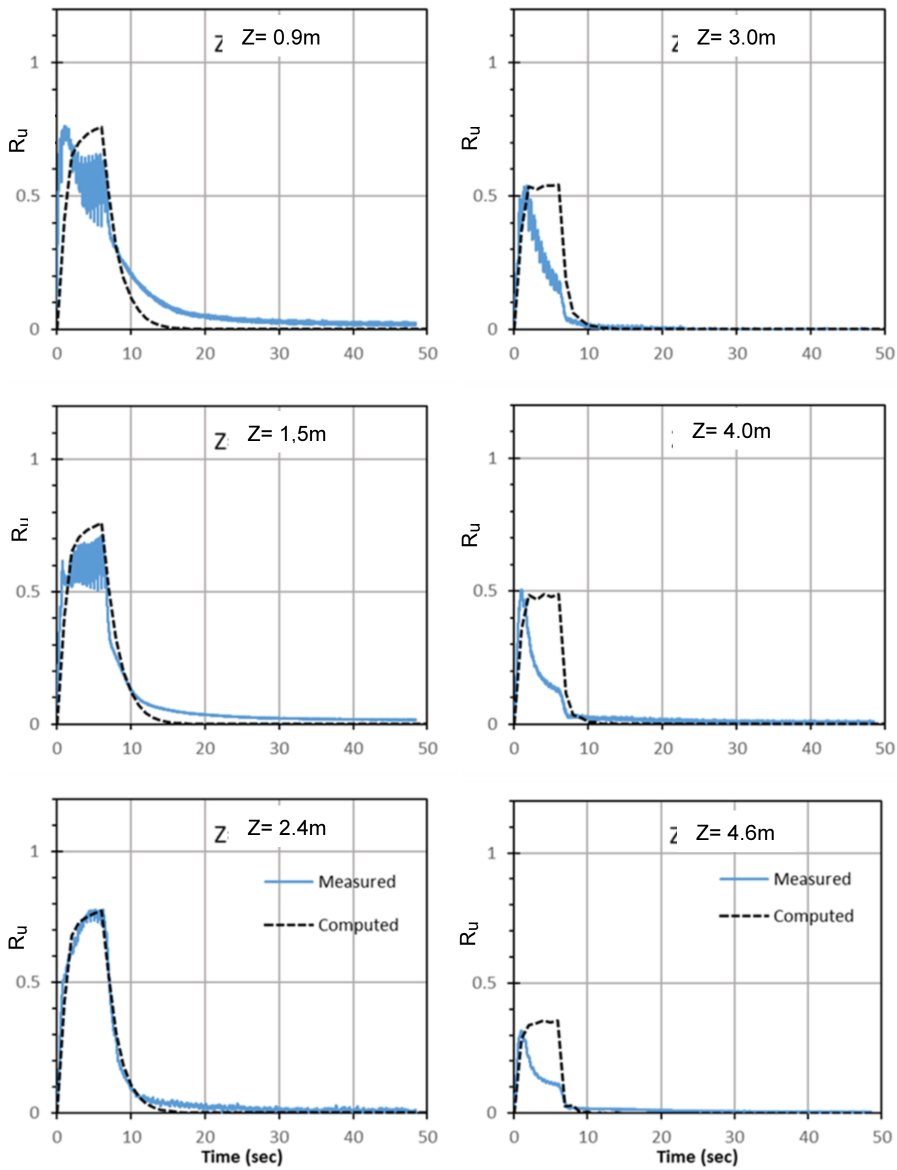

Figure 6. Meaasured and computed Ru versus time curves for six depths during the second round of testing with peak input motion of 0.10g.

The peak excess pore pressure ratio, R_u, is plotted as a function of depth for each shaking test (namely, 0.05, 0.1, and 0.2g) for each round of testing in Figure 7.

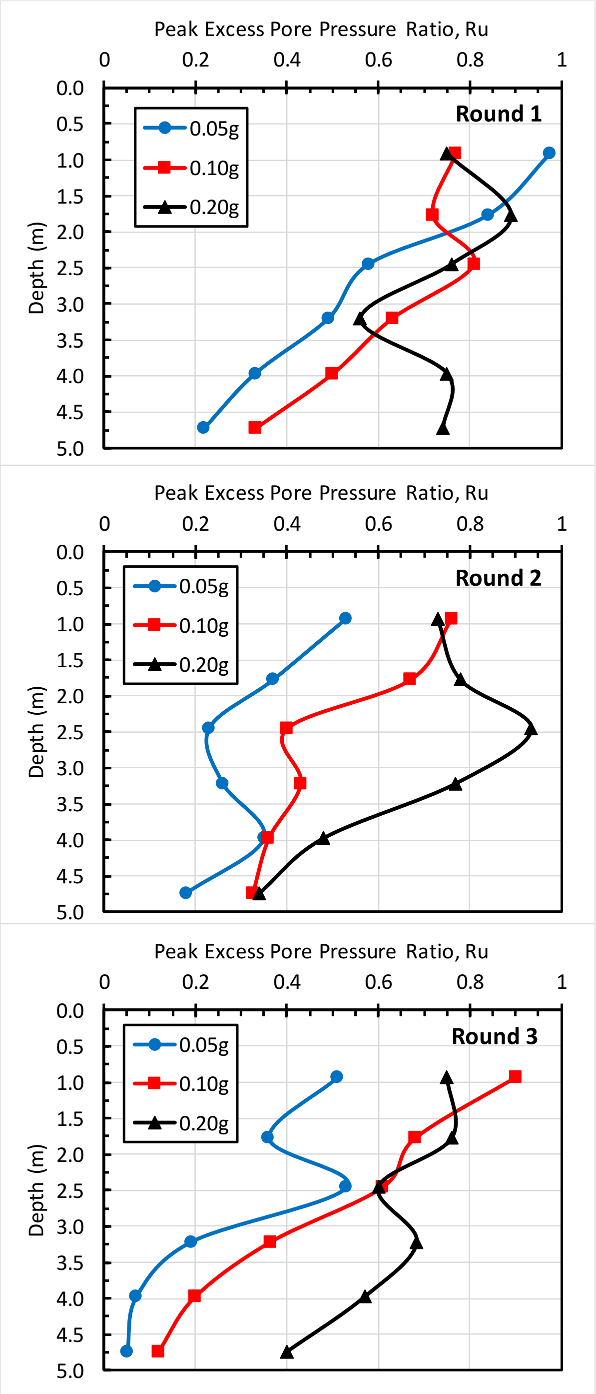

Figure 7. Peak excess pore pressure ratio for three arrays vs. depth plots for round 1, 2, and 3 test with 0.05, 0.10, and 0.20g base input motions per round. Peak values are averages at each depth for three transducers [11].

Of course, ratios closer to the drains would likely have been lower. Typically, the peak R_u decreases with depth despite the fact that without drains, the entire sand layer within just a few cycles at 0.05g acceleration. This result indicates that the drains are being effective in reducing excess pore pressure; however, they are increasingly effective with depth. This confirms results observed in centrifuge tests with drains [6, 7]. Thefore, some combination of surface densification and drainage might provide an optimum solution. In this regard, the densification produced by drain insertion in field applications could be particularly beneficial.

With each progressive round of testing, the peak excess pore pressure ratio generally decreased. Although this effect may be partially attributable to increasing density, analyses of the development of pore presure with respect to induced shear strain suggest that this a beneficial effect of drainage. At low relatively densities, the drains may not have sufficient capacity to prevent liquefaction, but their effect may become more significant as density increases.

3.2. Settlement Behavior

With drains, about 80% of the ground settlement occured during shaking and 95% occurred after 5 seconds of additional pore pressure dissipation. Within each round, settlement generally increased with acceleration, while settlement decreased with each round of testing as the sand compacted.

Figure 8 provides a comparison of the cumulative settlement as a function of the number of tests for the nine tests with drains spaced at 0.9 m, in comparison with previous studies without drains. All settlement has been scaled to match a soil depth of 4.9 m, where necessary. The LG1 test involves repeated tests with a peak acceleration of 0.1g with an untreated sand layer 4.9 m deep (Thevanayagam, personal communication). The IPS1 test result involved testing with induced partial saturation treatment in a 3-m thick layer of the total 4.9 m thickness [12]. Settlement has been scaled up proportionally based on measurements showing that the untreated sand was responsible for the vast majority of the settlement. In the IPS1 testing, the sand was subjected to 0.1g acceleration for six tests and 0.2g acceleration for two tests. Significantly more settlement was observed for tests where 0.2g accelerations were applied.

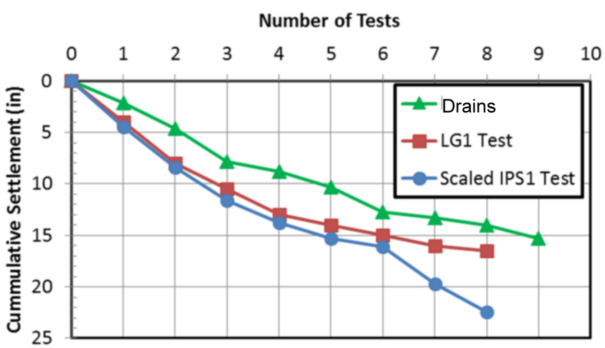

Figure 8. Cumulative settlement versus number of shaking tests in laminar box with drains in comparison with laminar box without drains (LG-1 and IPS1 Test) [12] (Thevanayagam, personal communication, 2015).

Although the comparisons are not perfect nor completely direct because of differing acceleration levels, using 0.05g, 0.1g, and 0.2g accelerations with the drains, it is clear that the cumulative settlement curve for the profile with vertical drains is significantly lower than without. In many cases the settlement is only 60 to 70 percent of the settlement for the untreated sand. Considering that the untreated test curves are generally for 0.1g, while the treated sand experienced three 0.2g cycles, the reduction produced by the drains may be even greater than represented in Figure 8.

4. Analysis of Test Results

The performance of potentially liquefiable layers with prefabricated vertical drains has commonly been analyzed using the computer program FEQDrain [13]. FEQDrain uses an axi-symmetric finite element model of the soil profile and composite drain system. The program models an individual drain within a grid of drains using a "radius of influence" concept based on the drain spacing, also called a "unit cell" model. The program computes the excess pore pressure ratio for each soil layer within the radius of influence. It does this by computing how much pore pressure is generated by the earthquake shaking in a given time interval and then subtracting how much pressure has dissipated from water flowing to the drain in the same time interval. Key parameters in this analysis are the hydraulic conductivity of the sand in the vertical and radial directions along with the initial modulus of compressibility (m_{vo}) of the sand.

The program is able to compute head loss in the drain and considers storage volume of the drain. The model also accounts for non-linear increases in the modulus of soil compressibility resulting from elevated pore pressure and includes a pore pressure generation model based on the number of cycles to cause liquefaction proposed by Seed et al. [14]. Therefore, the user must specify the number of cycles required to cause liquefaction without drains, the number of cycles produced during shaking, and the duration of shaking. Besides calculating pore pressure response, the program can also calculate settlement associated with drainage and dissipation of pore pressure.

The hydraulic conductivity was relatively well-defined versus depth based on borehole permeability testing at the beginning of each round of testing and was found to decrease with increasing relative density. However, estimation of the initial modulus of compressibility was more problematic. Initial analyses using the initial m_{vo} values recommended by Pestana et al. [13] produced poor agreement with the measured pore pressure response. In addition, the recommended moduli were too stiff based on liquefaction induced settlement models (e.g. [15]). Therefore, correlations based on liquefaction induced settlement and relative density were used to make initial estimates of m_{vo}. These values generally fell within the center of the range of m_{vo} values back-calculated by field measurements.

Plots of computed R_u versus time are also shown in comparison to the measured curves in Figure 6 and the agreement is generally reasonable. However, at greater depths ($\geq$ 3 m), the drains appear to be functioning better in dissipating pore pressure than would be expected based on the computer model. Despite the good agreement, design of drains in practice would require the appropriate consideration of the variability in hydraulic conductivity and compressibility which are often difficult to quantify.

The settlements computed by the model were generally over-predicted by approximately 50%; however, this level of agreement is consistent with agreement between many liquefaction-induced settlement prediction equations and measured settlements [16].

Parametric analyses using the computer model indicate that the relatively rapid loading rate (15 cycles in 7.5 seconds) in the laminar shear box testing produced a rather severe test for the drains in comparison to a M7.5 earthquake where duration might be on the order of 40 seconds [9]. Analyses with 15 cycles over 40 seconds (for M7.5 earthquake) indicated significantly lower peak excess pore pressure ratios and lower settlements than for the laminar shear box tests. In contrast, reducing the hydraulic conductivity by 50% produced a significant increase in peak excess pore pressure ratio and settlement.

5. Summary and Conclusions

A series of shaking tests were performed with a 6-m tall laminar shear box in which 75 mm diameter vertical drains were placed in a triangular arrangement with a center-to-center spacing of 0.9 m. Shaking consisted of 15 cycles of loading with peak accelerations of 0.05, 0.10, and 0.2g. Tests were performed to evaluate the abiltiy of the drains to reduce pore pressure and settelement in comparison to sand without drains. The sand in the box was initially deposited at a relative density of about 25% but increased with repeated shaking.

Based on the results of the laminar shear box testing the follwing conclusions can been made:

1. Vertical drains were effective in reducing the peak excess pore pressure ratios relative to tests without drains. The drains were more effective at greater depths (≥ 3m).
2. The drains carried large volumes of water to the surface and eliminated sand boils observed in tests without drains. The rate of pore pressure dissipation was significantly increased with drains and excess pore pressure ratios typically dropped below 0.2 within a few seconds after the end of shaking.
3. Although the drains did not eliminate pore pressure induced settlement, the settlement was reduced by about 30 to 40% of the settlement without drains. This reduction is similar to that observed in centrifuge testing with vertial drains [6 and 7] and blast induced liquefaction test [4].
4. Numerical analyses can provide reasonable estimates of pore pressure generation and dissipation with proper consideration of hydraulic conductivity and sand compressibility. However, design should include appropriate consideration of potential variability in these parameters which are often difficult to quantify.

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