EFFECTS OF DRAINS ON THE PERFORMANCE AND DAMAGE POTENTIAL OF SHALLOW-FOUNDED STRUCTURES

Balaji Paramasivam, Shideh Dashti, Abbie Liel, Juan Olarte
Department of Civil, Environmental and Architectural Engineering - University of Colorado Boulder, Boulder, Colorado, USA.

ABSTRACT
Prefabricated vertical drains (PVDs) are commonly used in practice to mitigate the liquefaction hazard and its consequences. However, the influence of enhanced drainage on ground motion characteristics, foundation settlement, building tilt, and the demand imposed on the superstructure is not well-understood. This shortcoming hinders the development of performance-based procedures for designing liquefaction mitigation strategies. This paper presents results of three centrifuge experiments conducted at the University of Colorado Boulder on 3-story and 9-story potentially-inelastic, moment resisting, steel frame, scaled model structures founded on a layered liquefiable soil deposit. The influence of enhanced drainage with PVDs on accelerations imposed on the foundation and superstructure, foundation settlement and tilt, roof accelerations, and deformation patterns within the beam and column fuses was investigated. The results of these tests indicate that PVDs can be successful in reducing the extent and duration of large excess pore pressures in the underlying soil and in reducing permanent foundation settlements. However, their influence on transient and permanent foundation tilt, as well as the seismic demand transferred to the superstructure and permanent roof drift depend strongly on the dynamic properties and yield characteristics of the structure. For example, installation of PVDs significantly reduced the transient and permanent tilt of the 3-story structure designed to remain essentially elastic, while they amplified foundation accelerations and hence, the strains on column fuses. On the other hand, when PVDs were employed around a taller 9-story structure designed with a lower yield strength, they greatly amplified inelastic deformations in the superstructure, which in turn further amplified foundation rotation in the direction of permanent roof flexural drift due to the P-Δ effect. The experimental results presented in this paper point to the importance of considering the combined properties and response of the soil-foundation-structure system when evaluating the effectiveness of liquefaction mitigation strategies in design.

1 INTRODUCTION

Recent earthquakes have provided many examples of the damaging effects of soil liquefaction on buildings. For example, after the 1999 Kocaeli (Turkey) and the 2011 Christchurch (New Zealand) earthquakes, shallow-founded structures on liquefiable soils suffered excessive permanent settlement and tilt, exceeding serviceability limits, and causing significant economic losses (Bird et al. 2004; Sancio et al. 2004; Cubrinovski and McCahon 2012). Mitigation techniques are often employed to avoid liquefaction or alleviate its effects. However, the influence of various mitigation techniques on ground motion characteristics and their subsequent effects on building performance and damage potential is still unknown, due to a limited number of well-documented case histories on mitigated sites with adequate instrumentation (Hausler 2002).

Geotechnical centrifuge modeling is a cost-effective tool for physically modeling and evaluating the response of complex soil-structure systems under realistic confining pressures. A number of researchers have previously conducted centrifuge experiments of liquefiable soils with either (i) shallow, rigid foundations alone, or (ii) model single degree-of-freedom (SDOF) elastic structures (Liu and Dobry 1997; Hausler 2002; Dashti et al 2010a,b). Some of these studies were extended to evaluate the effectiveness of specific mitigation techniques on soil and building response (Liu and Dobry 1997; Hausler 2002; Dashti et al 2010b). These previous studies aimed to simulate the building contact pressure and, in some cases, first-mode natural frequency of a realistic structure. Also, the mitigation techniques were mostly designed to limit excess pore pressure generation in the soil and ground deformations. The influence of liquefiable ground and mitigation techniques on the performance of more realistic, potentially-inelastic, damageable, multi-degree-of-freedom (MDOF) structures have not yet been modeled physically. A few centrifuge experiments with MDOF inelastic structures were conducted on dry sand, without liquefaction (Chen et al 2010; Mason 2011; Liu et al 2012).

In short, there is currently a lack of physical model studies to evaluate the effectiveness of mitigation techniques holistically, including the response of soil in building's vicinity, settlement and tilt of the foundation, the seismic demand on the superstructure, and the damage potential of inelastic MDOF structures. This shortcoming hinders the development of performance-based approaches for mitigation design.

In this research, a series of centrifuge tests was conducted at the University of Colorado Boulder (CU-Boulder) facility, to evaluate the performance and damage potential of 3-story and 9-story, potentially inelastic, moment-resisting, steel frame structures on liquefiable ground with and without a number of mitigation techniques. This paper discusses the preliminary results obtained from the centrifuge tests on the two structures when mitigated with prefabricated vertical drains (PVDs). PVDs (or earthquake drains) are alternative to traditional stone columns and are used to limit the extent of pore pressure generation and liquefaction (Rollins et al 2003; Howell et al 2012). PVDs are made of hollow perforated plastic pipes with an internal diameter of 75-200mm (Rollins et al 2003; Rollins et al 2004; Howell et al 2012). The perforated plastic pipes are wrapped with geotextile to avoid clogging by transportation of fines from the surrounding soil. Previous full-scale and centrifuge studies on PVDs (often in the absence of structures) showed good performance in terms of speeding up the dissipation of excess pore pressures within their zone of influence, which consequently reduced vertical settlement and lateral spreading (Rollins et al 2004; Howell et al 2012). In this study, the PVDs were used around the perimeter of two MDOF model structures. The influence of PVDs on excess pore pressure generation, settlement, tilt, foundation accelerations, and deformation patterns at the beam-column connections of the structures are presented and discussed in this paper in order to provide insight into the effectiveness of PVDs for systems-level building performance.

2. CENTRIFUGE EXPERIMENTAL SETUP

A series of dynamic centrifuge tests were performed using the 5.5m-radius, 400 g-ton centrifuge facility at CU-Boulder at 70g of centrifugal acceleration. Figure 1 shows the schematic of the test series presented in this paper and their instrumentation. Soil models with structures were constructed in a new flexible-shear-beam (FSB) container of length 968mm, width 376mm, and depth 304mm in model scale. Olarte et al. (forthcoming) detailed the design, construction, and the boundary effects of the FSB container. Different sand layers were dry pluviated to achieve the required relative densities (D_r) using an automatic sand pourer. Figure 1 shows a schematic of the soil profile, structures, and instrumentation. The bottom dense sand layer and the middle liquefiable layer were made of fine, clean, uniform Ottawa F-65 sand. The top non-liquefiable, dense crust layer was prepared using coarse Monterey 0/30 sand. The properties of the Ottawa and Monterey sands were discussed by Olarte et al. (forthcoming).

In order to evaluate the effectiveness of mitigation on structural performance, two potentially-inelastic, 3-story and 9-story moment-resisting steel frame structures were designed and modeled in this study. Due to the restrictions on the model container dimensions, shake table capacity, overhead space, and the objective to model two identical structures simultaneously in each test, building footings' plan dimensions were restricted to 9.6m x 9.6m in prototype units [136mm x 136mm in model units]. In this study, a 3-story structure (A) was modeled in centrifuge as a simplified, scaled, 3DOF structure; a nine-story structure (B) was represented as a scaled, 2DOF model structure, to capture its first two primary modes. Initially, the target prototype structures were designed based on the strength and drift requirements of AISC (2010) and ASCE-7 (2010). Then, their properties were converted to model scale units using the scaling factor (N = 70), as explained by Olarte et al. (forthcoming). The dead and live loads of the structure were modeled as lumped masses at respective floors. The beam and column sections of the structures were selected by scaling the moment of inertia of prototype beams and columns. The location of maximum moment and potential nonlinearity in the model structures was designed to occur at the reduced sections (fuses) located at beam ends and column bases. The replaceable fuses in the model structures were made of nickel to achieve the desired stiffness and strength. Beams, columns, and lumped masses were constructed of steel. Structure A was founded on a stiff mat foundation embedded at a depth of 1m from the soil surface in prototype units. Structure B was designed with a 1-story basement that was 3m deep from the soil surface in prototype units. The foundation and basement walls of the structures were made of aluminum to represent the approximate unit weight of reinforced concrete. Table 1 shows the properties of the model structures used in this study.

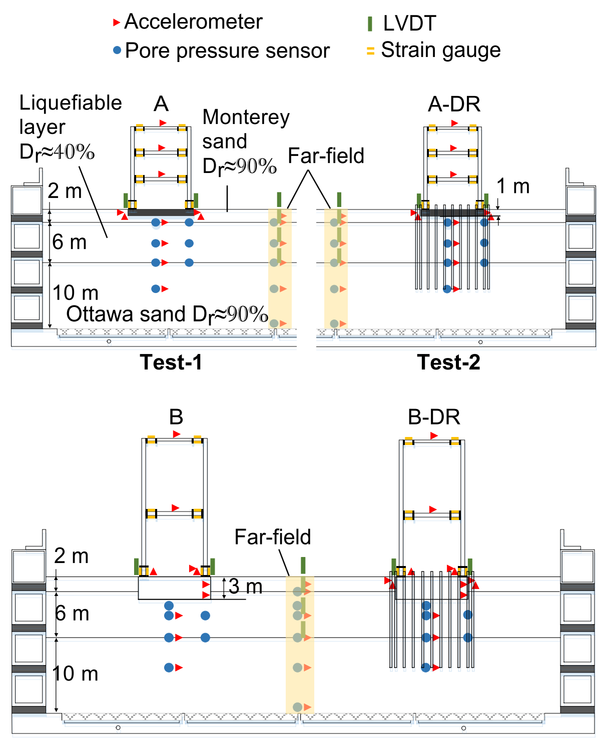


Figure 1. Schematic and instrumentation layout of centrifuge tests conducted at CU-Boulder

As shown in Figure 1, the liquefiable soil under Structures A-DR and B-DR (DR standing for enhanced drainage) was mitigated with PVDs. The PVDs were installed around the foundation of the structure at a center-to-center spacing of 1.2m in prototype units [17mm in model scale] to simulate drainage equivalent to that provided by realistic PVDs and to limit excess pore pressures (r_u) to 0.6, as is typical. The design and construction of model PVDs were explained by Olarte et al. (forthcoming).

Table 1. Properties of the model building structures

	Structure A	Structure B
No. of stories	3	9[1]
Height (H)/Width (B) ratio	1.8	2.5[2]
Ultimate base shear (V)/ Weight (W) ratio [3]	0.74	0.065
Natural periods (s)		
Fixed base [4]	0.58; 0.15; 0.06	2.82; 0.71
Flexible base [5]	0.53; 0.16; 0.07	2.66; 0.71
Fuse characteristics		
Cross-section dimension (width x depth)	0.532x 0.33m	0.532 x 0.22m
Yield stress (MPa)[6]	150	137
Yield strain (%)[6]	0.136	0.067
Foundation		
Depth of embedment (m)	1	3
Bearing pressure at the base (kPa)	80	187[7]

Note: [1] Physically modeled as equivalent 2 DOF structure
[2] Due to overhead space limitation, the height of the model structure was not scaled exactly from the prototype structure.
[3] Ultimate base shear estimated from nonlinear pushover analysis on the structure in OpenSees.
[4] Measured from impact hammer tests on model structures when fixed-based
[5] Measured using centrifuge ambient vibrations (small strains) when structures were placed on soil
[6] Nickel yield stress was estimated from unidirectional tensile tests, and yield strain from the ratio of measured yield strength to manufacturer-reported Young's modulus
[7] Includes the pressure from basement self-weight

Hydroxypropyl methylcellulose was used as the pore fluid during model saturation with the viscosity 70 times greater than the water (Stewart et al., 1998), in order to satisfy dynamic scaling laws (Taylor 1994). In the saturation phase, the model (soil with structures) was first flushed with CO_2. Then, the model and fluid container was kept under a vacuum pressure of 70 kPa. The vacuum pressure in the fluid container was decreased automatically at a controlled rate to initiate and continue flow at a constant rate, similar to the procedure used by Stringer and Madabhushi (2009), until saturation was completed.

All the model specimens were subjected to a series of earthquake motions at the container base in flight using the servo-hydraulic shaking table. In this paper, the result of only one motion, Kobe-L, with an achieved PGA = 0.3-0.37g; Mean period, T_m = 0.9s; and Arias Intensity, I_a, = 1.6-1.9 m/s, are discussed in detailed. Figure 2 shows the acceleration and Arias intensity time histories and the 5%-damped spectral accelerations of the Kobe-L motion measured at the base of the container during all three tests presented in this paper.

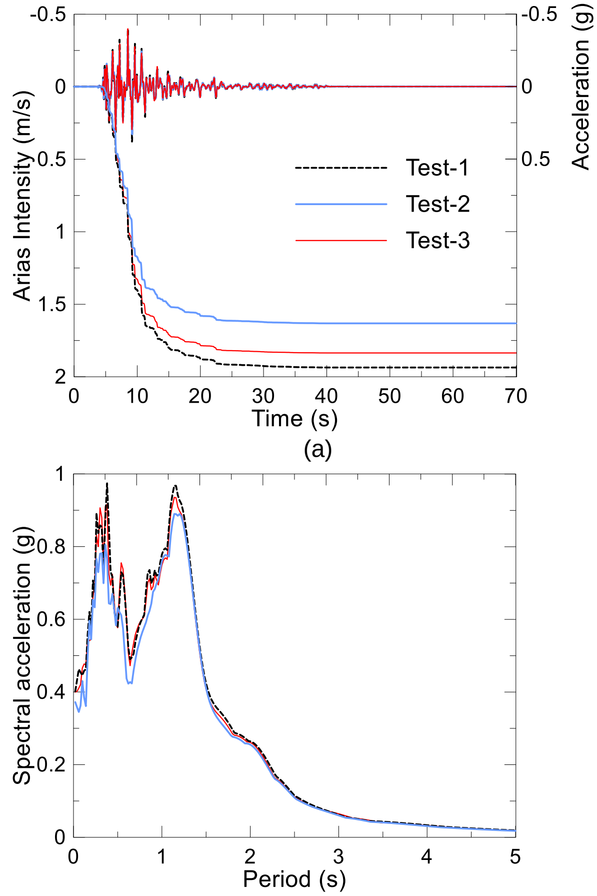


Figure 2. Base motion properties: (a) time histories of base acceleration and Arias Intensity, and (b) spectral acceleration.

3. PRELIMINARY RESULTS AND OBSERVATIONS

3.1. Influence of PVDs on excess pore pressures

The effect of PVDs on soil response under the foundation was first evaluated by measuring excess pore pressure generation during and after shaking. Figure 3 shows the excess pore pressure ratio (r_u) time histories, measured in the middle of the liquefiable soil layer under the center and edge of the structures. Here, r_u is defined as the ratio of recorded excess pore pressures (Δu), to the initial vertical effective stress at a given depth and location below the foundation (σ'_{vo}). The target design limit r_u of the PVDs (0.6) is also plotted in the same figure for comparison.

The results indicate that the soil under the mitigated structures (A-DR & B-DR) experienced rapid dissipation of excess pore pressures and smaller net r_u values compared to the unmitigated cases. The peak r_u at the center of the structure A-DR was slightly greater than the PVD design limit, because the PVDs were only placed around the perimeter of the foundation, and the footing center was farther than the radius of influence of PVDs. As expected, however, the peak r_u values around the edge of the mitigated foundations were reduced compared to the center and kept below the design value 0.6. The increased bearing pressure under Structures B and B-DR generally led to smaller peak r_u values compared to Structures A and A-DR. Rates of excess pore pressure dissipation in the foundation soil in A-DR and B-DR also led to a reduction in the SSI-induced deviatoric settlements due to ratcheting.

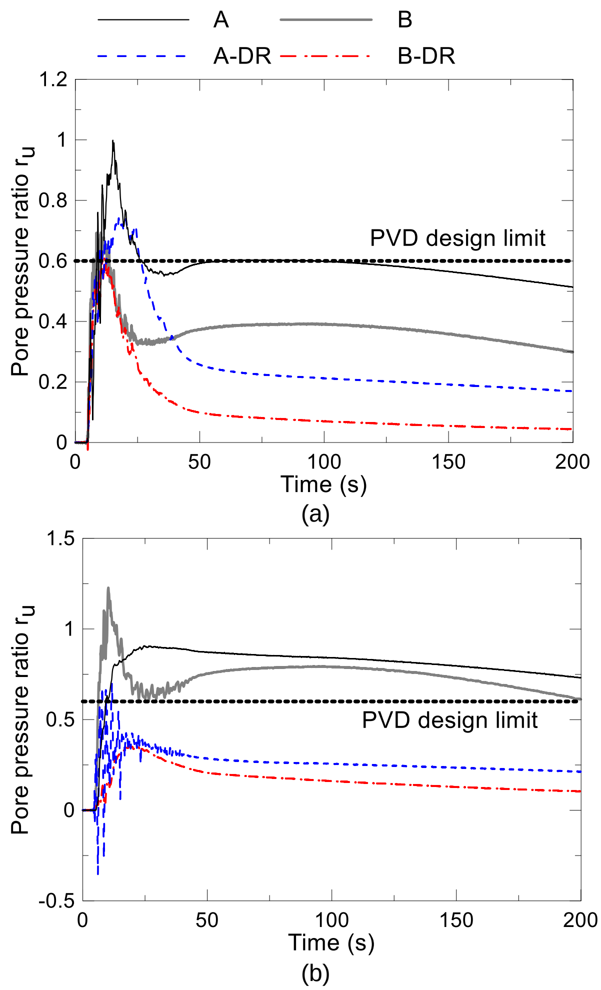


Figure 3. Excess pore pressure ratios measured at the middle of the liquefiable soil layer underneath the (a) center and (b) edge of the different structures.

3.2. Influence of PVDs on foundation response

<u>Settlements</u>: Time histories of foundation settlement are shown in Figure 4, comparing the response of mitigated (A-DR and B-DR) and unmitigated (A and B) structures. The average of LVDT recordings at the four corners of the foundation provided the average settlement of each structure. Far-field settlements recorded at the top of the liquefiable layer with vertical LVDTs during the three tests are also plotted for comparison.

The majority of settlement of the foundations and far-field soil occurred during shaking, although notable settlements (mainly volumetric) continued to occur in the far-field for a significant time period after shaking ended. The observed minor differences among far-field settlement recordings in the three tests (with identical soil profiles) is attributed mainly to the seismic interaction of soil with the nearby structures and mitigation strategy, which varied from test to test. In other words, the response in what we refer to as “far-field” was influenced by nearby structures to a small degree. Total foundation settlements were always greater than those in the far-field, primarily due to the presence of additional deviatoric mechanisms. Settlements of unmitigated Structures A and B were 57% and 34% greater than those of mitigated Structures A-DR and B-DR, respectively. Importantly, Structure A with a smaller contact pressure and embedment settled more than Structure B.

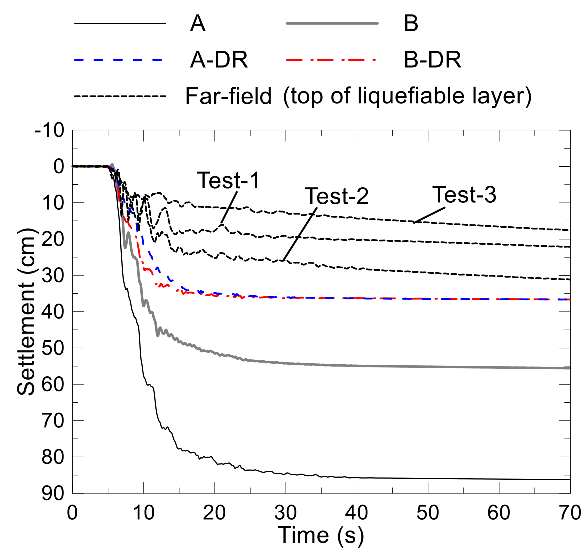


Figure 4. Average foundation settlement.

Settlements in the far-field are mainly controlled by the volumetric mechanisms (Dashti et al. 2010a): 1) settlement due to partial drainage ($\varepsilon_{p\text{-}DR}$); 2) reconsolidation ($\varepsilon_{p\text{-}CON}$); and 3) sedimentation ($\varepsilon_{p\text{-}SED}$). Under the structures, shear or deviatoric type deformations (partial bearing capacity loss, $\varepsilon_{q\text{-}BC}$, and soil-structure interaction induced building ratcheting, $\varepsilon_{q\text{-}SSI}$) are often the controlling mechanisms. Settlement of Structure A was found to exceed that of the heavier Structure B. The generation of large excess pore pressures under Structure A during the Kobe-L motion led to a greater strength loss and softening, which amplified shear type deformations ($\varepsilon_{q\text{-}BC}$ and $\varepsilon_{q\text{-}SSI}$) and volumetric strains due to partial drainage ($\varepsilon_{p\text{-}DR}$). Greater confinement under Structure B limited net r_u values during this particular motion, while its greater embedment (1-story basement) reduced the thickness of the liquefiable soil contributing to different mechanisms of deformation, reducing the net settlement of this structure. The PVDs along the perimeter of the foundation of Structures A-DR and B-DR limited net excess pore pressures by speeding up drainage underneath both buildings, which helped reduce deviatoric strains caused by strength loss in the foundation soil ($\varepsilon_{q\text{-}BC}$), while they likely amplified volumetric strains due to partial drainage ($\varepsilon_{p\text{-}DR}$). The net effect was a reduction in total settlement of Structures A-DR and B-DR compared to A and B.

Rotations: Figure 5 summarizes the foundation rotation time histories measured on the four structures during the Kobe-L motion. The ratio of the difference in LVDT recordings on the North and South ends of the foundation to its width provided the transient and permanent foundation rotation. Foundation rotation is one of primary interest or concern due to its significance for building performance and damage.

Significant transient and permanent foundation rotations were observed on Structure A without mitigation. The permanent rotation of the unmitigated Structure A was approximately 49% greater than the mitigated Structure A-DR. In contrast, mitigation with drains amplified the rotation of the taller structure. The rotation of Structure B-DR was 41% greater than the unmitigated Structure B. This was mainly due to the P-Δ effect (Gazetas 2015) on the taller, heavier, and structurally weaker Structure B, as discussed in more detail in section 3.3. Similar to settlement, rotation of Structure A was found to be greater than Structure B due to greater net excess pore pressure ratios (r_u) and strength loss in the foundation soil.

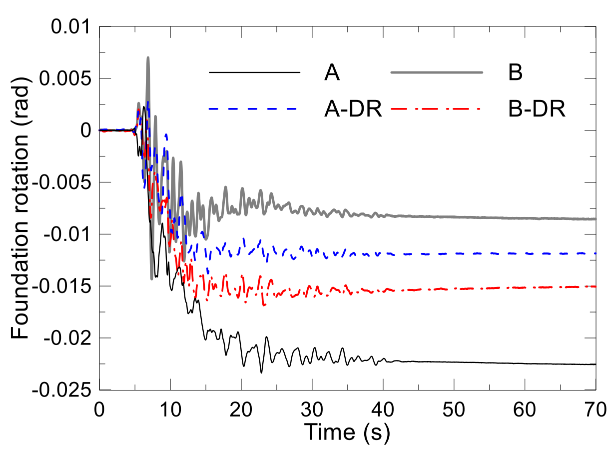


Figure 5. Transient and permanent foundation rotation.

Foundation accelerations: The influence of PVDs on soil-structure interaction and the accelerations experienced on the foundation was evaluated by plotting the spectral ratio of foundation to far-field surface accelerations (5% damped) in Figure 6. An amplification of spectral ratios was observed in periods around 1.1-1.3s in particular on the mitigated Structure B-DR. Lower net excess pore pressures due to mitigation, combined with a greater bearing pressure of Structure B-DR (amplifying both kinematic and inertial interaction), led to a greater amplification of its foundation accelerations compared to other structures.

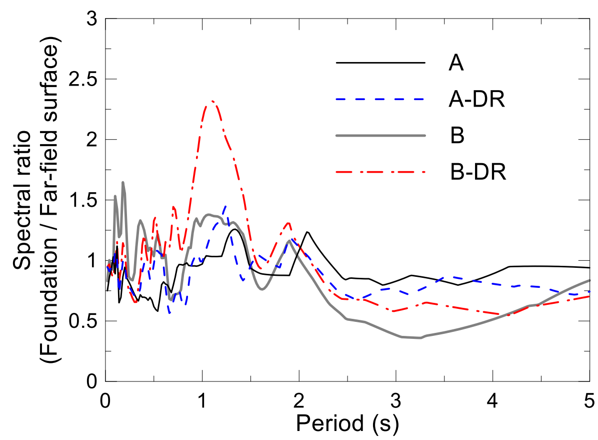
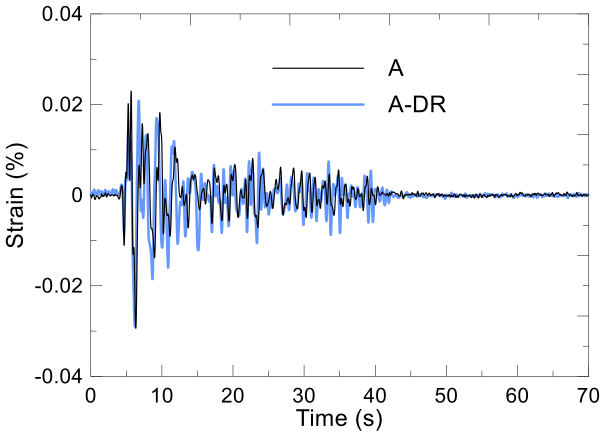


Figure 6. Spectral ratio of foundation horizontal acceleration to far-field horizontal acceleration at a depth of 1m from the soil surface.

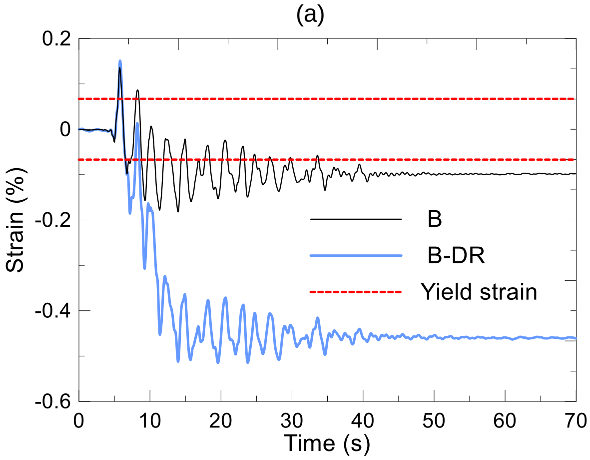
3.3. Effect of PVDs on superstructure response

Figure 7 displays the strain time histories recorded on the fuses at the base of the columns of the four structures during the Kobe-L motion. Strain gauges were placed on both sides of the fuses to measure their bending strains. In this study, the response of the superstructure of the shorter, 3-story Structure A was observed to be mostly elastic, while the taller 9-story model Structure B exhibited inelastic deformations. This pattern was expected based on their design (B was specifically designed to yield under the applied earthquake motions simulated in centrifuge, while A was designed to remain elastic). Figure 7(a) shows that the maximum strains recorded on the column fuses of Structures A and A-DR were significantly less than the yield strain (= 0.136%). Therefore, Structures A and A-DR primarily remained in their elastic range, and no yielding was observed during this motion. Yet, the addition of PVDs slightly amplified transient bending strains on Structure A-DR compared to A, due to a slightly greater demand transferred to the structure when liquefaction was mitigated with drains, but no permanent strains were measured on either structure. Figure 7(b) shows the strain time history of column fuses on Structure B during the same motion. Structural damage was observed in Structures B and B-DR, as expected, and the permanent strains recorded on Structure B-DR column fuses were approximately 3.5 times greater than those of Structure B.

Figure 8 compares the roof spectral accelerations (5% damped) on the different structures during the Kobe-L motion. Using PVDs greatly amplified the roof spectral accelerations in elastic Structure A-DR compared to the unmitigated Structure A. This may be explained by inelastic, large deformations in the soil under Structure A, damping out some of the seismic energy that would otherwise transfer to the superstructure, increasing the roof spectral accelerations (as confirmed on A-DR). The difference in the roof spectral accelerations was less significant between Structures B and B-DR. This was due to the inelastic deformations in the underlying soil (causing foundation settlement and rotation) combined with large inelastic flexural drifts and yielding of fuses, which reduced the accelerations transferred to the roof.



(a)



(b)

Figure 7. Strain time histories recorded at the column fuses of: (a) Structures A and A-DR; and (b) Structures B and B-DR.

These results suggest a tradeoff between mitigation effectiveness and superstructure response. Drains surrounding Structures A-DR and B-DR reduced the likelihood of liquefaction and strength loss in the underlying soil. This effectively amplified the acceleration demand and inertial forces transferred onto the structural elements. As a result, the PVD-mitigated structures deformed more through flexure modes in the superstructure than their unmitigated counterparts. Referring back to Figure 5, in cases where the response of the superstructure was kept within the elastic range (e.g., A and A-DR), the foundation rotation was governed primarily by deformations and softening within the underlying soil. The drains surrounding Structure A-DR reduced strength loss and net r_u values in the underlying soil by rapidly draining the excess pore pressures. Hence, they reduced foundation rotations in A-DR compared to A. The foundation rotation of Structures B and B-DR was due to the combined effects of inelastic deformations within the superstructure (yielding of fuses and permanent drift) and softening in the underlying soil. In this case, mitigation with drains in Structure B-DR reduced average foundation settlement but amplified the seismic demand transferred to the superstructure, causing yielding of fuses and permanent roof drift. After yielding of fuses started, P-Δ effects amplified moments about the base in the direction of permanent roof drift, and subsequently increased the tendency of the structure for permanent rotation.

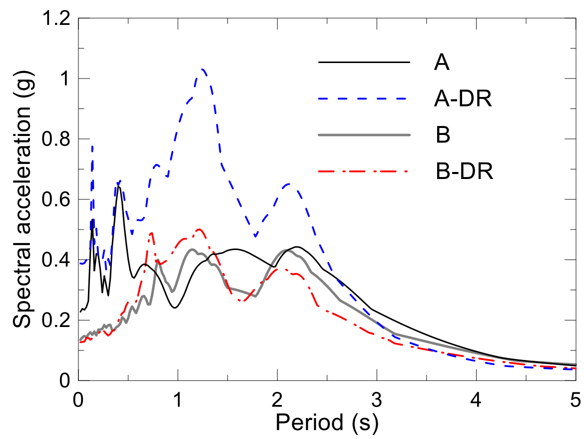


Figure 8. Spectral accelerations (5%-damped) measured on building roofs.

4. CONCLUSIONS

This paper describes three centrifuge experiments to evaluate the influence of PVDs as a liquefaction mitigation strategy on the response of soil-foundation-structure system. The performance of 3-story and 9-story steel moment-resisting frame, scaled model structures on layered liquefiable ground was evaluated with and without PVDs, in terms of excess pore pressure generation, foundation settlement and rotation, accelerations experienced on the foundation and structure roof, and deformation of column fuses. Mitigation with drains improved soil response by more rapidly dissipating excess pore pressures beneath the structure, which effectively reduced net foundation settlements but transferred a greater seismic demand to the foundation and superstructure. The influence of drains on permanent foundation rotation and permanent strains in structural elements depended on the yield capacity and dynamic properties of the structure. For example, the 3-story structure, which remained in its elastic range, experienced reduced foundation rotation and slightly greater fuse strains when mitigated with PVDs. On the other hand, the 9-story structure that yielded at its column fuses experienced increased foundation rotation and significantly greater fuse strains and flexural drifts when mitigated with PVDs due to the P-Δ effect. The insight from the experimental results presented in this paper point to the importance of considering the properties and performance of the soil-foundation-structure system holistically when designing mitigation strategies, with the goal of improving the performance not just in soil but at a systems level.

5. ACKNOWLEDGMENTS

This material is based upon work supported by the National Science Foundation (NSF) under Grant No. 1362696. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of the NSF. The authors would also like to extend their appreciation to Dr. Peter Kirkwood, Simon O Petit, and Mahir Badanagki for their assistance and ideas during the execution of centrifuge tests presented in this paper.

6. REFERENCES

American Institute of Steel Construction (AISC). 2010. *Specification for structural steel building*, 2nd edi., American National Standards.

American Society of Civil Engineers (ASCE) ASCE/SEI 7-10. 2013. *Minimum design loads for buildings and other structures*: 636.

Bird, J.F., Sancio, R.B., Bray, J.D. and Bommer, J.J. 2004. The ground failure component of earthquake loss estimations: A case study of Adapazari, Turkey, *13th world conference on earthquake engineering,* Vancouver, Canada.

Chen, Z., Hutchinson, T.C., Trombetta, N.W., Mason, H.B., Bray, J.D., Jones, K.C., Bolisetti, C., Whittaker, A.S., Choy, B,Y., Fiegel, G.L., Montgomery, J., Patel, R.J. and Reitherman, R.D. 2010. Seismic performance assessment in dense urban environments: evaluation of nonlinear building-foundation systems using centrifuge tests, *5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics,* Rolla, USA.

Cubrinovski, M. and McCahon, I. 2012. *Short Term Recovery Project 7: CBD Foundation Damage. Natural Hazards Research Platform*, University of Canterbury, Christchurch, New Zealand.

Dashti, S., Bray, J., Pestana, J., Riemer, M. and Wilson, D. 2010a. Mechanisms of seismically induced settlement of buildings with shallow foundations on liquefiable soil. *Journal of Geotechnical and Geoenvironmental Engineering,* 136(1): 151-164.

Dashti, S., Bray, J., Pestana, J., Riemer, M. and Wilson, D. 2010b. Centrifuge testing to evaluate and mitigate liquefaction-induced building settlement mechanisms, *Journal of Geotechnical and Geoenvironmental Engineering*, 136 (7): 918-929.

Gazetas G. 2015. 4th Ishihara lecture: Soil-foundation-structure system beyond conventional seismic failure thresholds, *Soil Dynamics and Earthquake Engineering*, 68: 23-39.

Hausler, E.A.2002. *Influence of ground improvement on settlement and liquefaction: A study based on field case history evidence and dynamic geotechnical centrifuge tests,* Ph.D. thesis, University of California, Berkeley, USA.

Howell, R., Rathje, E.M., Kamai, R. and Boulanger, R. 2012. Centrifuge modeling of prefabricated vertical drains for liquefaction remediation, *Journal of Geotechnical and Geoenvironmental Engineering*, 138(3): 262-271.

Liu, L. and Dobry, R. 1997. Seismic response of shallow foundation on liquefiable sand. *Journal of Geotechnical and Geoenvironmental Engineering,* 123(6): 557–567.

Liu, W., Hutchinson, T., Kutter, B.L., Hakhamaneshi, M., Aschheim, M. and Kunnath, S. 2013. Demonstration of compatible yielding between soil-foundation and superstructure components, *Journal of Structural Engineering,* 139(8): 1408–1420.

Mason, H.B. 2011. *Seismic performance assessment in dense urban environments,* Ph.D. thesis, University of California, Berkeley, USA.

Olarte,J.C.,Paramasivam, B., Dashti, S., and Liel, A. (forthcoming). Mitigation-Soil-Foundation-Structure interaction on liquefiable ground, Soil Dynamics and Earthquake Engineering (under review).

Rollins, K.M., Anderson, J.K.S., McCain, A.K. and Goughnour, R.R. 2003. Vertical composite drains for mitigating liquefaction hazard, *13th international offshore and polar engineering conference,* Hawaii, USA: 498-505.

Rollins, K.M., Goughnour, R.R., Anderson, J.K.S. and Wade, S.F. 2004. Liquefaction hazard mitigation by prefabricated vertical drains, *5th international conference on case histories in geotechnical engineering,* New York, USA.

Sancio, R.B., Bray, J.D., Durgunoglu, T. and Onalp, A. 2004. Performance of buildings over liquefiable ground in Adapazari, Turkey, *13th world conference on earthquake engineering*, Vancouver, Canada.

Stewart, D.P., Chen, Y.R. and Kutter, B.L. 1998. Experience with the use of methylcellulose as a viscous pore fluid in centrifuge models, *Geotechnical Testing Journal*, 21 (4): 365–369.

Stringer, M.E. and Madabhushi, S.P.G. 2003. Novel computer-controlled saturation of dynamic centrifuge models using high viscosity fluids, *Geotechnical Testing Journal*, 23 (6): 1-6.

Taylor R. 1995. *Geotechnical centrifuge technology*, 1st ed., Taylor & Francis, London.