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Centrifuge study of the seismic response of embankments on liquefiable soils improved with dense granular columns

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ABSTRACT: This paper presents preliminary results from four centrifuge tests, performed at the University of Colorado Boulder, to evaluate the relative influence of different mitigation mechanisms provided by dense granular columns (DGC) on the seismic performance of embankments founded on liquefiable soil deposits. The first test was designed as the baseline, unmitigated case, and the soil below the embankment in the subsequent three tests was treated with a grid of DGCs that had an area replacement ratio (A_r) = 10%. The second test evaluated the influence of shear reinforcement and enhanced drainage provided by these columns. The third test isolated the effect of shear reinforcement by preventing drainage. The fourth and last test combined the effect of shear reinforcement with densification (without drainage) provided by DGCs. For the conditions investigated, the experimental results showed that densification was more effective than drainage in limiting embankment deformations. In addition, there was no significant difference between settlements measured in tests with and without drainage, which suggests that shear reinforcement was more important than drainage in improving the overall embankment's seismic performance.

1 INTRODUCTION

Over the past few decades, dense granular columns (DGC) have become a common soil improvement strategy beneath embankments and other geo-structures founded on liquefiable deposits (Adalier & Elgamal 2004). Case-histories from past earthquakes have consistently demonstrated the effectiveness of this method in mitigating the effects of soil liquefaction (Mitchel & Wentz 1991, Mitchell et al. 1995, Hausler 2001, Nikolaou et al. 2016). DGCs are known to improve site performance through: (i) increasing the density of the surrounding soil during installation; (ii) enhancing drainage to control net excess pore water pressures; and (iii) introducing a stiff element that provides shear reinforcement to the surrounding soil (Baez & Martin 1993). Through these combined mechanisms, depending on the characteristics of the motion, site, and the geotechnical structure, successful performance has been achieved even with relatively small area replacement ratios (A_r) as low as 10% (Hausler 2002). However, the relative contribution of these mitigation mechanisms to the overall performance of the soil-embankment system is currently not well-understood, as is necessary for their reliable forward design. It is not easy to separate these effects with the existing limited, and typically non-instrumented, case-history observations. Furthermore, there is a lack of physical model studies evaluating and separating these effects under controlled conditions to, for instance, validate numerical models.

Previous experimental studies have primarily focused on the influence of prefabricated drains or granular columns on liquefiable gentle slopes (Adalier et al. 2003, Howell et al. 2012, Vytiniotis & Whittle 2017, Badanagki et al. 2018), showing that enhancing drainage can help limit seismic deformations, depending on the characteristics of the motion and columns' A_r. Badanagki et al. (2018), for example, showed successful slope performance in centrifuge with DGC A_r greater than about 20%, which is different from A_r in case history observations. Previous numerical parametric studies representing granular columns as unit cells have also demonstrated their effectiveness in reducing lateral displacements due to a combination of drainage and shear reinforcement (Elgamal et al. 2009, Asgari et al. 2013, Rayamajhi et al. 2016b). They have also provided valuable insight into shear deformations between the columns and their surrounding soil (Rayamajhi et al. 2016a), and have indicated DGCs can significantly improve site performance, particularly for A_r ranging from about 20 to 30%. However, these studies have neither evaluated the influence of DGCs on the seismic performance of more complex and realistic geo-structures on top of liquefiable soil profiles, nor explored the possible effects of installation-induced densification provided by DGCs on the overall response of the soil-structure system.

In this paper, we present preliminary results of four reduced-scale dynamic centrifuge tests, performed at the University of Colorado Boulder's Center for Infrastructure, Energy, and Space Testing (CIEST) 400 g-ton centrifuge facility, to evaluate the influence of DGCs on the seismic performance of embankments founded on liquefiable soil deposits. All tests considered a fully saturated, layered, liquefiable soil profile beneath a dense, dry, coarse sand embankment. The first test (BS) was designed as the baseline, unmitigated case. The soil below the embankment in the subsequent three tests was treated with a grid of DGCs that had an A_r = 10%. The second test (RF-DR) evaluated the influence of shear reinforcement and enhanced drainage provided by DGCs. The third test (RF) isolated the effect of shear reinforcement by preventing drainage with a latex membrane placed around the columns. The fourth and last test (RF-DS) combined the effect of shear reinforcement with densification (without drainage) provided by DGCs. The results of these experiments are aimed to provide insight into the relative influence of different mitigation mechanisms (i.e., shear reinforcement, drainage, and densification) on key engineering demand parameters that govern the embankment's design (e.g., peak shear stresses and accelerations on the embankment, peak excess pore pressures in the soil below, and cumulative embankment deformations), and provide data for the calibration and validation of advanced numerical models in the future.

2 EXPERIMENTAL PROGRAM

A series of centrifuge experiments, with a prototype-to-model scale factor of 70, were performed at the University of Colorado Boulder's (CU) 400g-ton (5.5 m radius) centrifuge facility, to systematically evaluate the influence of dense granular columns (DGC) and the relative importance of the mechanisms of densification, enhanced drainage, and reinforcement, on the seismic performance of embankments founded on liquefiable soil deposits during 1D horizontal earthquake loading. Figure 1 shows the typical configuration and instrumentation layout used in these experiments. Dimensions are presented in prototype scale, following accepted scaling relations (Tan & Scott 1985, Garnier et al. 2007).

For all tests, a dense layer (12 m-thick in prototype scale) of Ottawa sand F65 was dry pluviated to attain a relative density (D_r) of approximately 90% at the bottom of a flexible-shear beam container. Subsequently, a loose layer of Ottawa sand (4 m-thick) with D_r≈40% was modeled as the liquefiable material. This layer was overlaid by a 2 m-thick layer of coarse Monterrey sand 0/30 with Dr≈90% to create a non-liquefiable crust. The DGCs with diameter = 1.75 m and a center-to-center spacing of 4.9 m (in prototype scale) were placed vertically (in a square pattern) at an elevation of 8 m above the bottom of the container, prior to finishing sand pluvation, to avoid localized densification of sand during their installation, and to keep the density of the surrounding soil well controlled and uniform within each layer. The engineering soil properties of these materials, as measured at CU, are provided in Table 1.

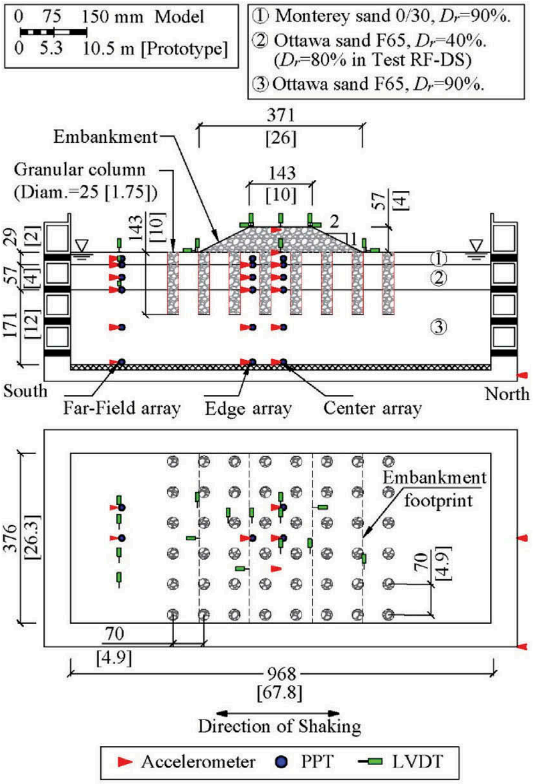

Figure 1. Instrumentation layout of the centrifuge experiments in elevation (top) and plan (bottom) views.

Table 1. Engineering properties of different soil layers used in centrifuge experiments.

	Ottawa sand F65	Monterey sand 0/30	Granular material (embankment & columns)
Specific gravity, G_s	2.65	2.64	2.66
Minimum void ratio, e_{min}	0.53	0.54	0.62
Minimum void ratio, e_{min}	0.81	0.84	0.92
Uniformity coefficient, C_u	1.56	1.30	1.54
Hydraulic conductivity, k (cm/s)	1.41×10^{-2} (D_r=40%) 1.19×10^{-2} (D_r=90%)	5.29×10^{-2}	2.90

Test BS was considered a non-mitigated case, absent granular columns. In Test RF-DR, DGCs were encased within geotextile filters (to avoid clogging in subsequent shakes and allow water flow) and placed at designated locations shown in Figure 1. In Test RF, DGCs with geotextile filters were encased within a thin (0.2 mm-thick) latex membrane to prevent drainage while providing shear reinforcement. The last test, RF-DS, the DGCs were encased within latex (similar to Test RF), but the soil surrounding the DGCs was also pluviated at a denser state of $D_r \approx 80\%$. In all tests with mitigation, an A_r = 10% was used for the DGC's. A_r is defined as the ratio of granular column area to the total treatment area in plan view (Baez & Martin 1993).

It is known that the installation of granular columns in the field typically induces ground densification, as well as an increase in lateral earth pressures in the surrounding soil. However,

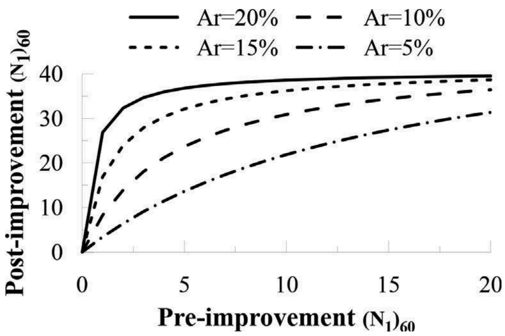

Figure 2. Pre- and post-improvement $(N_1)_{60}$ during the installation of DGCs (After Baez 1995).

due to the difficulties of column installation in flight during centrifuge experiments, in Test RF-DS, densification was considered in an approximate and roughly uniform manner. By making use of the design chart for granular columns developed by Baez (1995) (based on field data from 18 case histories) that is shown in Figure 2, the initial relative density of the potentially liquefiable layer was converted to an equivalent pre-improvement $(N_1)_{60}$ value (SPT blow count normalized to an overburden pressure of 1 atm) through the expression:

$$D_r = \sqrt{\frac{(N_1)_{60}}{C_d}} \tag{1}$$

C_d is a parameter that accounts for the grain-size composition of soil. Typical C_d values for fine sand used in laboratory tests are about 35 (Idriss & Boulanger 2008). For A_r =10%, based on equation (1), the post-improvement $(N_1)_{60}$ value was estimated and converted to an equivalent relative density (D_r) for the soil surrounding the columns after installation. This resulted in $D_r \approx 80\%$ noted previously for the middle sand layer in Test RF-DS (see Figure 1). Both Monterey and Ottawa sand layers with a $D_r \approx 90\%$ were considered not to be significantly affected by DGC installation. Note, however, that this method of simulating installation-induced densification does not capture the changes in soil fabric under vibration, but it isolates the effect of soil density or void space.

After dry preparation, the models were saturated with a solution of hydroxypropyl methylcellulose in water (Stewart et al. 1998) with a viscosity 70 times greater than that of water, to satisfy both diffusive and dynamic scaling laws (Taylor 1995). Initially, the soil model was flushed with CO_2 from the bottom of the container, after which it was kept under constant vacuum. Then, the vacuum level in the fluid tank was controlled automatically (using a computer system similar to that proposed by Stringer & Madabushi 2009) to maintain a safe and constant flow rate below that required for flow-induced liquefaction until saturation was completed. The model was subsequently spun to 70 g of centrifugal acceleration, after which several 1D horizontal earthquake motions were applied to its base in flight using a servo-controlled hydraulic shake table (Ketchum 1989). This paper focuses on the soil-embankment system performance during the first major motion: the horizontal component of a scaled version of the 1995 Kobe earthquake recorded at the Port Island station, with Peak Ground Acceleration (PGA) = 0.37 g, mean period (T_m) = 0.88 s, significant duration ($D_{5\text{-}95}$) = 12.1 s, and Arias Intensity (I_a) = 2.2 m/s.

3 PRELIMINARY EXPERIMENTAL RESULTS

3.1 *Influence of granular columns on embankment performance*

The effectiveness of DGCs and the relative importance of mitigation mechanisms of densification, enhanced drainage, and reinforcement, on the seismic performance of the embankment

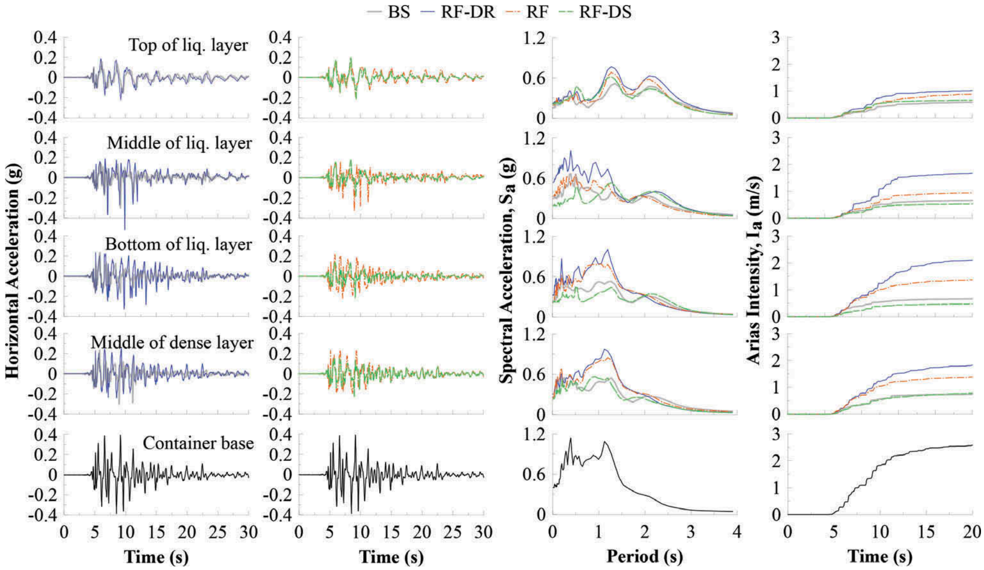

Figure 3. Acceleration time histories, response spectra (5%-damped), and Arias Intensity (I_a) time histories recorded at the center array of all models during the Kobe motion.

was evaluated in terms of accelerations and excess pore pressures in the foundation soil as well as deformations within the embankment. Figure 3 compares the results among different tests in terms of the time history of accelerations and Arias Intensities as well as response spectra in the center array below the embankment. Figure 4 compares the results in terms of the time history of excess pore pressure ratio (i.e., $r_u = \Delta u/\sigma'_z$, where Δu is the excess pore water pressure and σ'_z is the initial vertical effective stress at a given depth) in the center and edge arrays as well as the settlement and angular distortion of the embankment.

In general, all models exhibited significant de-amplification of accelerations at shorter periods (higher frequencies) and amplification at higher periods (shorter frequencies) from the base toward the ground surface due to soil softening. On the other hand, even though large excess pore water pressures were measured within the bottom layer of dense Ottawa sand, liquefaction (defined as r_u=1.0) was typically not reached in that layer particularly under the center of the embankment with a greater initial vertical stress. As expected, Test RF-DR was successful in reducing the extent and duration of large excess pore pressures in all layers when compared to the unmitigated, baseline case BS. Test RF, which provided similar shear reinforcement but did not enhance drainage (or even laterally inhibited drainage slightly with the use of latex around the columns), reduced the rate of dissipation and increased net excess pore pressures particularly after strong shaking at most depths compared to Tests BS and RF-DR. Interestingly, a more uniformly dense profile below the treated embankment in Test RF-DS amplified the excess pore pressures at lower elevations (within dense Ottawa sand) in a similar manner as the case without mitigation (BS), while reducing the extent of softening in the top layer of Ottawa sand compared to all other tests. In all tests, excess pore pressures at locations close to the ground surface and below the embankment experienced low r_u values, owing primarily to the high drainage capacity of the top Monterey sand layer.

Tests RF and RF-DR showed similar patterns of amplification in low-period accelerations at higher elevations within the looser layer of Ottawa sand. This increased acceleration demand within the treated soil was associated with shear reinforcement of the DGCs under the confining pressure and boundary conditions of the embankment above (as opposed to the case of a free-face, where shear reinforcement may reduce accelerations in the surrounding soil, as observed previously by Badanagki et al. 2018). The added drainage in Test RF-DR

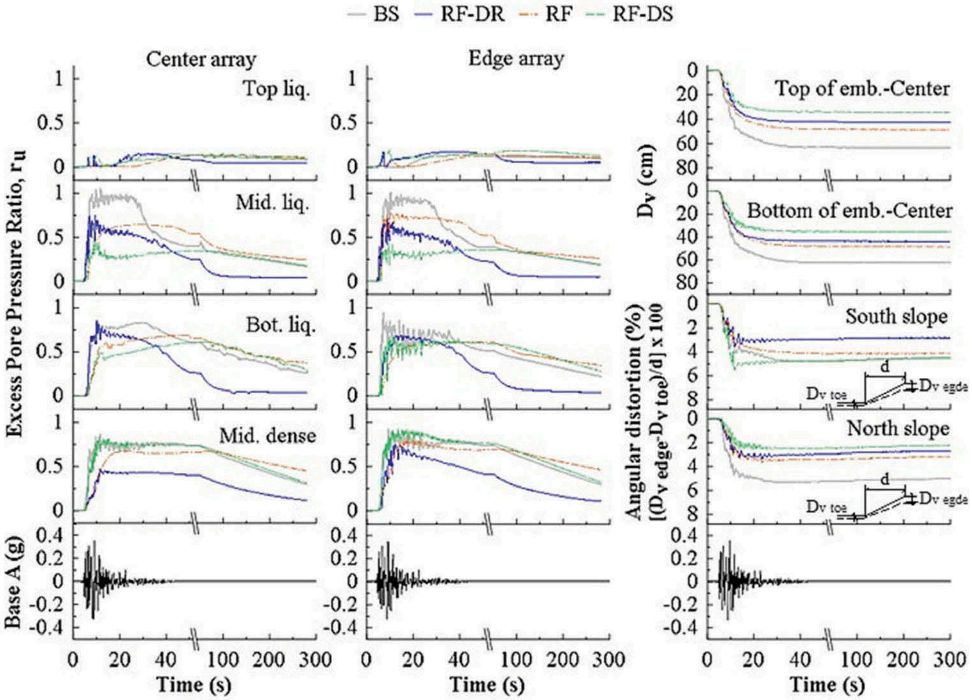

Figure 4. Base acceleration, excess pore pressure ratio, vertical displacement, and angular distortion time histories measured during the Kobe earthquake motion.

further amplified low-period accelerations by dissipating excess pore pressures and increasing soil stiffness more rapidly, particularly at higher elevations.

As shown in Figure 3, the combined effects of reinforcement and densification with no drainage (Test RF-DS) resulted in lower spectral accelerations at all depths along the center array, in a roughly similar manner as the unmitigated case BS. This response may be explained by the large excess pore pressure recordings at lower elevations, which in a way, reduced the seismic demand transferred to higher elevations. Figure 4 showed that creating a more uniformly dense deposit in RF-DS led to larger r_u values within the lower dense layer of Ottawa sand similar to those measured in Test BS (with no DGCs or densification) and significantly greater than those measured in Tests RF-DR and RF. Even though increased density of the top Ottawa sand layer reduced the extent of softening within that layer, greater pore pressures at lower elevations in Test RF-DS reduced soil's strength and stiffness, de-amplifying high-frequency accelerations that propagated upward toward the embankment.

Figure 4 also shows the time histories of vertical displacement (settlement) D_v recorded at the top and bottom of the embankment along its centerline. Overall, the embankment itself did not experience notable volumetric strains, as it was placed at a very dense initial state. As a result, total embankment settlements were almost entirely due to the settlement of the soil deposit below (as shown in Figure 4), which was caused by a combination of volumetric and shear type deformations (Dashti et al. 2010). DGCs used in Tests RF-DR, RF, and RF-DS were successful in reducing settlements with respect to the unmitigated case, BS, with the common denominator of shear reinforcement. The comparison between Tests RF-DR and RF-DS, however, shows that the densification mechanism was more effective than drainage in limiting the embankment's settlement for the conditions investigated. The combination of shear reinforcement, de-amplification of low-period accelerations, and reduction in net excess pore pressures at higher elevations observed in Test RF-DS reduced the embankment's permanent settlement by about 50% when compared to Test BS. In addition, there was no significant difference between settlements measured in Tests RF-DR and RF, suggesting that shear reinforcement was more important than drainage in limiting net settlements below the embankment.

To quantify changes in the angle of embankment's slope over time, its *angular distortion* was approximated as the difference in vertical displacements between the edge ($D_{v\ edge}$) and the toe ($D_{v\ toe}$) of the embankment slope, divided by the horizontal distance (d) between those two locations after construction or before shaking. Figure 4 shows the angular distortion time histories experienced by the north and south slopes of the embankment during the Kobe motion. In general, Tests BS and RF-DS showed a non-symmetrical deformation pattern, whereas Tests RF-DR and RF exhibited similar combinations of volumetric and shear deformations at both slopes. Nevertheless, on the north side, the angular distortion patterns among tests were similar to the settlements, showing that combined reinforcement and densification was most successful in reducing distortions, and that drainage did not provide additional benefits to reinforcement in limiting those deformations. It must be noted, however, that these conclusions are only applicable to the conditions (geometry, soil properties, and ground motion) investigated here and cannot be generalized before additional numerical investigation.

4 SUMMARY AND CONCLUSIONS

Preliminary results from four dynamic centrifuge tests studying the influence of DGC on the seismic performance of embankments on top of liquefiable soil deposits were presented. Overall, it was found that DGCs used in Tests RF-DR, RF, and RF-DS were effective in improving performance when compared to the unmitigated case. However, the results from Tests RF-DR and RF-DS showed that densification was more effective than drainage in limiting the embankment's deformations. Additionally, no significant difference was found between settlements measured in Tests RF-DR and RF, suggesting that shear reinforcement was more important than drainage in limiting net settlements beneath the embankment.

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