In-Ground Gravel-Rubber Panel Walls to Mitigate and Base Isolate Shallow-Founded Structures on Liquefiable Ground

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ABSTRACT

The effectiveness of a new liquefaction mitigation strategy is investigated experimentally for newly constructed shallow-founded structures: an in-ground gravel-rubber (GR) panel wall system. The goal was to limit the negative consequences of liquefaction in terms of permanent seismic deformation, while benefitting from the positive consequences of liquefaction in terms of base isolation. The influence of GRs was systematically evaluated on the seismic performance of a layered liquefiable deposit in the far-field and near two different model structures. The structures represented the key properties of a 3-story building (A) on a 1 m-thick mat foundation and a 9-story building (B) with a 1-story basement. The performance of Structure A with GRs was also compared with a similar structure without mitigation and with conventional mitigation strategies that either enhanced drainage alone (e.g., prefabricated vertical drains) or increased shear stiffness around the foundation's perimeter (e.g., structural walls). Test results showed that the GR wall system could greatly improve the overall seismic performance of short-period structures like A, but may be detrimental to long-period structures like B. The GRs below Structure A effectively isolated the total system, reducing average and differential settlements below the foundation (although not necessarily to acceptable levels), while also reducing the seismic demand transferred to the superstructure, a combination rarely observed by conventional mitigation strategies. The same GR system under Structure B experienced greater seismic moments and shear stress, inducing large shear deformations in soil that led to this structure's significant rotation and flexural deflection. The foundation continued to rotate even after shaking due to P-Δ effects, resulting in its overturning failure. These results show that GR systems can be quite effective for low-rise structures, but additional reinforcement may be necessary to reduce foundation tilt. Use of such mitigation measures under taller and heavier structures must be accompanied with great

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caution. Despite their practical limitations, evaluation of GR panel walls may guide future developments of combined, economical, and sustainable mitigation strategies that improve the overall performance of the soil-structure system.

Keywords: Liquefaction; Centrifuge modeling; Soil-foundation-structure interaction; mitigation; base isolation; in-ground panel walls; performance-based seismic design.

1 INTRODUCTION

Liquefaction mitigation techniques are often employed to alleviate the liquefaction hazard, its associated ground deformations and, ideally, building settlement and tilt. Centrifuge model studies have previously been performed to investigate the seismic performance of shallow-founded structures on liquefiable ground with different mitigation techniques (e.g., Liu and Dobry 1997; Balakrishnan and Kutter 1999; Hausler 2002; Adalier et al. 2003; Dashti et al. 2010b; Mitrani and Madabhushi 2012). These experiments aimed to simulate the contact pressure and, in some cases, the fundamental frequency of a realistic prototype structure and, as such, typically modeled the structure as a rigid mass or a single-degree-of-freedom (SDOF), linear-elastic oscillator. One such study, Dashti et al. (2010a), argued that building settlements on liquefiable deposits are controlled by: 1) volumetric settlement mechanisms of partial drainage during shaking ($\varepsilon_{p\text{-}DR}$), sedimentation ($\varepsilon_{p\text{-}SED}$), and reconsolidation settlement ($\varepsilon_{p\text{-}CON}$); and 2) shear or deviatoric settlement mechanisms of partial bearing capacity loss ($\varepsilon_{q\text{-}BC}$) and SSI-induced ratcheting ($\varepsilon_{q\text{-}SSI}$). Different mitigation strategies may be used to strategically isolate and minimize mechanisms that contribute the most to total building movements.

Ongoing research by the authors aims to holistically assess the effectiveness of different mitigation techniques on the performance of soil-foundation-structure (SFS) systems, considering multiple-degree-of-freedom (MDOF), inelastic structures on liquefiable deposits, through both numerical and centrifuge modeling. Olarte et al. (2017; 2018a,b) and Paramasivam et al. (2018a), for example, conducted centrifuge experiments to evaluate the seismic response of 3- and 9-story structures founded on layered, liquefiable deposits with three traditional mitigation techniques: 1) ground densification, 2) enhanced drainage through prefabricated vertical drains (PVD), and 3) soil reinforcement with stiff in-ground structural walls. The model structures in these experiments were designed as potentially inelastic, meaning that structure could undergo inelastic deformations under seismic demands that exceeded their design level and overstrength. These studies showed that traditional methods of mitigation can be successful in limiting shear deformations ($\varepsilon_{q\text{-}BC}$ and $\varepsilon_{q\text{-}SSI}$) or the extent and duration of large pore pressures (hence, softening which limited the contribution of $\varepsilon_{p\text{-}CON}$ and $\varepsilon_{p\text{-}SED}$) in the foundation soil, reducing net settlements of the structure. However, liquefaction mitigation often amplified the acceleration and deformation demands imposed on the superstructure (e.g., strains or drift demands) compared to the unmitigated cases, and had possibly adverse effects on the foundation's permanent rotation or tilt. As a result, these studies identified important tradeoffs in performance between the unmitigated cases, in which soil liquefaction essentially "isolated" the superstructure but led to excessive settlement and tilt, and the mitigated cases, which experienced higher seismic demands on the superstructure and reduced settlement (although not necessarily to acceptable levels) and sometimes tilt. Overall, these experimental studies found that none of the traditional techniques adequately improved the performance of the entire SFS system to acceptable design levels for the conditions evaluated in centrifuge studies.

These observations identified a need for a mitigation strategy that combines the benefits of traditional methods (e.g., enhanced drainage and shear reinforcement to reduce settlement) with the positive isolation attributes of liquefaction (e.g., energy dissipation and period elongation). In this study, we designed, fabricated, and tested in the geotechnical centrifuge a new, hypothetical, in-ground, gravel-rubber panel wall system (GR), consisting of alternate layers of rubber and gravel. We compared the influence of GRs on the SFS system to two traditional mitigation techniques that either enhanced drainage alone (PVDs) or increased shear stiffness in the perimeter soil while inhibiting lateral drainage (in-ground structural walls or SWs). The GR panel walls were vertically stiff, in order to limit the settlement and tilt of the structure, and laterally flexible, to isolate the structure and attempt to reduce transverse accelerations transferred to the foundation and superstructure. In addition, the gravel and rubber layers in the panel walls enhanced drainage (e.g., each rubber layer had holes to accommodate vertical drainage through gravel), and the rubber increased the system's damping characteristics. In this paper, we experimentally investigate the influence of the GR panel wall system on seismic site response and performance of the SFS (soil-foundation-structure) system on a layered, liquefiable deposit. The tests enable a comparison of the GRs with other conventional mitigation measures for one structure, focusing on PVDs and SWs, and a comparison of two different structures with GRs. These two structures have different dynamic properties, embedment depths, bearing pressures, and strength: the 3-story structure had a 1 m-thick mat foundation (Structure A), and the 9-story structure on a 1-story basement (Structure B). Although such a technique may have important practical limitations, this hypothetical exercise may guide future developments of combined liquefaction mitigation strategies that are both low-cost and environmentally sustainable.

2 CENTRIFUGE EXPERIMENTAL SETUP

This paper presents the results of four dynamic tests performed using the 5.5 m-radius centrifuge facility at the University of Colorado Boulder. Table 1 details the characteristics of the centrifuge experiments. The first experiment, Test FF_{GR}-$FF_{GR,L}$, investigated seismic site response under 1D horizontal shaking in a layered liquefiable deposit (with no structures) mitigated using GRs, and the same walls surrounded by latex (GR,L) to avoid drainage. Test A_{GR}-B_{GR} examined simultaneously the response of 3- and 9-story structures (A and B) on the same soil profile mitigated with GRs. Test A_{UM} and Test A_{DR}-A_{SW} simulated the response of Structure A first without any mitigation (UM), and when mitigated with PVDs (DR) and stiff, in-ground structural walls (SWs) around its perimeter.

Figure 1 shows the geometry and relative density of the soil profile in different tests, and Table 2 summarizes the key properties of different soil layers. The soil-structure models were constructed in a flexible-shear-beam (FSB) container made of alternate layers of hollow aluminum ring and rubber layers (Paramasivam 2018b). An automated sand pourer was used to dry pluviate each layer of sand to the target relative density and thickness (Kirkwood et al. 2018), to achieve greater uniformity and repeatability than possible with manual devices. A solution of hydroxyl propyl methylcellulose with a viscosity 70 times greater than that of water was used as the pore fluid to satisfy the dynamic and diffusion scaling laws. A computer-controlled automatic saturation setup was used to saturate the soil models (Paramasivam et al. 2018a). The water table level was maintained just above the surface to ensure complete saturation of all soil layers. Soil models after saturation were placed on the centrifuge arm and spun to 70g of centrifugal acceleration. All the soil layers were subject to slight changes in relative density after saturation and more importantly, after spinning up to higher gravity. However, these changes in density were not measured at all locations due to the limited capability of instrumentations.

Soil and structure models, in general, were instrumented with four different types of sensors at key locations to record accelerations, excess pore pressures, displacements, and bending strains in mechanical fuses (Figures 1a through e). These include 32 accelerometers, 19 pressure transducers, 16 LVDTs, and 32 strain gauges. Additional details on instrumentation are provided in the supplemental section S1 as well as Paramasivam (2018b). In this paper, the reported residual deformations during each motion are not cumulative. All the units reported in this paper are in prototype scale, unless otherwise noted.

2.1 Model Structures

Two "special" code-conforming steel moment resisting framed structures were designed and modeled for centrifuge testing by Olarte et al. (2018a), as shown in Figure 1 and detailed in supplemental section S2. These structures were fully designed according to modern seismic provisions and typical practice to be as realistic as possible for high seismic areas, accounting for centrifuge constraints. The 3-story structure (A) was simplified with 3DOFs to capture the three primary lateral modes of deformation (fixed-base fundamental frequency, f_{STo} = 1.72 Hz), as well as the inertial mass, stiffness, base shear strength, and overturning moment expected for a 3-story structure in a high seismic area. The 9-story structure (B) was simplified with 2DOFs (f_{STo} = 0.45 Hz) that captured only the first two lateral modes of vibration, due to constraints related to constructability at reduced scale and centrifuge overhead clearance. As a result, Structure B had inertial mass, stiffness, and base shear strength expected in a typical 9-story structure, but not the base moment nor higher mode effects. Inelastic response in the model structures was designed to concentrate at the beam ends and column bases in replaceable "fuses" (Figure 1c). Model Structure B was designed for a lower seismicity site and was therefore weaker than Structure A. Hence, Structure B was expected to experience nonlinearity and inelastic deformations during the motions used in centrifuge. Supplemental section S2 provides additional details on these structures.

Structure A rested on a 1 m-thick mat foundation with an embedment depth of 1 m, while Structure B had a 1-story basement with an embedment depth of 3 m (Figure 1c). The footprint of Structures A and B were identical, but the bearing pressure of B below its basement (187 kPa) was greater than that of A below its mat foundation (76 kPa).

2.2 Mitigation Techniques

This study aim to examine mechanistically how combining aspects of drainage, reinforcement, and base isolation through the GR panel walls influence the performance of the SFS system, when compared to traditional mitigation techniques that isolate either the influence of drainage (e.g., PVDs) or shear reinforcement (SWs). In an effort to isolate the effects of different mitigation mechanisms, none of these experiments considered the influence of installation-induced ground densification (during model preparation), maintaining soil fabric and density among different tests.

In-ground gravel-rubber panel walls

Figure 2 illustrates the three different mitigation techniques modeled in this study. The GR panel walls were designed as a closed grid (2 x 2) structure, consisting of alternate layers of coarse sand (or fine gravel) and solid rubber. The length and width of the outer walls were designed to exactly match the foundation dimensions, and the depth was selected as 11 m in prototype scale (achieving the same total treatment depth as SWs and PVDs, described below). Based on the constraints of model scale constructability, the wall thickness was selected as 0.7 m in prototype scale, which led to an area replacement ratio (defined as the ratio of panel wall total area to plan-view foundation area) of 38%. The grid-like configuration of panel walls presumes that this system is for newly-built structures.

The detailed dimensions, spatial distribution, and thickness of rubber and gravel layers, as shown in Figure 1f, were designed to satisfy the following goals to the extent possible: 1) target a fundamental frequency of the soil-mitigation system under the confinement of structures that is between approximately 0.5 and 0.8 Hz, to isolate the structure and avoid resonance with any of the structures' fundamental modes; 2) maintain sufficient lateral stiffness to limit shear strains within the liquefiable layer and hence, seismic settlements at the surface; 3) provide sufficient drainage capability to limit the extent and duration of large excess pore pressures in the soil; 4) provide sufficient vertical stiffness to reduce permanent settlement and tilt of the structures; and 5) minimize the area replacement ratio of panels to reduce cost.

To design the mitigation, 3D modal analyses of different configurations of the GR system (fixed at its base without the surrounding soil, but considering the bearing pressures of the two structures) were conducted using the finite element program, *Abaqus*, in prototype scale. The primary reason for this simplified approach during design was to increase the computational speed for a large number of simulations required when determining the stiffness, thickness, and spatial distribution of rubber and gravel layers that would achieve the target fundamental frequency. However, this simple approach meant that nonlinear and nonstationary seismic interactions among the liquefying soil, superstructure, and GR panel walls could not be reliably represented, affecting the accuracy of the estimated modal frequencies, as explored in Supplement S3. Nevertheless, this approach was judged appropriate for the design phase. Subsequently, the optimum configuration was selected (Figure 1d), which yielded a fundamental frequency of 0.8 Hz for the GR system, satisfying the target goal. In the selected configuration, thick rubber layers were provided at the deeper locations, and the rubber thickness was reduced toward the top of the wall. In contrast, the thickness of gravel layers increased from the bottom to the top. This arrangement was selected to increase the system's lateral flexibility without notably compromising shear stiffness of the GR system.

Static analyses were subsequently performed on the modeled GR under the structures' vertical loads, to estimate static settlement, assuming that the entire structure's load would be transferred to the GR walls during centrifuge spin up. In static analyses, the soil surrounding the GR system was simplistically represented as lateral stress along the panel walls. The maximum vertical settlement of the GR was obtained as 62 and 110 mm in prototype scale under the weight of Structures A and B, respectively. These estimates were clearly unacceptable for typical foundation design, but they were accepted in this preliminary design stage, owing to simplifications and conservatism embedded in the numerical modeling (see supplemental section S3). Nevertheless, if such methods are to be used in practice, the mitigation and structure design need to consider static settlement and reduce its effects on the foundation.

In the centrifuge model, coarse silica sand (or fine gravel) with a mean particle diameter of D_{50} = 1.2 mm in model scale was selected to represent gravel layers in the GR panel walls. This particle size was selected to provide greater permeability than the surrounding, loose, fine Ottawa sand (by a factor ranging from 10 to 100), while having at least 10 particles across the thickness of the panel walls. The gravel layers were constructed by dry pluviation at a $D_r \approx 95\%$. Natural rubber with a hardness of 50A of three different thicknesses was used to construct the rubber layers. We created holes (diameter = 3.18 mm, spacing = 17 mm in model scale) in the rubber layers to enable vertical drainage through gravel. A woven polyester filter with an aperture opening size of 0.178 mm was placed around the GR walls to avoid clogging by fines from the surrounding soil during consecutive shakings. The GR walls were placed directly underneath the foundation, and thus could carry gravity loads from the superstructure.

A temporary support system was designed to construct the GR panel walls in Tests FF_{GR}-$FF_{GR,L}$ and A_{GR}-B_{GR}. In particular, an open-ended, square, aluminum support box was placed around the perimeter of the walls, and steel rods with spacers were used inside the box to guide the construction of grid structure. This arrangement was designed to be stiff enough to provide resistance against bulging of gravel layers during specimen construction. An acrylic frame was bolted at the top of each grid to avoid rotation in plan during pluviation of soil inside and around the gravel-rubber walls. The instruments inside the grids were tied to the rods and placed at their respective depths. The temporary support system was removed after construction of the gravel-rubber walls and pluviation of surrounding Ottawa sand layers. Further details on the design and construction of panel walls are provided in supplemental section S3.

Prefabricated vertical drains and in-ground structural walls

In Test A_{DR}-A_{SW}, PVDs and SWs were placed around the perimeter of Structures A_{DR} and A_{SW}, respectively, extending from the middle of dense Ottawa sand to the soil surface. Their perimeter placement makes them suitable for new or existing structures. The design and fabrication of PVDs and SWs are detailed in Olarte et al. (2017) and supplemental section S3. Structure A_{DR} was treated with 72 PVDs, placed in a triangular pattern at a center-to-center spacing of 17 mm in model scale [1.2 m in prototype units], as shown in Figure 2b, to enhance drainage and limit net pore pressures without notable shear reinforcement. Similarly, the

SW around Structure A_{SW} (Figure 2c) was designed as a stiff in-ground structure that limited shear strains without enhancing drainage. These walls were placed close to the footing area, with a gap of 2.2 mm in model scale [157 mm prototype], to minimize shear movements of soil through the gaps. This separation was not intended to represent a realistic condition. Instead, the primary objective was to fabricate the walls as close to the foundation as possible in light of the fabrication constraints, in order to minimize the contribution of shear strains to total deformations within the structural walls.

2.3 Ground Motions

A series of five, 1D horizontal earthquake motions were applied in the same order to the base of the container in flight during all tests using a servo-hydraulic shaking table (Paramasivam 2018b). These motions were selected to cover a range of characteristics in terms of amplitude, frequency content, and duration. However, the test results in this paper are discussed only for the first two significant motions, referred to as Kobe-L and Joshua-H, because of significant changes in soil properties and geometry after Joshua-H. Owing to notable changes in soil and structural properties after the first motion, Kobe-L was the most reliable motion for comparison. The results of Joshua-H are presented nevertheless, to evaluate the system's performance during a motion with different characteristics.

Table 3 summarizes the mean properties of the first two major motions as recorded on the container base during the four tests. Figure 3 shows the acceleration and Arias Intensity time histories, as well as the acceleration response spectra of Kobe-L and Joshua-H motions, showing reasonable repeatability among different experiments, particularly in terms of response spectra (Paramasivam 2018b).

3 CENTRIFUGE TEST RESULTS AND OBSERVATIONS

3.1 Influence of Gravel-Rubber Panel Walls on Seismic Site Response

To examine the seismic response of a layered liquefiable deposit with no structures, test results from FF_{GR} and $FF_{GR,L}$ with the GR system were compared with the far-field in the same test with no mitigation (FF_1;location shown in Figure 1a). The far-field location, in this study, was selected half-way between the two hypothetical structures to maximize distance to foundations, mitigation techniques, and container walls and reduce the influence of their interaction on the far-field location. Despite the absence of buildings in Test FF_{GR}-$FF_{GR,L}$, some interaction was still expected among FF_1, the mitigation, and container boundaries (as in any centrifuge test), which should be considered when interpreting the results.

All FF profiles demonstrated a significant reduction in spectral energies (at a frequency range of 0.8-2 Hz) from the base to the soil surface due to extended softening (i.e., r_u values reaching 1.0 at most depths) and damping, as shown in Figures 4 and 5. Only frequencies ranging from about 0.35 to 0.7 Hz were slightly amplified from the base to the surface, representing the site's effective, strain compatible, fundamental frequency, f_{so}'. The unmitigated FF_1 column also showed some higher frequency amplifications (around 10 Hz) near the surface during Kobe-L due to soil's dilation or re-stiffening after extensive softening. As a result, peak ground accelerations (PGAs) were amplified toward the surface in FF_1, an effect that was suppressed in the presence of GRs. The minor difference in FF accelerations among mitigation techniques was due to the combined effects of shear reinforcement and energy dissipation mechanisms of the GRs. The GRs limited the extent of shear strains and correspondingly the dilation tendency of the loose soil inside, hence, reducing high frequency accelerations and PGAs compared to FF_1 within the loose layer of Ottawa sand. However, the overall similarity of accelerations at lower frequencies among the three FF measurements indicates that the GRs did not significantly alter the site f_{so}', regardless of their drainage ability, indicating that the system maintained liquefying soil's base isolation effects.

The generation of excess pore pressures (Δu) was not affected notably by the mitigation system during shaking, leading to peak r_u of near 1 in all cases. However, pore pressure generation was a bit slower in FF_{GR} due to the enhanced drainage, as shown in Figures 4 and 6. In addition, the loose Ottawa sand layer experienced slightly lower peak Δu values during strong shaking in FF_{GR} and $FF_{GR,L}$ compared to FF_1, primarily due to the additional shear reinforcement provided by the GR walls, and in the case of FF_{GR}, enhanced drainage. The greater drainage rate in FF_{GR} was also able to expedite the rate of Δu dissipation and redistribution after strong shaking, as shown in Figure 6. The latex surrounding the panel walls in $FF_{GR,L}$ restricted drainage as planned, resulting in a slower post-shaking dissipation rate even compared to FF_1 at lower elevations.

During the first significant Kobe-L motion, FF_{GR} experienced a reduction of 19% in permanent surface settlement compared to FF_1, as depicted in Figure 6. We attributed this reduction in net surface settlements to the reduction in the duration of liquefaction, reducing the contribution of sedimentation ($\varepsilon_{p\text{-}SED}$), despite the increase in volumetric strains due to partial drainage ($\varepsilon_{p\text{-}DR}$). Additional surface settlements in both locations during the subsequent Joshua-H motion were smaller than Kobe-L due to altered soil properties (e.g., soil densification) after the first motion. In particular, FF_1 experienced greater densification prior to the Joshua-H motion compared to FF_{GR} and therefore exhibited smaller surface settlements, by about 13% compared to FF_{GR}. Settlements in $FF_{GR,L}$ are neither presented nor discussed in this paper, because the LVDTs on the surface of $FF_{GR,L}$ showed heave during shaking, owing to the undrained response of the gravel layers within latex. In contrast, the post-test excavation of soil model after multiple shaking events and adequate time for drainage indicated that the soil inside the grids of $FF_{GR,L}$ experienced settlement that was similar in magnitude to the FF_1 location. Due to the observed contrast between transient and long term surface deformations at this location, $FF_{GR,L}$ settlements are not presented in Figure 6.

3.2 Influence of Gravel-Rubber Panel Walls on Response of Structure A

In this section, we compare the response of Structure A when placed on the gravel-rubber system (A_{GR}) with the unmitigated case (A_{UM}) and the cases mitigated with PVDs (A_{DR}) and in-ground structural walls (A_{SW}).

As expected, the soil beneath Structure A_{UM} generated large Δu and experienced liquefaction soon after Kobe-L shaking began. Use of PVDs around A_{DR} did not notably alter the peak value of Δu under the center of the foundation (Figures 7e and 7m), but it significantly reduced Δu near the foundation edges during shaking, which were within the drains' radius of influence (Figure 7f). Nevertheless, PVDs increased the rate of dissipation (both under the center and edge) compared to A_{UM} and other mitigated structures after shaking. The presence of SWs, on the other hand, substantially increased net Δu generation compared to A_{UM} by inhibiting lateral flow away from the foundation soil and slowing down vertical flow (Olarte et al. 2017). The results in Figures 7a through 7d and 7i through 7l show that the Δu response below A_{GR} was remarkably similar to that in the corresponding far-field FF_{GR} during both motions. We hypothesize that the greater stiffness of GR walls compared to the surrounding Ottawa sand led to a greater transfer of structure's gravity loads and dynamic demands (moment and shear stress) to the GR panels below the foundation, making the near-field soil response similar to the far-field, FF_{GR}. As a result, the soil below A_{GR} essentially had an initial effective stress consistent with FF_{GR}, and hence, liquefaction (r_u=1.0) was achieved in its underlying soil. This response was not observed below A_{SW}, because the structural walls were not directly below or attached to the foundation.

The GR panel walls successfully reduced foundation's seismic settlement and rotation relative to the unmitigated structure A_{UM} during both motions, as shown in Figure 8. The greater degree of strength loss below the center of A_{UM} compared to A_{GR} activated deviatoric ($\varepsilon_{q\text{-}BC}$ and $\varepsilon_{q\text{-}SSI}$) and volumetric ($\varepsilon_{p\text{-}DR}$ and $\varepsilon_{p\text{-}SED}$) deformation mechanisms during Kobe-L, leading to its significant permanent settlement and rotation during shaking. Changes in soil density and geometry during the second motion reduced the difference in settlements among structures, but A_{UM} continued to rotate more than others.

Among the mitigated cases, settlement of Structure A_{GR} was greater than A_{DR} and A_{SW} during Kobe-L. The same Structure A_{GR} settled similarly to A_{SW} and less than A_{DR} during Joshua-H, but this motion did not start with similar soil properties and geometries below the foundation. Despite the GR system being designed to combine some of the positive attributes of PVDs and SWs, its relatively poor settlement response was the result of a number of important differences. First, as discussed above, unlike A_{DR} and A_{SW}, we hypothesize that the structure's (Structure A_{GR}) gravity load and seismic demands was mostly taken by the GR panel walls. Under this assumption, loose Ottawa sand experienced liquefaction below the foundation of A_{GR}, amplifying its seismic deformations (both volumetric and shear type mechanisms) compared to A_{SW} and A_{DR} in Kobe-L. During Joshua-H, the soil below A_{DR} also liquefied and experienced sedimentation ($\varepsilon_{p\text{-}SED}$) and large shear strains, producing the swapped trends in settlement. Second, by enhancing drainage, GR also amplified $\varepsilon_{p\text{-}DR}$ compared to A_{UM} and A_{SW}, but not as much as A_{DR}. Third, the GR system provided lateral stiffness against shear deformations in the foundation soil relative to A_{DR} and A_{UM}, but not to the same extent as A_{SW}. Compared to A_{SW}, greater shear strains ($\varepsilon_{q\text{-}BC}$ and $\varepsilon_{q\text{-}SSI}$) were observed during excavation in loose Ottawa sand immediately below A_{GR} within the top gravel layer (shown in photographs later in the paper). The net effect of these mechanisms was a greater settlement of A_{GR} compared to both A_{DR} and A_{SW} during Kobe-L, but less settlement compared to A_{DR} during the second, stronger Joshua-H motion.

Structure A_{GR} experienced similar permanent and transient rotations to A_{DR}, but less than A_{UM} during both motions. The gravel layers in GR were expected to provide reinforcement against shear deformations. However, the inertial moment and shear demands from the superstructure induced shear deformations near the top of the GRs, resulting in notable permanent rotations during both motions. Overall, with their greater shear stiffness, SWs were most successful in limiting the rotation of structures like A, despite generating larger Δu in the Ottawa sand within. A more detailed discussion of the mechanisms of settlement and rotation in A_{SW} and A_{DR} was provided by Olarte et al. (2017) and Paramasivam et al. (2018a).

Test results during Kobe-L showed that the mitigated structures generally experienced greater foundation and roof accelerations compared to A_{UM}, as illustrated in Figures 9 and 10. However, A_{GR} experienced the lowest foundation transverse and roof accelerations of the mitigated cases during both motions, roughly approaching the unmitigated case, with even slightly lower energy at some frequencies. This result shows that the GR system successfully reduced the soil-mitigation system's shear stiffness, lengthened the system's fundamental period, and, importantly, increased the system's damping characteristics, leading to reduced transverse accelerations imposed on the foundation and roof, and what we are referring to as isolation effects. In contrast, A_{SW} experienced the greatest amplifications of foundation and roof accelerations over a wider range of frequencies (0.5 – 1.2 Hz) due to the shear reinforcement provided by SWs.

As shown in Figure 11, the presence of GR panel walls under A_{GR} amplified total drifts in the superstructure slightly compared to A_{UM} (particularly during Kobe-L). This increase was primarily associated with greater rocking (or transient rotation) of the foundation, as opposed to flexural deformations. Flexural drifts, which influence the level of damage imposed on the superstructure, were controlled and kept as low as the unmitigated case, due to the GR's flexibility and damping capabilities.

Flexural drifts in A_{GR} were lower than A_{SW} in both motions. The presence of PVDs around A_{DR} did not notably alter the drifts compared to A_{GR} during Kobe-L, but amplified both total and flexural drifts during the Joshua-H motion, which had notable content near the building's fundamental mode. We note that in this test series, no structure became inelastic or permanently deformed, due to the large lateral strength of structures like A relative to the applied seismic demand.

Figure 12 shows the transient lateral displacement profiles of both unmitigated and mitigated structures like A during Kobe-L at different time instances. Figure 12 shows an approximately rigid body translation for A_{UM}, compared to a fundamental "frame" mode deformation pattern in A_{DR} and A_{SW}. Compared to A_{DR} and A_{SW}, the GR system successfully retained some of the isolation characteristics of the unmitigated case, and exhibited more of a rigid body translation response. These results are confirmed by Figure 13, which shows that the bending strains recorded at the column fuses were roughly similar between A_{UM} and A_{GR}.

Overall, as summarized in Figure 14, use of GRs below Structure A_{GR} reduced its permanent settlement and rotation compared to the unmitigated counterpart. In comparison with PVDs and SWs, foundation settlement and rotation of A_{GR} appeared to depend on the initial soil properties and ground motion characteristics. However, the GRs effectively isolated the structure and amplified damping of the soil-mitigation system, reducing the transverse acceleration and deformation demand imposed on the superstructure relative to the other mitigated cases, regardless of the motion characteristics.

3.3 Influence of Structural Properties on the Performance of GR Panel Walls

In this section, we experimentally evaluate the consequences of using GRs under a heavier, taller, more flexible, and weaker 9-story structure (B) than A. This study attempts to understand the limits of the GR panel wall system for taller buildings; this is similar to the known difficulties with base isolation techniques that cause resonance for longer-period structures (Naeim and Kelly 1999).

We first revisit the Δu measurements in Figure 7(a through d and i through l), now considering B_{GR}. Test results for B_{GR} during Kobe-L showed a similar rise in Δu among FF_{GR}, A_{GR}, and B_{GR}, indicating that even the heavier Structure B_{GR} did not affect Δu in Ottawa sand below the foundation inside the panel walls. This trend further confirmed the observation that structural gravity loads were transferred primarily to the GR panel walls, rather than the sand layer below the foundations. The results also show that the soil under the center of B_{GR} (particularly in the middle of loose Ottawa sand, Figure 7c) exhibited a slight reduction in Δu after 12 s during the Joshua-H motion. This reduction in Δu was due to shear-induced soil dilation exacerbated by the overturning failure of B_{GR} during this motion (as discussed later in this section).

Despite its greater foundation pressure and inertial demand, B_{GR} underwent similar or smaller average seismic settlements compared to A_{GR}, as shown in Figures 8a and 15a. This similarity is attributed in part to the greater compressive and shear stiffness of the GR panel walls (as both rubber and gravel had pressure-dependent properties), as well as a greater encasement provided by geotextile surrounding the panel walls under the confinement of B_{GR}, which reduced the contribution of $\varepsilon_{q\text{-}BC}$ to its net settlement. In addition, the greater embedment depth of B_{GR} compared to A_{GR} further decreased $\varepsilon_{q\text{-}BC}$.

Unlike the trends with settlement, Structure B_{GR} underwent significantly greater foundation rotations than A_{GR}, as illustrated in Figure 15. Figure 8 showed that rotations below B_{GR} accumulated over a longer period of time compared to A_{GR}, even after strong shaking, when the foundation's average settlements had become constant. The significant permanent rotation of B_{GR} was the result of soil-mitigation-structure interaction. As the majority of this structure's gravity load was transferred to the panel walls, the soil inside the grids under B_{GR} experienced significant strength loss and likely liquefaction. Meanwhile, the greater bearing pressure, inertial mass, and height of B_{GR} imposed larger inertial moments and shear stresses to the panel walls compared to A_{GR}. These demands from the superstructure simultaneously induced: 1) large seismic moments about the foundation that redistributed the vertical pressure to one side of the GR panel walls, while reducing the pressure and shear stiffness on the other side momentarily; and 2) large shear deformations in the liquefied Ottawa sand inside the grid as well as the top gravel layers within the panel to one side. These inertial effects created an asymmetric concentration of shear strains within the soil in the first cycle, which accumulated rotation during and after shaking. These rotations had a compounding or second-order effect, as additional rotations induced greater moments on the columns and foundation due to P-Δ effects (Wilson and Habibullah 1987), which subsequently amplified rotation further. In addition, uplift caused a gap between soil and foundation to occur on one side. The gap was then filled by the surrounding soil as the structure rotated, resulting in accumulation of rotations without further average settlements, as shown in Figure 15a. The foundation rotation continued to accumulate during Joshua-H, eventually leading to its overturning failure. The large inertial demands from B_{GR} caused bulging of the top gravel layers in the GR system, as shown in Figure 15b and c.

The greater average shear stiffness of GRs under the heavier weight of B_{GR} also changed the frequency content of foundation and roof accelerations. In particular, Figure 9 showed that this stiffness slightly amplified foundation's transverse accelerations compared to A_{GR}, with peak spectral energies concentrated near a frequency range of 0.8-1.4 Hz, close to Structure B_{GR}'s first two modes. In contrast, roof accelerations in B_{GR} were notably less than those in A_{GR}. This de-amplification of accelerations through B_{GR} was due to its greater flexibility and inelastic response (i.e., permanent deformations observed on beam and column fuses), as shown in Figure 13.

These results show that GR panel walls may be effective in limiting the average settlement of most structures. However, they can amplify foundation's permanent rotation due to shear deformations that can be exacerbated under large inertial moment and shear demands from the superstructure. Therefore, an in-ground GR panel wall system needs to be designed for additional moment and shear demands from the superstructure during dynamic shaking. This is particularly important near taller, heavier, and weaker structures.

4 CONCLUSIONS

This study aims to evaluate a class of in-ground mitigation techniques that combine some of the positive attributes of conventional methods - in terms of reduced ground deformations - with those of a liquefied soil - in terms of base isolation. Here, we describe a series of centrifuge tests evaluating the influence of a newly designed, in-ground, gravel-rubber (GR) grid panel wall system on the seismic performance of a layered, liquefiable deposit in the near and far-field and of two different model structures.

The GRs did not prevent liquefaction in the soil inside the grids, regardless of the characteristics of the structure or the base motion. The GR panel walls seemed to carry the gravity load and seismic demands (inertial moment and shear stress) from the building (in a similar manner to pile foundations), and the soil response approached that in the far-field in terms of excess pore pressures and degree of softening. However, the GR system expedited the drainage of excess pore pressures compared to the far-field soil or the unmitigated structures.

In general, use of GRs reduced the permanent seismic settlement and rotation of the shorter-period, lighter, and stronger Structure A compared to the unmitigated case. The GR panel walls were not laterally as stiff as in-ground structural walls (SWs). Hence, the liquefied sand inside the grids and the panel walls experienced asymmetric dynamic shear strains under the building's inertial demand, accumulating greater rotations compared to the building on SWs, but similar to the case with prefabricated vertical drains (PVDs). Similarly, the net settlement in the GR mitigated structure was greater than or equal to SW. In comparison with PVDs, GRs allowed for greater settlement during the first short duration motion with similar initial properties. The damping characteristics of GRs effectively isolated the SFS system, limiting acceleration and deformation demands in the superstructure, regardless of base motion characteristics.

Use of GRs under a taller, heavier, and more flexible 9-story structure increased the shear reinforcement characteristics compared to the same GRs under the 3-story structure, leading to similar average settlements on both structures. However, it allowed for significant rotations on the 9-story structure. The greater seismic moments and shear stress demand from the 9-story structure induced: 1) greater pressures on one side and less pressure and stiffness on the other side of the GR panel wall; 2) large asymmetric shear deformations in the soil within and top of GR panels. These effects led to the accumulation of large foundation rotations during and after shaking. Additional rotations induced greater moments on the column fuses and foundation due to P-Δ effects, causing an uplift of this structure and, eventually, its overturning failure.

Overall, the test results show that the GR panel wall system could be beneficial, roughly satisfying foundation's design objectives for the newly constructed low-rise structures. The configuration of PVDs and SWs described in more detail below is applicable to both new and existing structures, while GRs (designed in this study) under A_{GR} would likely be only practical for newly constructed structures. In addition, practical considerations and field construction of the GR panel wall system require further investigation. Further, additional design considerations (e.g., designing wall thickness as a function of seismic demand) and reinforcement (e.g., confining gravel layers with geogrids) are required to reduce shear deformations in the panel walls caused by the seismic demand from the superstructure. This becomes particularly important near taller, heavier, and weaker structures, where greater inertia and P-Δ effects can lead to dramatic consequences on the foundation. Gravel and rubber (e.g., scrap rubber tire) materials are readily available in the market and relatively cheaper compared to steel sheet pile walls or cement grout walls. Nevertheless, detailed material characterization (strength, stiffness, and damping) of scrap rubber tires would be required prior to their use in mitigation design. Further experimental and numerical studies are needed to develop a practical system for field construction and general application.

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