Impact of Spatial Variations in Permeability of Liquefiable Deposits on the Seismic Performance of Structures and Effectiveness of Drains

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ABSTRACT: Sand deposits are often stratified with thin layers of low-permeability silt. Previous studies have shown that the presence of such sharp variations in permeability could slow down the dissipation of earthquake-induced excess pore pressures and cause void redistribution and shear localization. However, the relative importance and influence of these phenomena on seismic site response, soil-structure interaction, response of foundation and superstructure, and on the effectiveness of liquefaction countermeasures is not well understood. Here, we present the results of dynamic centrifuge tests that evaluate the response of 3- and 9-story, inelastic, steel structures (A and B) founded on layered liquefiable deposits with and without a silt cap. The thin silt layer is also evaluated in terms of its influence on the effectiveness of prefabricated vertical drains (PVD) as mitigation. The results indicate that a thin silt cap may have beneficial or detrimental effects on structure's performance, particularly when evaluated in terms of foundation's permanent rotation (or tilt). Under the lighter, stronger and stiffer Structure A, concentration of shear strains in the relatively thin loose zone below the silt layer reduced Structure A's permanent rotation by 60 to 100%, compared to the same structure on the soil profile without silt. However, the greater inertial moment and shear demand on the foundation and loose zone below the silt from the heavier, weaker and more flexible Structure B initiated larger shear deformations and rotations, leading to larger dilation tendencies and a momentary reduction in excess pore pressures in the soil below. This amplified accelerations on the foundation, flexural deformations in the superstructure, and P-Δ effects that further exacerbated rotation and damage to the superstructure. The effect of PVDs was similar on both profiles, reducing the foundation's permanent settlement (by up to 57%) and tilt (by up to 49%), but the influence of silt on their performance was similar to that of unmitigated structures. These results point to the

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importance of identifying and characterizing thin inter-layers in the soil profile, together with the key properties of structure, foundation, and ground motion, when assessing and mitigating the consequences of liquefaction.

KEYWORDS: Liquefaction; Mitigation; Prefabricated Vertical Drains; Silt Interlayers; Heterogeneity; Centrifuge Modeling; Soil-Foundation-Structure Interaction.

INTRODUCTION

Significant progress has been made in recent years in our understanding of the liquefaction phenomenon and its effects on shallow-founded structures, using both 1g shaking table and centrifuge tests (Liu and Quio 1984; Liu and Dobry 1997; Hausler 2002; Dashti et al. 2010a,b; Olarte et al. 2017; Paramasivam et al. 2018). The majority of prior physical model studies, however, considered uniform or layered deposits of clean, saturated sand with a free-draining crust. In reality, it is rare to find deposits without major spatial variations in permeability. More often, granular deposits are stratified with thin layers of low permeability soils, like silts and clays (Ishihara 1985; Andrus et al. 1991; Kokusho and Fujita 2001). If an earthquake causes the stratified granular soil deposits to liquefy, the presence of these thin layers could slow down seepage of pore water from the thicker liquefiable deposits below, causing void redistribution and strain localization that could impact the performance of structures or slopes (Fiegel and Kutter 1994; Kokusho 1999; Kulasingham et al. 2004; Dashti et al. 2010a,b). These layers have also been observed to strongly influence the extent of soil softening and surficial manifestation of liquefaction during the 2010-11 Christchurch earthquake in New Zealand (Cubrinovski et al. 2017). Moreover, such permeability contrasts might influence the effectiveness of liquefaction countermeasures, especially drains. Yet, the influence of sharp contrasts in permeability on the seismic performance of liquefiable sites, realistic and potentially-inelastic structures, and liquefaction countermeasures are currently not well understood. Improved understanding is necessary for performance-based procedures for mitigation design.

This paper presents results from a series of dynamic centrifuge tests modeling shallow-founded, multi-degree-of-freedom (MDOF) structures on layered liquefiable deposits with and without a thin, low-permeability, silt cap. The model buildings used in the study were 3- and 9-story, potentially-inelastic, seismically-designed, steel, moment-resisting-frame structures on a mat and a 1-story basement, respectively. In addition, the 3-story building model was mitigated with prefabricated vertical drains (PVDs). In the first part of this paper, we describe the effects of a thin, low-permeability silt cap on far-field site response and performance of unmitigated MDOF structures with different properties, in terms of excess pore pressures, foundation settlement and rotation, and acceleration and deformation demands on the superstructure. In the second part, we focus on the effectiveness of PVDs in improving the performance of soil and structure in the presence of a silt cap, considering a configuration of PVDs that could be used for new or existing structures.

BACKGROUND AND PRIOR PHYSICAL MODEL STUDIES

Sand deposits are often stratified with sublayers of low-permeability soils as a result of natural sedimentation processes. In practice, these thin layers may be difficult to identify and characterize, and they are often treated as a uniform sand layer in design and assessment of liquefaction (Kokusho 2003). The influence of such sharp transitions in permeability, which are common in the field, on seismic site response and performance of structures is currently not well understood and often not considered during the design of liquefaction mitigation strategies.

Research is ongoing to understand how these sudden contrasts in soil permeability affect excess pore pressure generation and fluid migration, accelerations, deformations, and soil-structure interaction (SSI) during earthquakes. Boulanger and Truman (1996) illustrated theoretically the concept of void redistibution and shear localization, considering a gently sloping liquefiable layer of clean sand overlaid by a thin silt layer. Earthquake-induced excess pore pressures within liquefiable sand dissipated from the base upward, but the presence of a low-peremability silt cap slowed the upward seepage. Excess pore pressures were shown to redistribute beneath the interface between silt and liquefiable sand. This redistribution led to an increase in sand's void ratio (i.e., loosening) below the silt interface, while decreasing sand's void ratio (densification) near the base. They referred to this phenomenon as void redistribution, which could lead to localization of shear strains and deformations in the looser zone (below the silt interface). An extreme case of void redistribution could lead to the formation of a water lense below the sand-silt interface, with potentially drastric consequences.

The influenece of such a thin silt layer on a sloping liquefiable ground (even with a few degrees of inclination) has been studied using both 1g shaking table and centrifuge model tests (Fiegel and Kutter 1994; Kokusho 1999; Kulasingham et al. 2004). Kokushu (1999) performed 1g shaking table tests on saturated sandy slopes with a thin silt seam as a horizontal or inclined (parallel to slope) layer. Both conditions increased post-shaking deformations compared to the baseline test with no silt. Based on these experiments, Kokusho and Fujita (2001) attributed the extent of post-earthquake flow failure during the 1964 Niigata, Japan earthquake to void redistribution or even formation of a water lense. Kulasingham et al. (2004) performed twelve dynamic centrifuge tests on saturated sandy slopes, with and without a thin silt seam. They observed that void redistribution and shear localizations can occur when the sand layer below silt has an initial relative density (D_r) of about 35% or less. Based on these 12 centrifuge tests, they concluded that the potential for void redistribution to cause shear localization and associated slope deformations depends on the initial relative density of sand below the silt, slope geometry, and ground motion chracteristics.

Void redistribution (and, in some cases, formation of a water lense) below the sand-silt interface has also been observed in level, layered liquefiable soils in both simple 1D column tests (Kokushu 1999; Ozbener et al. 2009) and centrifuge experiments (Dobry and Liu 1992; Maharajan and Tatsuko 2013; Lee et al. 2014). Lee et al. (2014) showed that low permeability silt layers at different depths can notably alter accelerations and excess pore pressures within liquefiable sand. Far-field surface settlements measured in deposits with a silt layer were also less than those without silt.

Only a few physical model studies have investigated the influence of silt layers on the seismic response of shallow-founded structures underlain by liquefiable deposits. Liu and Quio (1984) conducted a series of 1g shaking table tests with model rigid foundations on layered liquefiable deposits, using a transparent rigid container. They observed that a water layer could form adjacent to the foundation first, which then extended laterally over a wider distance. A similar observation was reported in Dobry and Liu

(1992)'s centrifuge experiments. Subsequently, Dashti et al. (2010a) modeled single-degree-of-freedom elastic structures on a layered liquefiable deposit with a thin silt cap, and they observed localized shear deformations beneath the sand-silt interface. Nevertheless, these studies did not systematically evaluate the role of a silt cap on the response of structures with different properties (e.g., contact pressure, inertial mass, modal frequencies, embedment depth, etc.), nor the effectiveness of common mitigation strategies on such profiles.

More recent work examining mitigation includes Olarte et al. (2017; 2018a,b) and Paramasivam et al. (2018), who performed a series of dynamic centrifuge tests on realistic MDOF, potentially-inelastic, structures founded on liquefiable soil deposits, with several mitigation techniques: densification, PVDs, stiff in-ground structural walls, and combined densification with enhanced or inhibited drainage. These studies aimed to holistically evaluate the seismic performance of the soil-foundation-structure (SFS) system on liquefiable soil deposits with different mitigation strategies, but only included sandy deposits. As a result, there is a lack of physical model studies to evaluate the influence of variations in soil permeability on seismic soil-foundation-structure interaction (SFSI) and soil-foundation-mitigation-structure interaction (SFMSI). This hinders a reliable evaluation of the liquefaction hazard for different structures on realistic and layered soil deposits and the design of context-specific mitigation strategies.

CENTRIFUGE EXPERIMENTAL PROGRAM

Dynamic centrifuge tests presented in this paper were performed using the 5.5 m-radius, 400 g-ton centrifuge facility at the University of Colorado Boulder (CU). Table 1 summarizes the characteristics of the centrifuge experiments discussed in this paper, and Figure 1a shows the schematic and instrumentation layout of the tests. In this series of tests, two scaled model structures with different properties (A and B) were placed on two soil profiles that consisted of a liquefiable layer of clean sand, with and without a thin silt cap. One of the structures (A) was also evaluated with a mitigation strategy involving PVDs on each of the soil profiles. The model specimens were constructed in a flexible-shear-beam (FSB) container of length 986 mm, width 376 mm, and depth 304 mm in model scale units (Olarte et al. 2017), and spun to 70g of centrifugal acceleration. All the units reported in this paper are in prototype scale, unless otherwise indicated.

Two potentially-inelastic 3- and 9-story, special code-conforming steel moment resisting framed structures were designed and modeled for centrifuge testing by Olarte et al. (2018a), as shown in Figure 1b. The 3-story structure (A) was simplified as a 3-degree-of-freedom (3DOF) model to capture all three lateral modes of deformation (fixed-base periods of T_n = 0.58, 0.15, and 0.06 sec) 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 taller 9-story structure (B) was simplified as a 2DOF model to capture the first two, most important modes of vibration (T_n = 2.82, 0.71 sec), due to limited centrifuge overhead clearance and constructability challenges in model scale. As a result, Structure B had the inertial mass, stiffness, and base shear strength expected in a typical 9-story building, but not the base moment (due to sacrificed total height of the building) nor higher mode effects. The structures were designed to be potentially-inelastic, meaning that they could experience inelastic deformations under seismic demands that exceeded their design level, thus differing from linear-elastic or rigid model structures typically employed in centrifuge. Inelastic response in the model structures was designed to concentrate at the beam end and column base replaceable "fuses", as shown in Figures 1b and 1c. Fuses used in Structures A and B were similar, but the fuse for Structure B was thinner. As designed and modeled, Structure B was considerably weaker than Structure A (94% difference in yield base shear from static pushover analyses under fixed-based conditions). The additional details on the design of prototype structures and scaled model structures are described in the Appendix S1.

The footprint of Structures A and B were identical, but the gross bearing pressure of Structure B (187 kPa) was greater than that of A (76 kPa). The reported bearing pressure of each structure includes the weights of the superstructure, foundation or base plate, and basement when applicable. The shorter and lighter Structure A rested on a 1 m-thick mat foundation embedded to a depth of 1 m, while the taller and heavier Structure B consisted of a 1-story, hollow basement embedded to a depth of 3 m. Each test involved two structures, separated by a distance (center-to-center) of 3.5 times the width of their foundation to reduce their interaction to the extent possible. The far-field location – at the center of the container or mid-way between the two structures, as highlighted in Figure 1a – was selected to be away from the structures and container boundaries, to roughly approximate free-field site response. Nevertheless, this location was still influenced by structures and container boundaries, and hence, does not imply true free-field conditions.

Tests A_{UM}, B_{UM}, and A_{DR} simulated the response of Structures A and B when unmitigated (i.e., subscript UM) and Structure A when mitigated with PVDs (i.e., DR). These structures were placed on a layered soil profile, referred to as Profile 1 (baseline), which consisted of a 10 m-thick, dense layer of Ottawa sand ($D_r \approx 90\%$), overlaid by a 6 m-thick, loose layer of Ottawa sand ($D_r \approx 40\%$), followed by a 2 m-thick, dense, draining crust composed of coarse Monterey sand ($D_r \approx 90\%$), shown in Figure 1a. Test $A_{UM,Silt}$-$A_{DR,Silt}$, also simulated the response of Structure A when unmitigated (UM) and mitigated with drains (DR), but on a different soil profile containing a silt cap, referred to as Profile 2. In Profile 2, the top 2m-thick Monterey sand layer was replaced by a 0.5 m-thick layer of silica silt (Sil-Co-Sil 102) overlain by a 1.5 m-thick layer of the same dense Monterey sand. Test $B_{UM,Silt}$ evaluated the seismic response of the unmitigated Structure B (UM) on Profile 2 with silt. Due to a deeper embedment of Structure B, the silt layer in Test $B_{UM,Silt}$ was placed only around the foundation and not below. Only Structure A (on both soil profiles) was mitigated with PVDs, because the weaker, heavier, and more flexible Structure B was not designed to accommodate the increased seismic demand resulting from the presence of drains (Paramasivam et al. 2018).

The key properties of soil materials used in this study are summarized in Table 2. An automated sand pourer was used to dry pluviate each layer of sand to the required density (Kirkwood et al. 2018). In Tests $A_{UM,Silt}$-$A_{DR,Silt}$ and $B_{UM,Silt}$, the silica silt was first sieved uniformly over the entire area of container from a constant height and then compacted statically using a pressure of 5 kPa.

A solution of hydroxyl propyl methyl cellulose (HPMC) with a kinematic viscosity 70 times greater than that of water was used as the pore fluid, to satisfy the dynamic scaling laws (Taylor 1995). Initially, the soil model and fluid tank were kept under a vacuum pressure of 70 kPa. Then, the vacuum pressure in the fluid tank was gradually reduced through an automated control system for upward flow into the model container, similar to the procedure described by Stringer and Madabhushi (2009). The flow rate was maintained between 20-24 ml/min, to avoid flow-induced liquefaction in models consisting of sand. However, for models with a silt cap, the flow rate was reduced to 2 ml/min when the fluid front approached silt. The water table level in all the models was raised slightly above the soil surface to ensure all layers were fully saturated before and after spinning.

PVDs were placed vertically around the perimeter of mitigated Structures A_{DR} and $A_{DR,Silt}$, on each of the two soil profiles, but not underneath the structures. These PVDs extended from the middle of dense Ottawa sand to the soil surface. PVDs were configured in a triangular pattern with a center-to-center spacing of 1.2 m in prototype units, as shown in Figure 1d. PVDs were placed during model construction using the temporary steel support (as opposed to being inserted in place) to maintain the soil fabric and density among different tests for comparison. Therefore, these experiments did not mimic the densification process typically expected in the field during PVD installation. The design, construction, and properties of model PVDs and selection of the treatment depth and pattern used in these experiments were detailed by Olarte et al. (2017) and Paramasivam et al. (2018).

A series of five 1D horizontal earthquake motions was applied to the base of the container in flight using a servo-hydraulic shaking table in all tests. The ground motion series employed in this study, same as Olarte et al. (2017, 2018a) and Paramasivam et al. (2018), was selected to cover a range of amplitudes, frequency contents, and durations. Here, the test results are presented only for the first two significant motions, referred to as Kobe-L and Joshua-H, due to major changes in soil properties and geometry after Joshua-H. Table 3 summarizes the mean properties of these base motions as recorded in different tests. Figure 2 shows the acceleration and Arias Intensity time histories as well as response spectra (5% damped) of the motions recorded at the base of the container, showing relatively minor differences among different tests.

Both soil and structure models were instrumented at key locations to record accelerations, excess pore pressures, displacements, and strains in the mechanical fuses. In this paper, the residual deformations (from the displacement sensors) in the far-field soil as well as the foundation’s permanent settlements and rotations during the first Kobe-L motion were removed from the results of Joshua-H (e.g., Figures 5 and 8). Due to notable changes in the properties and geometry of both soil and structures that were observed after the first motion, Kobe-L is the most reliable and critical motion for comparison. The results of the second motion are presented, nevertheless, because of its different characteristics influencing structure's performance. Further, the results during Joshua-H should be compared with care due to changes in initial site conditions (caused by pre-shaking history detailed by El-Sekelly et al. 2016) after Kobe-L.

CENTRIFUGE TEST RESULTS AND OBSERVATIONS

Influence of a Silt Cap on Far-Field Site Response

In this section, we evaluate the effects of a low-permeability silt cap on the seismic response of a layered liquefiable soil deposit in the far-field in terms of accelerations, excess pore pressures, and settlements. Far-field results are presented only from tests involving the lighter structures (A), as the heavier Structure B exhibited greater SFSI effects on far-field soil response. Figure 3a compares the time-frequency (Stockwell) spectra (Kramer et al. 2016; Stockwell et al. 1999) of transverse accelerations and time histories of excess pore pressure ratio (r_u) in Tests A_{UM} (Profile 1) and $A_{UM,Silt}$-$A_{DR,Silt}$ (Profile 2) for the first, Kobe-L, motion. Due to highly nonlinear and nonstationary soil response during dynamic shaking, the Stockwell spectrum was selected to illustrate and quantify the time-dependent variations in the motions' intensity and frequency content. The Stockwell transform combines positive elements of the windowed Fourier transform (which delineates frequency content) and the wavelet transform (which defines the time instance when frequency content changes) to provide a more balanced resolution of motion in both time and frequency (Kramer et al. 2016; Stockwell et al. 1999). The Stockwell transform is particularly helpful in identifying the influence of liquefaction and softening (quantified by r_u) on the magnitude and frequency content of accelerations (Kramer et al. 2016). Figure 3b shows the corresponding acceleration response spectra for both Kobe-L and Joshua-H. Here, the state of liquefaction is defined as Δu at a given depth reaching the corresponding initial vertical effective stress ($\Delta u = \sigma'_{vo,FF}$, or $r_u = \Delta u / \sigma'_{vo,FF} = 1.0$).

Surface accelerations were similar in both profiles in the far-field. However, the far-field soil in Test A_{UM} showed a consistent decrease in accelerations as shear waves propagated from the base to the surface. The far-field soil in Test A_{UM} reached liquefaction at all depths during both motions (although more slowly within dense Ottawa sand). The corresponding reduction in shear stiffness and increase in damping of all soil layers in Profile 1 reduced the amplitude of accelerations, particularly in frequencies greater than about 1 Hz. In contrast, Test $A_{UM,Silt}$-$A_{DR,Silt}$ showed amplification of accelerations in the lower, dense, layer in frequencies ranging from about 1 to 3 Hz during strong shaking, and then de-amplification at higher elevations. In Test $A_{UM,Silt}$-$A_{DR,Silt}$, liquefaction was again observed in loose Ottawa sand, but the rate of excess pore pressure generation was slower in the lower dense Ottawa sand compared to Profile 1. We hypothesize that the presence of a silt cap in Profile 2 led to void redistribution in the underlying soils. In this case, void redistribution led to densification of sand at greater depths (e.g., lower elevations within dense Ottawa sand), and a loosening of sand at shallower depths (i.e., within the looser layer Ottawa sand) in the early parts of shaking. These effects led to the observed reduction in net excess pore pressures and amplification of accelerations at the greater, densified, depths (more dilative response), when compared to Profile 1. Similarly, these effects led to larger net r_u values and a slight reduction in accelerations at shallower depths (more contractive response) compared to Profile 1. In Profile 2, r_u values at the bottom of loose Ottawa sand even exceeded 1.0 (as discussed later), likely due to the extent of loosening within that layer. A similar observation of excess pore pressure accumulation at the interface of sand layers was made by Kokushu and Kojima (2002) in simple 1D column tests and by Badanagki et al. (2018) in centrifuge tests in the presence of a silt cap.

Both profiles indicated a slight amplification of accelerations from the base toward soil surface at lower frequencies (around 0.4 to 0.6 Hz), which corresponded to the effective, strain-dependent, fundamental frequency of the site (f_{so}') during these motions. Figure 3a also shows large spectral energies at higher frequencies (2-7 Hz) for short instances, which were associated with dilation and re-stiffening of liquefied sand at large excursions of shear strain.

The presence of a silt cap slowed drainage during shaking and led to void redistribution in the underlying soil, influencing the generation and redistribution of Δu as shown in Figures 3 and 4. Figure 4 compares the dynamic total head isochrones ($\Delta u/\gamma_w$, where γ_w is the unit weight of water) in the far-field of the two profiles. The head limit ($\sigma'_{vo,FF}/\gamma_w$) corresponding to the state of liquefaction is also shown. In Test A_{UM}, without a silt cap, the liquefaction front propagated in time from top to bottom of loose Ottawa sand, and later to the lower dense layer, while seepage occurred primarily from the bottom up. The presence of a silt cap in Test $A_{UM,Silt}$-$A_{DR,Silt}$ did not seem to affect the liquefaction front's propagation in the early parts of shaking, although the generation of Δu in Test $A_{UM,Silt}$-$A_{DR,Silt}$ was slightly slower than that in Test A_{UM}. The entire loose Ottawa sand layer reached liquefaction soon after t = 6.5 s. Continued shaking generated large Δu in the lower dense Ottawa sand, while triggering liquefaction within loose Ottawa sand. Meanwhile, the presence of a silt cap above restricted outward drainage. As a result, Δu in the lower dense layer began to redistribute upward and accumulate in the originally dense soil immediately below the interface of loose Ottawa sand. These effects together with the possible slight relative settlement of pore pressure transducer within this highly loosened zone led to large r_u values at the interface of the dense and loose Ottawa sand layers, which even exceeded 1.0 momentarily. As a result, the hydraulic gradients (and hence, flow potential) were reduced within the dense layer, while the gradients within the looser layer were amplified. After shaking ceased, the slow vertical seepage through the silt layer eventually dissipated Δu in the soil below and caused solidification from bottom up, similar to Test A_{UM}

Figure 5 compares the time histories of far-field settlement recorded on top of loose Ottawa sand in both soil profiles. Far-field settlements were mainly associated with volumetric mechanisms (Dashti et al. 2010a): 1) partial drainage that occurs due to transient hydraulic gradients during shaking ($\varepsilon_{p\text{-}DR}$); 2) sedimentation that occurs when soil approaches the state of liquefaction ($\varepsilon_{p\text{-}SED}$); and 3) reconsolidation that occurs as excess pore pressures dissipate after shaking ($\varepsilon_{p\text{-}CON}$). The relative contribution of each settlement mechanism was evaluated based on qualitative observations, as they cannot be quantitatively separated during experiments.

Far-field settlements measured in Test A_{UM} were significantly greater than those in $A_{UM,Silt}$-$A_{DR,Silt}$ during Kobe-L. Test results indicate that far-field soil in Test A_{UM} began to settle soon after the motion started (main contributions from: $\varepsilon_{p\text{-}DR}$ and $\varepsilon_{p\text{-}SED}$) and continued to settle for a long duration after the motion ($\varepsilon_{p\text{-}SED}$ and $\varepsilon_{p\text{-}CON}$). In contrast, far-field soil in Test $A_{UM,Silt}$-$A_{DR,Silt}$ settled mainly during strong shaking (although less than Test A_{UM}), and its rate and magnitude slowed considerably afterwards. This response was likely caused by an initial disturbance to the thin silt cap during strong shaking, which was subject to large hydraulic gradients resulting from the localization of large excess pore pressures in the soil below, increasing slightly its effective permeability (Dashti et al. 2010b). The silt subsequently recovered its higher strength and lower permeability after strong shaking, slowing down outward seepage. Hence, the contribution and rate of post-shaking sedimentation ($\varepsilon_{p\text{-}SED}$) and reconsolidation ($\varepsilon_{p\text{-}CON}$) were notably reduced in Test $A_{UM,Silt}$-$A_{DR,Silt}$ compared to A_{UM}, due to the reduced flow rate. During the stronger Joshua-H motion, SFSI effects from neighboring structures more strongly influenced soil settlements in the far-field, resulting in an initial heave followed by long-term settlement. Nevertheless, Test A_{UM} continued to show larger net far-field settlements compared to $A_{UM,Silt}$-$A_{DR,Silt}$ after shaking.

Influence of a Silt Cap on Near-Field Soil and Foundation Response

Excess pore pressures

Figures 6 and 7 show the influence of a thin silt cap on excess pore pressures (Δu) in the near-field and far-field. The initial vertical effective stress (σ'_{vo}) estimated beneath the center and edge of each foundation is also shown, determined using elastic, static analyses of the SFS system in 3D in the finite element program OpenSEES (Ramirez et al. 2018). The far-field Δu response from the tests involving both structures are also included in Figures 6 and 7, to assess the flow mechanisms and directions around both types of structures.

In contrast to the far-field soil response (Figure 4, where larger r_u's were present below the loose-dense Ottawa sand interface), the silt layer beneath Structure $A_{UM,Silt}$ redistributed Δu below the sand-silt interface. This redistribution was due to the structure's additional confinement, influencing the stress field as well as Δu generation in the underlying soil (Figure 6). Test results from the Kobe-L motion (Figures 6a ,6d, 6g,and 6e) showed that soil beneath Structure A_{UM} generated Δu primarily during strong shaking (almost reaching liquefaction in the lower half of loose Ottawa sand), followed by drainage afterwards (t > 15 s). This large Δu generated under Structure A_{UM} created a laterally outward hydraulic gradient and flow tendency away from the center of the foundation, until achieving a more homogenous distribution of

Δu in the foundation soil ($t \leq 50$ s). Meanwhile, the upward flow from lower elevations caused a slight increase in Δu at the top of loose Ottawa sand immediately after strong shaking (Figure 6a). The Δu at all locations began to dissipate rapidly after approximately 50 s. The soil beneath Structure $A_{UM,Silt}$ experienced roughly similar (though slightly smaller) Δu tendencies during and soon after shaking ($t \leq 50$ s), but a notably slower dissipation rate in the long term ($t > 50$s), as shown in Figures 6a, 6b, 6d, and 6e. In fact, due to continued upward flow from lower elevations and a slower rate of vertical dissipation through the silt layer, Δu below the sand-silt interface continued to grow both under the center and edges of the foundation after strong shaking (until $t = 150$ s in Kobe-L). In addition, Δu under the edges of $A_{UM,Silt}$ slightly exceeded the corresponding $\sigma'_{vo,A}$ below the sand-silt interface (i.e. indicating $r_u > 1$). This behavior was likely in part due to the formation of a water film below the silt layer (similar to observations of Liu and Quio 1984 and Dobry and Liu 1992), and in part due to the small relative movement of pore pressure transducers under rocking-induced dynamic stresses (which are strongest near the foundation edges).

The silt around the heavier, taller, and more deeply embedded Structure $B_{UM,Silt}$ significantly affected the extent of Δu in the underlying soil compared to B_{UM}, both during and after shaking. The soil beneath Structure B_{UM} generated notable Δu during strong shaking ($t = 4$-13 s) in Kobe-L (Figures 6a, 6d, 6e, 6g, and 6h), followed by outward flow towards the surface and far-field ($t = 13$-30 s). In contrast, the soil under Structure $B_{UM,Silt}$ showed a sudden decline in Δu after a few significant acceleration cycles. This Δu reduction was caused by shear-induced dilation tendencies of soil in response to the foundation's significant rotation (explained in detail in the following section). These effects led to a notable inward hydraulic gradient and flow tendency towards the foundation. The soil under both Structures B_{UM} and $B_{UM,Silt}$ showed a slight increase in Δu after shaking ($t > 20$ s in Kobe-L), due to vertical flow from lower elevations. However, as with structures of type A, the silt layer around Structure $B_{UM,Silt}$ slowed down the rate of outward seepage after shaking ($t > 50$ s) compared to B_{UM}.

Most of the Δu trends were roughly similar and consistent beneath all the structures during the second, longer duration Joshua-H motion. However, the peak Δu measured beneath Structures A_{UM} and B_{UM} during Joshua-H were smaller than those during Kobe-L, due to changes in soil properties (e.g., densification due to reduced void ratio) and geometry (e.g., reduction in the thickness of looser soils due to shear deformations) after the first motion. Similarly, the longer duration Joshua-H motion accumulated slightly larger Δu under Structure $A_{UM,Silt}$ compared to Kobe-L and beneath A_{UM}. The sharp drop in Δu below Structure $B_{UM,Silt}$ during shaking was similar to that in Kobe-L, but more pronounced due to notably larger foundation rotations (detailed in the next section). In addition, the Δu response below the center of Structures B_{UM} and $B_{UM,Silt}$ showed high frequency content during Joshua-H, regardless of the soil profile of interest. This was likely due partly to soil densification after Kobe-L and partly to the structure's greater ratcheting behavior during Joshua-H.

Foundation settlement and rotation

Figure 8 shows the time histories of foundation settlement and rotation of all unmitigated structures compared to the far-field. Figure 9 compares the post-test excavation pictures and surficial evidence of ejecta near structures like A, after multiple shaking events. The excavation pictures (Figures 9a through 9f) were obtained on the plane cut through the center of structures parallel to shaking, and they show the deformation of horizontal layers and vertical columns of colored sand below and adjacent to different structures.

Figure 8, in general, shows that settlements recorded on the foundation of all the unmitigated structures were greater than those in the far-field due to additional deviatoric mechanisms, i.e., partial bearing capacity loss ($\varepsilon_{q\text{-}BC}$), and soil-structure-interaction induced building ratcheting ($\varepsilon_{q\text{-}SSI}$) (Dashti et al. 2010a). Although the presence of silt below Structures $A_{UM,Silt}$ and $B_{UM,Silt}$ may have slightly reduced volumetric strains due to partial drainage during shaking ($\varepsilon_{p\text{-}DR}$), it led to void redistribution and shear strain localization (Figures 9b and 9d). The latter had different effects on the foundation settlement and rotation depending on the structure's properties and ground motion characteristics.

Structure $A_{UM,Silt}$ experienced slightly reduced net settlement compared to A_{UM} (by about 11%) during Kobe-L, while having an opposite effect during the second, stronger Joshua-H motion (increase by 35%). The silt layer was previously shown (e.g., Figures 6a, 6b, 7a, and 7b) to slightly reduce the extent of Δu generation at greater depths and amplify them at shallower depths below the sand-silt interface, due to void redistribution (particularly during Joshua-H). This accumulation of Δu below the sand-silt interface correspondingly shifted the location of large lateral deformation and shear strains upward, compared to the soil under Structure A, as shown in Figures 9a and 9b. During Kobe-L, larger Δu generation below the center of Structure A_{UM}, particularly in the middle of loose Ottawa sand where r_u approached 1.0, amplified both shear and volumetric deformation mechanisms (e.g., mainly $\varepsilon_{q\text{-}BC}$ and $\varepsilon_{p\text{-}SED}$). This led to slightly greater settlements below A_{UM} compared to $A_{UM,Silt}$, with maximum shear strains concentrated along the middle of loose Ottawa sand (Figure 9a). The degree of softening (e.g., in terms of r_u) at lower elevations (the bottom and middle of loose Ottawa sand) below Structure A_{UM} were reduced during the second motion (Joshua-H), typically not reaching liquefaction because of altered soil properties and geometry. Simultaneously, a greater degree of void redistribution (e.g., larger concentration of Δu at higher elevations) was observed under Structure $A_{UM,Silt}$ during this motion, which amplified its settlement.

During both motions, the presence of silt and shear strain localization below the silt interface notably reduced the permanent rotation of Structure $A_{UM,Silt}$ compared to A_{UM} (by 60-100%), while increasing its transient rotation or rocking (e.g., in 5-15 s) only during Kobe-L, as shown in Figure 8e and Figure S2 in the Appendix S2. However, the same Structure $A_{UM,Silt}$ experienced significant rotations during the third motion (not shown in this paper) possibly due to the formation of sand/silt ejecta around its foundation, as shown in Figure 9h.

The average settlement of weaker, more flexible, and heavier structures like B was not sensitive to the presence of silt around their basement edges, showing similar results for Structures B_{UM} and $B_{UM,Silt}$. Further, both structures settled less than their lighter and shorter counterparts (A) during Kobe-L. This was likely due to the smaller degree of strength loss below their heavier foundation (in terms of r_u) and greater embedment depth compared to structures like A (as was observed previously by Paramasivam et al., 2018).

Unlike the results for Structure A, the presence of silt around the heavier Structure $B_{UM,Silt}$ led to notably greater foundation rotations compared to B_{UM} both during and after shaking. These trends were the result of interactions among foundation rotations, excess pore pressures and accelerations in the soil below, and flexural deflections in the superstructure. The greater confining pressure, inertial mass, and height of

Structure $B_{UM,Silt}$ amplified the moments transferred to the looser region below silt during both motions. These, in turn, increased the structure's tendency to experience ratcheting and hence, larger rotational accelerations at the foundation level. When combined with larger permanent deflections in the superstructure and P-Δ effects (Wilson and Habibullah 1987), greater ratcheting led to larger cumulative rotation of $B_{UM,Silt}$ in one direction compared to both $A_{UM,Silt}$ and B_{UM}. This trend is attributed to permanent deflections in the superstructure and P-Δ effects caused by these deflections and foundation tilt, which amplify the moment about the foundation, leading to further amplification of the tilt and movement in the same direction. As a result of larger rotations, the soil below $B_{UM,Silt}$ experienced shear-induced dilation tendencies, which momentarily reduced its excess pore pressures (particularly during Joshua-H). The reduction in pore pressures, in turn, amplified both transverse and rotational accelerations on the foundation with further adverse effects on rotation at the foundation and column levels, leading to additional P-Δ effects. As shown in Figures 6a, 6d, 6g, and 6h, however, the excess pore pressures under $B_{UM,Silt}$ started to rise again after shaking ceased due to upward flow that was slowed by silt around its perimeter. This led to additional slow and long-term foundation rotations after shaking. In contrast, structures on soil Profile 1 (without silt) did not experience notable settlements nor tilt after shaking. Although present in Kobe-L, these effects were more visible during the stronger Joshua-H motion, leading to the eventual collapse of $B_{UM,Silt}$ (discussed in more detail in subsequent sections).

Foundation transverse and rotational accelerations

Figures 10 and 11 compare the time-frequency (Stockwell) and response spectra of transverse and rotational accelerations on different foundations and far-field surface in both soil profiles. Time histories of r_u below the center of each structure in the middle of loose Ottawa sand are also presented. In all cases, significant soil softening in the loose Ottawa sand damped out foundation transverse accelerations in frequencies greater than about 1.0 Hz, compared to the base motion. Typically, slightly larger spectral accelerations were recorded on the foundations compared to the far-field in frequencies ranging from 0.4-0.8 Hz during Kobe-L and 1.5-3 Hz during Joshua-H, due to additional confinement and more limited softening (in terms of r_u) below the foundations. These effects also led to notably larger transverse and rotational accelerations recorded on the foundation of heavier structures like B compared to A (particularly $B_{UM,Silt}$, due to lower Δu in the soil below compared to B_{UM}). Both types of structure experienced major amplification of motions from foundation to roof near their corresponding modal frequencies, as expected. However, smaller roof accelerations were recorded on structures like B compared to A, due to extensive structural nonlinearity.

Structures A_{UM} and $A_{UM,Silt}$ experienced similar transverse accelerations on their foundations during both Kobe-L and Joshua-H. However, the formation of a limited shear deformation zone within looser soils (due to void redistribution under silt) beneath Structure $A_{UM,Silt}$ amplified rotational accelerations compared to A_{UM} during the first motion, Kobe-L. This effect was also evident previously in Figure 8e in terms of transient foundation rotation (or rocking), amplifying the peak roof spectral acceleration (Figure 11a) and total drift demand (Figure 12) on Structure $A_{UM,Silt}$ compared to A_{UM}. Zoomed time histories of foundation rotation for all structures like A are included in the Appendix S2 (e.g., see Fig. S2). Structure $B_{UM,Silt}$ experienced notably greater transverse and rotational foundation accelerations compared to B_{UM}, particularly at frequencies between 0.4 to 2 Hz. This increase in foundation accelerations on $B_{UM,Silt}$ was affected by void redistribution and dilation-induced reductions in Δu caused by structure's excessive rotation. Figures 10g and 10h show a clear correlation between the timing of a notable r_u reduction below $B_{UM,Silt}$ and amplification of its foundation's transverse and rotational accelerations when compared to B_{UM}. Accelerations are not plotted on Structure $B_{UM,Silt}$ during Joshua-H due to its collapse.

Influence of a Silt Cap on the Response of Superstructure

Figure 12 presents the time histories of roof total and flexural drift ratios. Figure 13 shows the lateral displacement profiles along the height of structures on both soil profiles during Kobe-L at different time instances. The peak compression strains (ε_{peak}) recorded on the beam and column fuses at the time of peak flexural drift are also shown in Figure 13. Drift ratios were estimated based on the procedure explained by Karimi and Dashti (2016) using the vertical and horizontal LVDTs (for Structure B) and accelerometers (for Structures A) at the foundation and roof levels. Figures 12 and 13 show that the lighter, stiffer, and stronger Structures A_{UM} and $A_{UM,Silt}$ behaved within their elastic range (not exceeding the yield limit) during both motions, because of their greater lateral strength (relative to demand) compared to structures like B.

The yield base shear of Structure A was estimated as 3200 kN from the numerical pushover analysis (shown in the Appendix S1), and the maximum base shear experienced by structures like A in these tests was 715 kN.

During Kobe-L, shear localization beneath Structure $A_{UM,Silt}$ amplified total drifts compared to A_{UM}, which was caused primarily by transient rocking on foundation (shown in Figure 8e and Figure S2 in the Supplement Material S2) and not flexural drift. Figure 13 shows that soil softening beneath Structure A_{UM} somewhat isolated its base, resulting in relatively uniform lateral displacements within the different stories of the superstructure. However, this structure's large permanent foundation rotation (e.g., Figure 8e) distributed larger bending strains on its column fuses and smaller strains on all beam fuses. Structure $A_{UM,Silt}$ experienced less permanent rotation, but similar flexural drifts to A_{UM}, leading to a more first-mode dominated bending strain distribution: maximum demand on column and first level beam fuses.

The heavier, more flexible, and weaker structures like B experienced permanent flexural distortions, as the applied motions exceeded their design level (factor of 1.5-1.8). During Kobe-L, Structures B_{UM} and $B_{UM,Silt}$ began to yield soon after the motion commenced. The flexural drifts experienced by Structure $B_{UM,Silt}$ were significantly greater than those on B_{UM}, due to greater transverse and rotational accelerations transferred to its foundation by the underlying soil after approximately 9 s (Figures 10g and 10h), as discussed previously. In the Kobe-L motion, Structures B_{UM} and $B_{UM,Silt}$ experienced roughly similar flexural drift and lateral displacement demands up to about t = 7 s (Figures 12 and 13). Both structures subsequently yielded at the first level beam and column fuses. At t = 9.6 s, larger permanent roof displacements and foundation rotations were accumulated in Structure $B_{UM,Silt}$, causing additional P-Δ moments about its base and strains in the column fuses. Rocking and flexural drifts continued during Joshua-H, leading to the eventual collapse of $B_{UM,Silt}$. Overall, these result show that the influence of a silt cap on structural performance strongly depends on the properties of the foundation and superstructure and their interaction with soil.

Influence of a Silt Cap on the Effectiveness of Mitigation with PVDs

Following our evaluation of the influence of a silt cap on unmitigated structures, PVDs were added around the perimeter of stronger and shorter structures like A on both soil profiles. The main objective was to evaluate the effectiveness of drains in the presence of a silt cap.

Near-field soil and foundation response

Figure 14 shows the time histories of excess pore pressure beneath the center and edge of Structures A_{DR} and $A_{DR,Silt}$. PVDs along the perimeter of the foundation on both soil profiles limited the extent and duration of excess pore pressures compared to the unmitigated cases (Figure 6), particularly near the edges. The center of the structure was less affected because it was farther from the drains' radius of influence. The presence of a silt cap beneath Structure $A_{DR,Silt}$ slightly impeded the drainage rate during shaking, increasing the demand on PVDs and net excess pore pressures at the top of loose Ottawa sand compared to A_{DR}.

Both Structures A_{DR} and $A_{DR,Silt}$ settled and rotated less than their unmitigated counterparts (A_{UM} and $A_{UM,Silt}$), particularly during the first motion when soil properties and geometry were similar among all experiments (Figures 8a, 8c, 8e, and 8g). PVDs around the foundation expedited the rate of excess pore pressure dissipation roughly similarly in both profiles, reducing the extent of void redistribution and shear localization below silt (Figure 14). In this way, PVDs limited shear deformations caused by strength loss ($\varepsilon_{q\text{-}BC}$ and $\varepsilon_{q\text{-}SSI}$) below mitigated structures on both profiles. Although rapid dissipation of pore pressures simultaneously amplified volumetric strains due to partial drainage during shaking ($\varepsilon_{p\text{-}DR}$), the net effect of drains was a reduction in total foundation deformation with or without silt.

Although drains improved foundation performance on both profiles, the presence of a silt layer slightly worsened the effectiveness of drains in terms of average settlements (Figure 8c), while improving it in terms of permanent rotation (Figure 8g). Structure $A_{DR,Silt}$ on Profile 2 settled 30% more than A_{DR} on Profile 1 during Kobe-L, while the same structure experienced 46% less permanent foundation rotation. Although drains were effective in amplifying the rate of dissipation in both soil profiles (particularly after strong shaking), the presence of silt under $A_{DR,Silt}$ still led to some void redistribution and amplification of Δu at the top of loose Ottawa sand during shaking (Figure 14), which slightly amplified this structure's settlement. Concentration of a small loose zone below the sand-silt interface under $A_{DR,Silt}$, on the other hand, tended to slightly de-amplify its foundation's transverse accelerations (Figure 15) and permanent rotations (Figure 8) during both motions compared to A_{DR}. These trends were similar to those observed for unmitigated structures like A. The results show that foundation's permanent rotation in general is more sensitive to the introduction of a silt cap compared to its average settlement, with or without drains.

Figure 9e showed the soil beneath Structure A_{DR} experienced typical shear deformation patterns increasing near the middle of loose Ottawa sand closer to the foundation's center, farther from PVDs. In contrast, vertical columns closer to PVDs near the edges showed lateral deformations increasing toward the surface, essentially following the deformed shape of the PVDs. This shows that PVD stiffness influenced the deformation patterns in soil, which should be considered in future numerical simulations. Structure $A_{DR,Silt}$, as shown in Figure 9f, experienced similar deformation patterns in vertical colored sand columns near PVDs. The center columns, however, showed greater shear deformations below the sand-silt interface, confirming a small degree of void redistribution and strain localization under the foundation center away from drains. In this study, PVDs were placed during model preparation and without any densification. Installation-induced densification in the field is expected to increase soil's strength, stiffness, and resistance to liquefaction around the PVDs, further reducing displacements and increasing the seismic demand on the foundation when compared to the drainage-only case (Olarte et al. 2018b).

Response of superstructure

Figures 15 and 16 provide data to assess the response of PVD-mitigated superstructures like A, including foundation and roof accelerations (Figure 15) and the time histories of total and flexural roof drift ratio (Figure 16). During Kobe-L, total drifts in $A_{DR,Silt}$ were slightly greater than A_{DR}. This was mostly attributed to slightly larger rocking-induced drifts (as shown in Figures 8g and 15), with negligible effects on flexural deformations. During the stronger, longer duration Joshua-H motion, all drift components in $A_{DR,Silt}$ (particularly rocking and total) were less than those in A_{DR}, due to significant soil softening beneath the silt layer (i.e., approaching r_u = 1 at the top of loose Ottawa sand, as shown in Figures 15g and 15h). The influence of silt on roof spectral accelerations was roughly similar during Kobe-L, but a slight reduction was observed during Joshua-H. In general, however, these effects were relatively minor. Although the presence of drains typically amplified accelerations transferred to the superstructure compared to the unmitigated cases, roof accelerations and flexural deflections within the superstructure were not strongly influenced by the presence of silt. These effects were expected to be more pronounced in the taller, heavier, and weaker structures like B, which were not tested with drains in this study.

CONCLUSIONS

In this paper, we describe a series of centrifuge tests evaluating the influence of spatial variations in soil permeability on the response of SFS systems and on the effectiveness of PVDs as a liquefaction mitigation technique in this context. In the first part of this paper, the seismic performance of unmitigated, 3- and 9-story, inelastic, steel, moment resisting frame structures are experimentally evaluated when founded on layered liquefiable deposits with and without a thin silt cap. In the second part, the influence of PVDs is evaluated on the performance of the 3-story structure on the same two soil profiles.

The presence of a thin silt cap in general slowed down drainage during and particularly after shaking, leading to the concentration of larger excess pore pressures below the sand-silt interface, void redistribution, and shear strain localization under the stresses induced by structures. However, the extent of these phenomena and their subsequent effects on the performance of the SFS system depended strongly on the properties of soil, structure, and ground motion.

For all the structures and motions tested, the foundation’s permanent rotation was most sensitive to the presence of a silt cap. The influence of silt on other response parameters (e.g., permanent settlement, acceleration demand on the foundation, and deformations within the superstructure) depended strongly on the properties of the structure. In the case of a shorter, lighter, and stronger 3-story structure with a shallow embedment (structures like A), localization of shear strains in a smaller zone below the silt cap significantly reduced permanent foundation rotation during all motions, given that no ejecta around this structure. Although a silt cap notably reduced permanent settlements in the far-field by slowing down vertical seepage, its effects on permanent average settlements and foundation accelerations were relatively minor for structures like A. In the case of a heavier, taller, and weaker 9-story structure with greater embedment (structures like B), the silt cap led to the opposite effect, a significant increase in permanent foundation rotation, which in turn led to shear-induced dilations in the soil below. Dilation tendencies in soil led to a sharp decrease in pore pressures and increase in transverse and rotational accelerations imposed on the foundation, causing larger flexural deflections in the superstructure and P-Δ moments that further exacerbated rotations. By sacrificing the total height of model Structure B, the inertial moments and dynamic shear stresses transferred by this structure to the soil below were expected to be less than a more realistic and taller prototype 9-story structure. However, most of the key parameters affecting the structure's behavior and transfer of loads to soil were still captured. Structure B's average foundation settlement, however, was not influenced noticeably by silt.

Addition of PVDs around the 3-story structures like A successfully increased the rate of excess pore pressure dissipation below the foundation after shaking on both soil profiles. Hence, PVDs could reduce this structure's permanent settlement and rotation on both soil profiles compared to their unmitigated counterparts. The presence of silt locally increased the drainage demand on PVDs, slightly amplifying structure's permanent average settlement relative to the case without silt. However, the concentration of shear deformations in a smaller zone below silt generally continued to reduce foundation's permanent rotation (as was observed for the unmitigated structures like A).

Overall, the experimental results show that a thin, low-permeability cap above a loose layer of clean sand can have beneficial or detrimental effects on building performance, particularly when evaluated in terms of foundation tilt and flexural deflections in the superstructure. The effects were beneficial for shorter and stronger structures like A (when no soil ejecta was observed around the structure), while detrimental for taller, heavier, and weaker structures like B (in this case leading to notable evidence of ejecta, excessive rotation, and inelastic roof drifts). These conclusions are based on one type of soil profile with two different structures under two different ground motions. The potential for void redistribution, shear localization, formation of ejecta, and the subsequent effects on the SFS system are expected to depend on the properties of the entire soil profile (initial relative density, thickness, continuity, and permeability of the liquefiable sand layer or layers as well as the spatial distribution of thin silt interlayers), structure (bearing pressure, inertial mass, foundation size and shape, H/B, V_y/W, and embedment depth), and ground motion. A limited number of these parameters were evaluated in this paper, and the relative importance of others remain to be investigated in future physical and numerical model studies.

From a practical perspective, this study points to the importance of carefully characterizing thin layers of reduced permeability and considering the soil profile at a systems level together with the key properties of the structure and foundation, when evaluating the consequences of liquefaction. The system performance can subsequently be evaluated numerically in a fully-coupled, effective stress, time-domain dynamic analysis. At this time, improved simplified predictive models are needed to reliably account for void redistribution and the associated shear deformations beneath these thin interlayers on total building settlement and tilt as well as other response parameters of interest (e.g., drift, acceleration, and base shear demand). Bullock et al. (2018a;b) recently proposed simplified, semi-empirical, probabilistic models for predicting settlement and tilt of shallow-founded structures on layered deposits that consider soil and structure properties as well as the presence of a silt cap. However, additional centrifuge and numerical modeling is needed to further validate these models for a wider range of conditions (e.g., varying the spatial distribution of silt interlayers).

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Table Captions

Table 1. Summary of centrifuge testing program.

Table 2. Properties of soil layers used in centrifuge models.

Table 3. Mean [and coefficient of variation in %] of input motion properties, as measured at the base of the container in all tests.