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RETROFITTING OF PILE FOUNDATION SYSTEMS AGAINST LIQEFACTION

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SUMMARY

To understand the fundamentals of lateral spreading on pile-soil interaction, several physical modeling tests were conducted. The tests focused on the adverse effects caused by a nonliquefiable shallow soil layer, specifically on the bending response of pile group foundations. A promising retrofitting technique that can be applied locally both during and after construction was also modeled and tested. The feasibility of the retrofitting technique to reduce bending moments was investigated using model piles and an inclined laminar box. The test results indicated that the retrofitting technique can reduce the bending moments in the piles about 60-70% and the technique is relatively simple and feasible.

INTRODUCTION

Experience from case histories and research have demonstrated that once lateral spreading has occurred due to liquefaction, the nonliquefiable ground surface also influences the soil response, and plays a major role in contributing to damage of pile foundations, in addition to damage resulting from lateral spreading. Case histories have indicated that liquefaction-induced lateral spreading has caused damage not only to pile foundations but also to the superstructure of buildings and bridges. To understand the fundamentals of lateral spreading and soil-pile interaction during this phenomenon, physical modeling tests have been conducted by researchers. For example, the response of a single pile foundation (embedded in two and three-layer soil deposits) to lateral spreading has been tested using geotechnical centrifuges or 1-g shake tables [e.g., 1, 2, and 3]. These tests modeled different geotechnical conditions and foundation systems that existed and experienced damage during past earthquakes, and increased the understanding of the behavior of single pile foundations during seismic-induced lateral spreading.

The repair of deep foundation systems is costly, and in many cases impossible to do. Accordingly, it is important to protect deep foundations from damage induced by seismic effects. Often protective measures can be inexpensive compared to repair costs. There are various remedial measures, or retrofitting methods, that can be utilized against liquefaction and liquefaction-induced lateral spreading for

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geotechnical structures. The remedial measures can be applied for (i) developed, and (ii) undeveloped sites. Some of the methods enable remediation of the site soil either during or after construction, or both. In this paper, a promising rehabilitation method was investigated, intended for the retrofitting of existing deep foundation systems. Due to its simplicity, this method can also be also applied during construction of the foundation systems. The retrofitting method isolates the existing shallow nonliquefiable soil layer, which is often major source of damage, from the existing foundation system. This is accomplished by removing the nonliquefiable soil layer surrounding the pile-cap system, forming a trench, and then filling the trench around the pile cap with a soft material that can easily yield under laterally imposed loads. The fill material used in this study was a soft clay slurry. For this project, an experimental program utilizing centrifuge modeling was conducted in order to assess the potential of using the retrofitting method for group pile foundations embedded in a three-layer soil deposit. Results from two dynamic centrifuge experiments are presented with and without the proposed retrofitting method, and compared.

TESTING PROGRAM

Equipment

Physical modeling experiments were conducted using the Rensselaer Polytechnic Institute (RPI) geotechnical centrifuge facilities. The 100g-ton centrifuge machine (Figure 1a) has an in-flight radius of 3.0 m and can test a payload of up to1 ton at 100 g or 0.5 ton at 200g. The centrifuge machine is equipped with a large computer-controlled centrifuge shaker and capable of imparting one-dimensional base dynamic excitation parallel to the centrifuge axis during flight.

A laminar box simulating shear beam boundary conditions for the soil in the free-field during shaking was employed for centrifuge modeling (Figure 1b). The internal dimensions of this box are 35.5 cm (W) × 71 cm (L) × 35.5 cm (H). The container consisted of a stack of 25 rectangular aluminum rings separated by roller bearings, arranged to permit a maximum relative displacement of up to about 3.0 mm between adjacent rings in the shaking direction, with minimal friction. The box was capable of accommodating both dry and saturated soil models with a maximum lateral strain about 20 % in the shaking direction. A thin latex membrane was used to line the inside of the box and prevent leakage of the contents. At the base, this latex membrane was glued to the box to prevent relative slip of the soil model during shaking. Detailed information on the centrifuge facilities and laminar box used can be found elsewhere [4].

Materials

The soil used to make the intermediate soil layer was Nevada No.120 sand placed at a relative density of

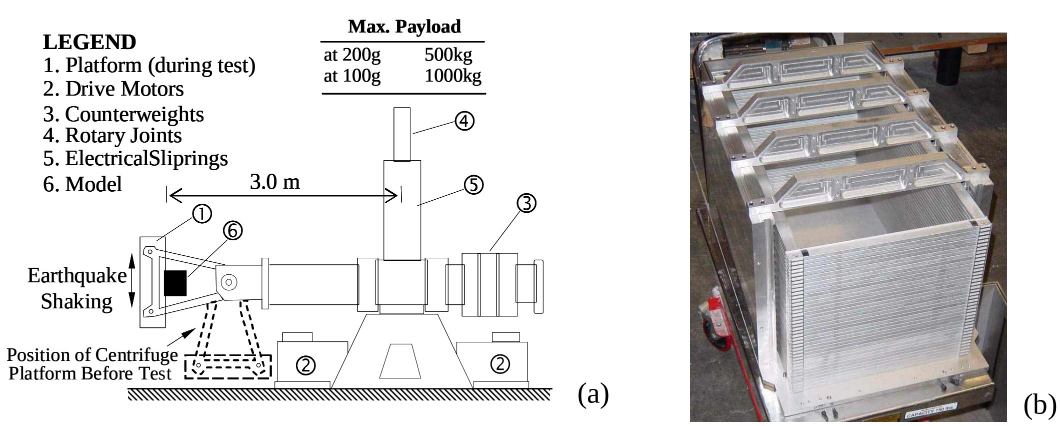

Figure 1. (a) Sketch of RPI's dynamic geotechnical centrifuge, (b) large laminar box.

Table 1. Index properties for Nevada sand.

D_{50} [mm]	0.08
D_{10} [mm]	0.15
Specific Gravity, *Gs*	2.67
Max. Void Ratio, e_{max}	0.887
Min. Void Ratio, e_{min}	0.511
Max. Dry Density [kN/m^3]	17.33
Min. Dry Density [kN/m^3]	13.87

Table 2. Index properties for kaolin clay.

Liquid Limit, *LL*	45
Plasticity Index, *PI*	10
Optimum Water Content, w_{opt} [%]	31
Max. Dry Density [kN/m^3]	13.5
c, [kPa], at w_{opt}	19.4
ϕ, [^{o}], at w_{opt}	21
Su, at *w*=50% [kPa]	<1-2

40% (D_r=40%). This is a clean, uniform, fine sand with a permeability of k=0.0021 cm/s (at 1 g) for D_r=40%. The monotonic and cyclic stress-strain response of the Nevada sand has been presented in [5], and the engineering properties of the soil are summarized in Table 1.

Naturally cemented and/or dense soil deposits or layers, as well as partially saturated sands in the field, have little liquefaction potential. Such a nonliquefiable formation was simulated using slightly cemented sand during the centrifuge tests. As shown in Fig. 2, the bottom and top cemented sand layers (Dr=~75%) consisted of a cement-Nevada sand mixture. The cementation was 10 % by weight of the clean Nevada sand. This methodology has been previously proposed and tested by Abdoun [6]. The total density for the saturated, slightly cemented sand was about 21 kN/m^3.

Kaolin clay was used to simulate a soft replacement/retrofitting material around the pile cap for Model 2 (Figure 2b). The results of laboratory tests performed to determine the engineering properties of the clay are shown in Table 2.

Model Description

Figure 2 shows a schematic of the laminar box and centrifuge soil and pile group models (Models 1 and 2), including instrumentation. All models were tested at 50 g. The prototype being simulated was a three-layer soil system with a 2x2 end-bearing pile group, with a pile cap embedded in the top cemented sand layer. In model units, the model height was approximately 20 cm. The soil deposit was fully saturated with water and inclined 2^o (in model units) to the horizontal. The prototype profile included a bottom layer of 2.0 m slightly cemented sand, followed by a 6.0 m layer of uniform Nevada sand placed at a relative density of about 40%, topped by a 2.0 m slightly cemented sand layer. At 50 g, the water saturated intermediate Nevada sand simulated a liquefiable coarse sand layer with Dr=40%, the top slightly cemented sand layer simulated a free-draining, nonliquefiable layer, and finally the bottom slightly cemented sand layer simulated a nonliquefiable medium-dense sand layer, and the free-field slope inclination was about (α_{field}) 5^o.

Each pile in the pile group had a prototype diameter of about 60 cm, length of 10 m, and bending stiffness of EI=8000 kN-m^2. The pile cap had dimensions of 2.9m (W) $\times$ 2.9 m (L) $\times$ 0.66 m (H) (in prototype units). Figure 2 also shows the details of the model pile group and pile cap. The tips of the piles in the pile group extended to the bottom of the testing box where they were embedded in the bottom cemented sand layer. The piles in the pile group were spaced three diameters center to center in both directions. In Figures 2a and 2b, pile P1 denotes the piles in the opposite direction of free-field movement, referred to also as “upslope”, and pile P2 denotes the piles located in the direction of free-field movement, referred to also as ”downslope”. Both upslope (P1) and downslope (P2) piles were equipped with strain gages to measure bending strains. The piles in the pile group were instrumented with a total of 16 pairs of half and quarter-bridge circuited strain gauges along their lengths, eight pairs of gages located on P1 and eight on P2.

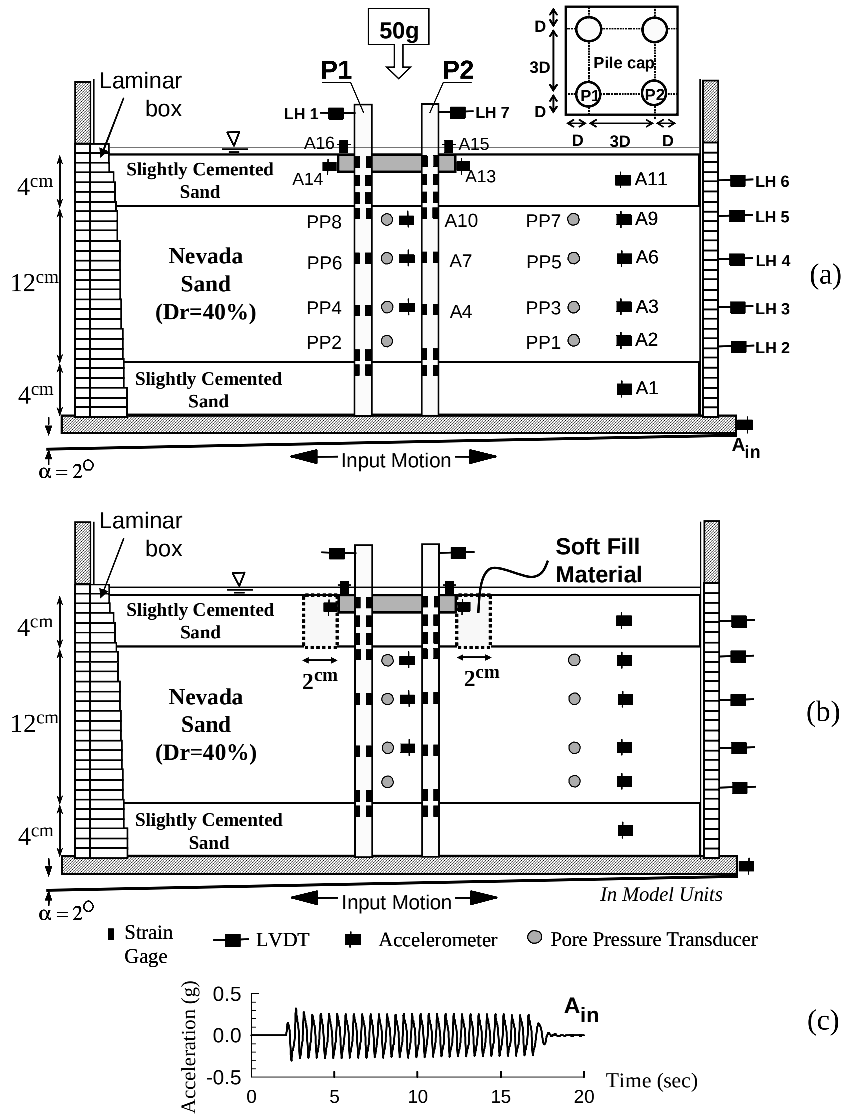

Figure 2. Setups and instrument locations of centrifuge models: (a) Model 1; (b) Model 2, and (c) prototype base acceleration.

The preparation of Models 1 and 2 (Figure 2) were the same, except that a retrofitting trench was installed around the pile cap in Model 2. As shown in Figure 2b, the retrofitting material for Model 2 was a soft clay slurry consisting of kaolin clay mixed with water which was placed in a trench surrounding the pile cap. The trench was 1 m-wide (in the shaking direction) and 2 m-deep (in prototype units). During the preparation of the top slightly cemented layer, the clay slurry was poured into the trench (at 1 g). The clay slurry had a water content of w=50% during placement into the trench. Then the model was saturated with water under a vacuum pressure of 90 kPa at 1 g. The shear strength of the kaolin clay-water mixture at this water content was determined to be less than 1-2 kPa during placement. The water content may have further increased during the saturation process of the model as well as during centrifuge spin-up. Thus, the strength of the slurry during in-flight shaking was very low, probably approaching zero. More detailed information about model construction and testing procedures can be found in Pamuk et al [7].

The models were excited by 30 cycles of a 100 Hz sinusoidal input parallel to the base of the laminar box, with a uniform acceleration amplitude of about 13 g. For the 50 g centrifuge acceleration, these values corresponded to a prototype frequency of 2 Hz and peak acceleration of about 0.26 g. The following measurements were obtained during in-flight shaking by using accelerometers (A), pore pressure transducers (PP), LVDT's and strain gages: (i) accelerations outside the laminar box, in the free-field and pile group core, and on the pile cap; (ii) excess pore water pressures in the free-field and pile group core, (iii) soil and pile lateral displacements; and (iv) pile bending strains.

RESULTS AND DISCUSSIONS

Recorded Accelerations

The acceleration recordings in the soil at different elevations and on the pile cap for Models 1 and 2 are compared in Figure 3. The acceleration records in the free-field in the intermediate layer showed a drop in positive acceleration amplitude while exhibiting large spikes in the negative upslope direction of shaking. This indicated that downslope lateral deformation was in progress after an associated loss of soil stiffness and strength due to liquefaction. Since the liquefied interlayer cannot transmit shear stresses, the acceleration record in the top cemented sand layer showed a significant drop in amplitude after about several cycles of shaking. The slightly cemented sand remained solid, and no liquefaction occurred during shaking, as expected. The acceleration in the bottom slightly cemented sand layer (not shown here) was very similar to the input acceleration, indicating that no sliding between the bottom layer and the base of the laminar box occurred. The accelerations in the pile group core showed different behavior compared to those measured in the free-field, probably due to the confining effects of the piles in the core area that prevented the soil in the pile group core flowing in the downslope direction. Acceleration of the pile cap in the horizontal direction for Model

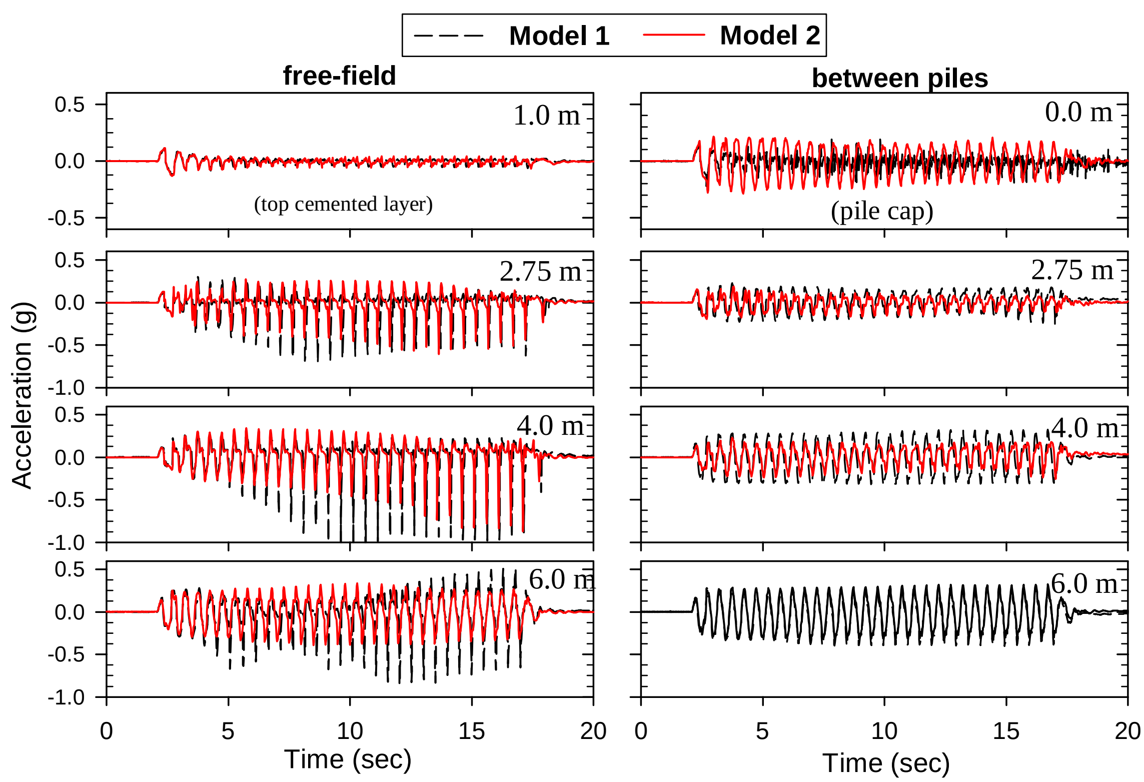

Figure 3. Comparison of accelerations measured in the horizontal direction in free-field and pile group core, and on the pile cap, Models 1 and 2.

1 was much lower in amplitude compared to the input base acceleration (Figure 2c), and consisted of many small spikes. The horizontal acceleration of the pile cap was typical of that recorded in the free-field in the middle of the top slightly cemented sand layer. The cap acceleration in Model 2 was similar to that of the base acceleration, and was much larger than that in Model 1.

Measured Displacements

Figure 4a shows the prototype lateral displacement time histories during shaking, measured by horizontal LVDT's attached to the rings of the laminar box and the pile cap for Model 1. Figure 4b compares the lateral displacement profiles in the free-field for Models 1 and 2. The lateral displacements were quite similar for each model, indicating that the free-field behavior was not affected by the soil conditions around the pile cap. Thus the tests showed a good repeatability. At all times maximum displacements occurred at the surface, and maximum displacement values of about 71 and 69 cm (prototype) were measured for Models 1 and 2, respectively.

The pile cap displacements during shaking for Models 1 and 2 are compared in Figure 5a. The cap displacements were about 70 cm for the nonretrofitted model (Model 1), and about 20 cm for the retrofitted model (Model 2). All displacements increased monotonically as the base shaking continued, except that the pile foundation of Model 2 was displaced much less than that of Model 1. The cap displacement in Model 2 increased monotonically for the first half of the shaking, and then remained unchanged through the end of shaking. This clearly indicated that the applied retrofitting method in Model 2 successfully reduced the lateral displacement of the pile heads during lateral spreading of the top cemented sand layer.

Measured Moments

The moment time histories measured during shaking are compared at the upper and lower boundaries of

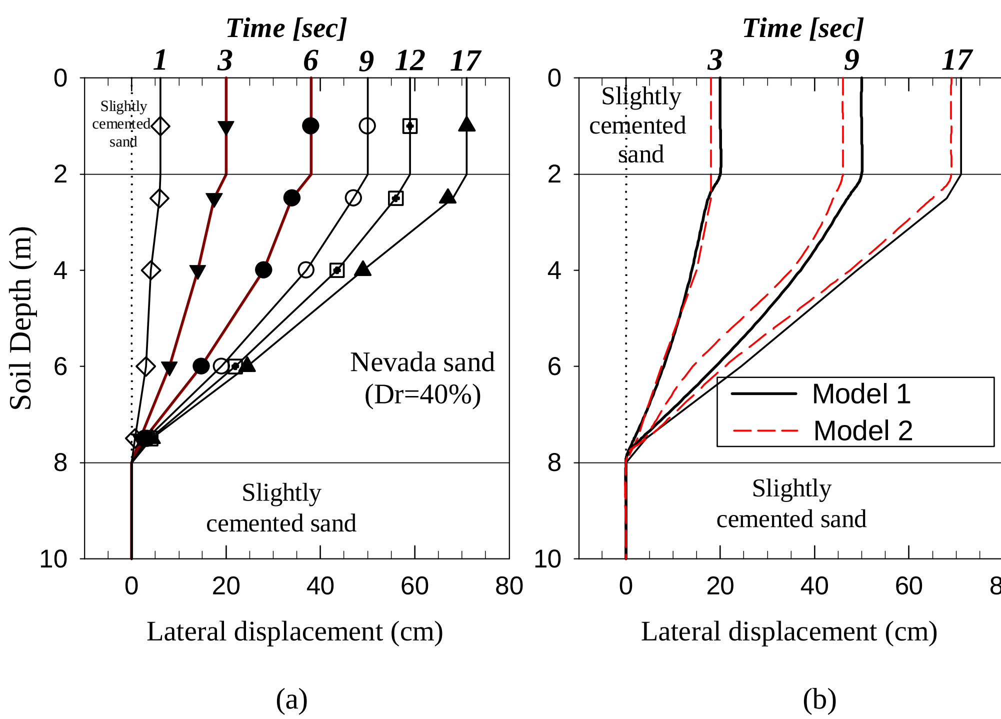

Figure 4. Free-field lateral displacement profiles at different times during shaking (a) in Model 1; (b) comparison of Models 1 and 2.

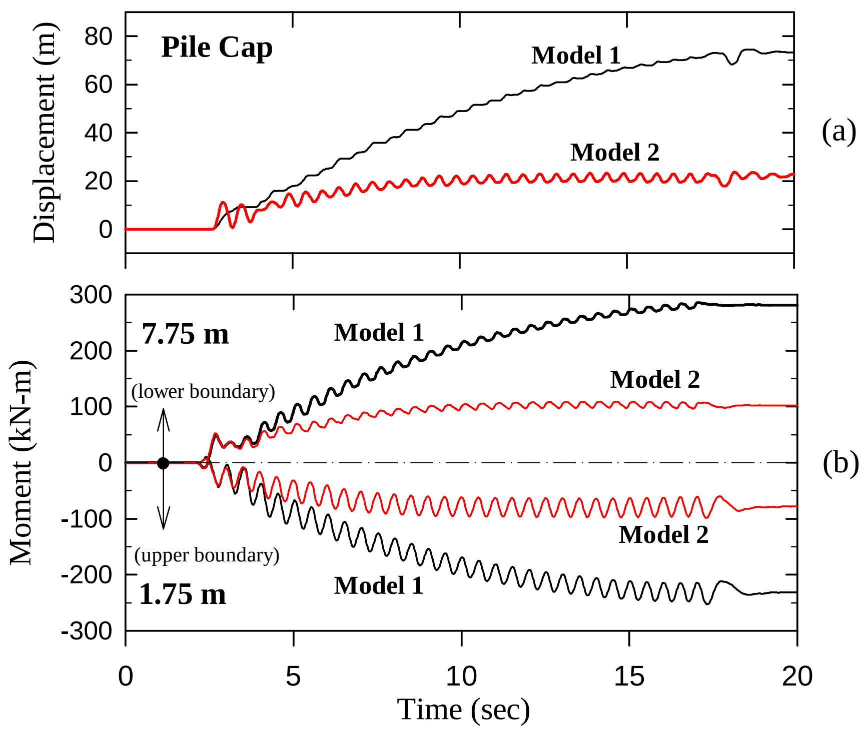

Figure 5. (a) Comparison of pile head lateral displacements in Models 1 and 2; (b) bending moment time histories measured in downslope piles in Model 2.

the liquefiable Nevada sand interlayer for downslope piles (P2) in Models 1 and 2 in Figure 5b. The moment profiles at the end of shaking are compared for both upslope (P1) and downslope (P2) piles in Models 1 and 2 in Figure 6. The moment profiles as well as time histories indicated that the moments increased monotonically during shaking in Model 1. This increase was significant for most of the shaking period, and then the rate of increase became more gradual, possibly due to the failure of the top cemented sand layer, through the end of shaking. However, in Model 2, the moments increased until the intermediate layer liquefied and failed, and then the moments remained essentially the same for the rest of the shaking period (Figure 5b).

In Models 1 and 2, the maximum moments occurred at the lower interface at all times during and at the end of shaking (Figure 6). The maximum negative bending moments occurred at the upper boundary and the maximum positive bending moments occurred at the lower boundary of the liquefiable loose Nevada sand during the early part of the shaking. As shaking continued, the bending moment profile at a given time continually exhibited a maximum at the lower boundary of the liquefied layer. However, the maximum at shallow elevations no longer occurred at the upper boundary of this layer. That is, the location of this shallow maximum moment propagated upward to the midsection of the top cemented layer (Figure 6), indicating that the nonliquefiable soil around and under the pile cap might be failing, hence causing a redistribution of the soil lateral pressure along the top of the pile group. In both models, the moments were linear in the liquefiable sand layer, and had double curvatures. The maximum moments were about 260 and 280 kN-m at the upper and lower interfaces, respectively, in the non retrofitted model (Model 1). Figure 6 shows that the retrofitting used in Model 2 significantly reduced the measured maximum moments. That is, an average moment reduction of about 70% at the shallow depths (below the pile cap) and 60 % at the bottom interface, indicating that the retrofitting used significantly reduced the imposed moments by the nonliquefiable shallow layer in the pile group.

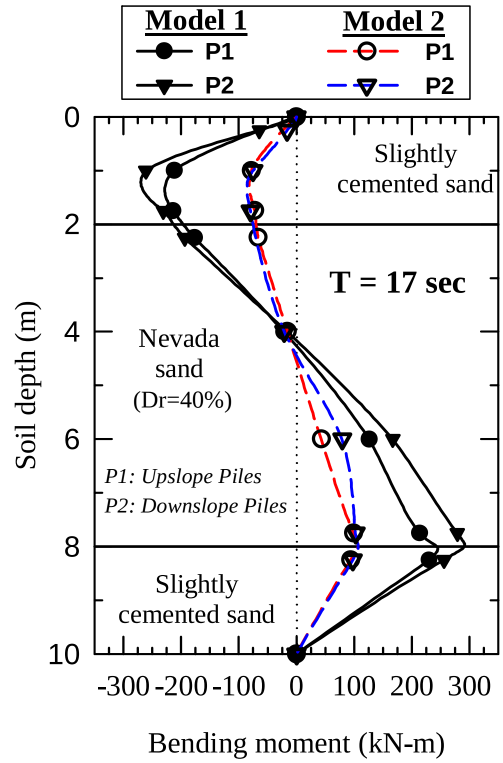

Figure 6. Comparison of measured bending profiles in upslope and downslope piles at the end of shaking for Models 1 and 2.

CONCLUSIONS

A dynamic experimental program was conducted to evaluate the potential of using a pile retrofitting technique for pile groups against liquefaction-induced lateral spreading. Two dynamic centrifuge experiments were performed to assess the reduction of adverse effects imposed by the nonliquefiable shallow layers on the pile group foundation. The results indicated that centrifuge modeling can be utilized as an effective tool to realistically simulate the soil-pile interaction during liquefaction and lateral spreading, including testing of new foundation techniques. The proposed retrofitting reduced lateral deformations of the group foundation, while providing significant reductions in the moments imposed by the nonliquefiable shallow layer. The bending profile in both nonretrofitted and retrofitted models had double curvatures, and demonstrated that the maximum moments always occurred at the lower boundary of the liquefiable interlayer. The reduction in bending moments by the implementation of the retrofitting method was about 70% at shallow depths (below the pile cap) and 60 % at the bottom interface, indicating that the lateral pressure exerted by the stiff shallow layer controlled the bending response of the pile group. The experiments indicated that the proposed method can be an effective way of retrofitting existing pile foundations embedded in multilayer soil deposits consisting of nonliquefiable shallow layers. Due to its simplicity, this method can also be also applied for the protection of new foundation systems i.e., during construction of the foundations.

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