Comparison on seismic response of rectangular closed
diaphragm wall and pile group in sloping liquefiable deposit

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Abstract. This paper presents fully coupled solid-fluid nonlinear dynamic finite element (FE) model on OpenSees platform to study the response of rectangular closed diaphragm wall (RCDW) subjected to earthquakes. Soil seismic behavior was simulated by the PDMY02 soil model available in OpenSees and soil-RCDW interaction was captured by the zero length elements with PyLiq1, TzLiq1 and QzSimple1 materials. Then, the FE numerical model was validated by data measured from centrifuge experiments. The validated numerical model was used to investigate the different seismic response between RCDW and pile group in sloping liquefiable deposit. The numerical simulation results showed that: a) Compared with pile group, RCDW can mitigate liquefaction in soil core subjected to earthquakes; b) RCDW was able to resist soil lateral spreading more effectively; c) The shear strain of soil core was restrained by RCDW which resulted in slow generation of excess pore water pressure. Therefore, RCDW had the advantage of liquefaction mitigation; d) Pile group had more deformation displacement, RCDW had more rigid displacement, resulted from the different stiffness and structural integrity between RCDW and pile group; e) Finally, it can be concluded that RCDW may be more suitable choice applied in the sloping liquefiable ground.

1. Introduction
Liquefaction-induced lateral spreading had caused extensive damage in building foundations during past severe earthquakes such as the 1964 Niigata earthquake[1], the 2008 Wenchuan earthquake[2] and the 2010 Chile earthquake[3]. Many researchers have investigated the seismic behavior of building foundations, included single pile, pile group and sheet piles.

Tarek et al.[4] investigated response of single pile and pile group in the level deposit subjected to earthquake and liquefaction-induced lateral spreading by conducting eight centrifuge experiments. Brandenberg et al.[5] studied seismic behavior of single pile and pile group in sloping liquefiable deposit by centrifuge experiments. The results showed that the loading patterns were very complex in the liquefied sloping ground. Boulanger et al.[6] investigated the seismic soil-pile-structure interaction by centrifuge experiments and proposed a dynamic nonlinear Winkler foundation analysis method for studying the soil-pile-structure interaction. Chang et al.[7] presented a two-dimensional nonlinear dynamic finite element model to study the response of soil-pile-structure system and the comparisons between numerical results and experiments data showed that the model can reproduced the centrifuge experiments well. Ramin and Ikuo[8] conducted shaking table model tests to investigate the seismic behavior of the pile group behind quay walls (sheet piles) and found that the distribution of the lateral
forces in pile group was mainly affected by the displacement and velocity of soil. Tang et al.[9] and Liu et al.[10] proposed a simplified analytical model to study the response of pile group behind quay wall based on shaking table tests. Su et al.[11] study the performance of soil-pile-quay wall system by shaking table tests, FE numerical simulation and global sensitivity analysis.

Although, piles foundations can perform well during the small earthquakes, but they will cause severe damage to the supported structures due to the small stiffness and bad structural integrity when subjected to major earthquakes. RCDW has a huge stiffness and good structural integrity which is a new type of liquefaction-mitigating foundation. RCDW was constructed by four pieces of reinforced concrete diaphragm walls connected by rigid joints. It has a hollow rectangular frame in cross section[12], as shown in Figure 1. Because of the good performance in liquefiable ground, RCDW used as foundation has been employed as deep-water bridge foundation and the offshore foundation of the cross-sea bridge the since 1979 [13]. But studies on RCDW are limited by now.

Wu et al.[14] concluded from static model tests that the bearing capacity will be improved and the settlement will be reduced by using RCDW instead of pile group in soft soils. Wu et al.[15-16] compared RCDW with pile group by the finite-difference analyses. They found RCDW can be a good option for being used as foundation in soft soil under static and dynamic loads. Li et al.[17] conducted centrifuge experiments to study the behavior of RCDW in sloping liquefiable ground and the results showed that RCDW can mitigate liquefaction of soil core.

2. Centrifuge test

The centrifuge model was tested in one-dimensional stacked-ring laminar container with internal dimensions of 600×350×449mm at a centrifugal acceleration of 50g. Hereafter, all units presented in this paper are in prototype scale unless stated otherwise.

The centrifuge model configuration is showed in Figure 2. The soil profile consisted of two soil layers with different relative density (Dr), thickness and slope angle. The upper deposit of uniform was a gentle sloping (5°) ground consisted of liquefiable Fujian standard sand with a 13-m-thick medium dense layer (Dr≈60%). The lower deposit was a level ground with a 7-m-thick dense sand layer (Dr=90%), which was non-liquefiable. The soil relative density was attained by the dry pluviation method and the experiment model was saturated by de-aired silicone oil to achieve a
viscosity of approximately 50 times that of water. The prototype RCDW is 15m high with its bottom embedded into the underlying dense sand layer, as illustrated in Figure 2. The cross section was a hollow rectangular, with external dimensions of 7.6m×7.6m and internal dimension of 6m×6m. The centrifuge model was subjected to a Taft wave with a peak base acceleration of 0.13g. More details on the centrifuge experiment can be found in Li et al. [17].

3. Numerical model

The numerical simulation is performed on Open System for Earthquake Engineering Simulation (OpenSees) which is a state-of-the-art new open-source code. OpenSees is developed to simulate the seismic response of structures in the civil engineering and geotechnical engineering. Figure 3 shows the finite element discretization of the soil-RCDW system simulated in prototype scale.

3.1. Model of sand and RCDW

The pressure-dependent multiple-yield-surface (PDMY02) constitutive model developed by Elgamal et al.[19] was used to simulate the nonlinear seismic behavior of the saturated sand. The SSPquadUP elements were applied to capture the interaction of solid and fluid. The parameters of sand in the numerical model were adopted according to recommendations of developers and the centrifuge experiment results. The parameters of PDMY02 constitutive are listed in the table 1.

The walls of RCDW were simulated by beam-column elements as illustrated in the Figure 3. In order to capture the whole deformation of RCDW, beam-column elements were connected by link elements. Elastic material was used to simulate walls and the parameters of elastic material were determined according to the RCDW in centrifuge experiments. The interaction between soil and RCDW was captured by soil-RCDW interface consisted of zero length elements with PyLiq1, TzLiq1 and QzSimple1 materials. PyLiq1 and TzLiq1 materials were accounted for lateral resistance and shaft friction, respectively. QzSimple1 material was used to compute end-bearing resistance.
Table 1. Parameters for constitutive model of sand.

| Column1                  | Dr=60% | Dr=90% |
|--------------------------|--------|--------|
| Mass density (Mg/m³)     | 1938   | 1997   |
| Reference shear modulus Gr (kPa) | 6.5×10⁴ | 1.3×10⁵ |
| Reference soil skeleton bulk modulus Br (kPa) | 1.7×10⁵ | 2.65×10⁵ |
| Friction angle           | 31     | 36.5   |
| Phase transformation angle | 31    | 26     |
| Contraction constants c1 | 0.087  | 0.013  |
| Contraction constants c3 | 0.18   | 0.0    |
| Dilation constants d1    | 0.0    | 0.3    |
| c                         | 0.85   | 0.55   |

3.2. Boundary conditions
Free field boundaries consisted of two soil columns were used to simulate the lateral boundaries conditions in the shaking direction (Figure 4). Purpose of free field boundaries was to capture the propagation of seismic waves in shaking direction. Therefore, free field boundaries were restrained to have solely pure shear deformation. Moreover, thickness of free field boundaries should be relative larger than thickness of interior soil meshes to avoid the influence of interior soil meshes response.

There was some free water above sloping ground surface which resulted in complex boundary conditions on the surface. The sloping surface condition was simulated by the hydrostatic pore pressure and nodal force induced by free water. The base of model was fixed in all direction. All the soil boundaries were set to be impervious.

3.3. Verification
Figure 4 show numerically computed and experimentally measured EPWP ratio time histories at different depths in the liquefiable layer and RCDW displacement time histories. The EPWP ratio and RCDW displacement time histories of numerical model showed reasonable agreement with experimental results in term of the trend and value. The numerical model underestimated the fluctuations of settlement and lateral displacement of RCDW. Overall, it can be indicated that the numerical model can capture seismic response of soil and RCDW in sloping liquefiable deposit.

Figure 4. Comparisons of the computed and experimental results.

4. Comparison and analysis
In this section, a numerical model of pile group (model P) was built to find the different behavior between pile group and RCDW in sloping liquefiable deposit, based on the above verified numerical model of RCDW (model R). The pile group had the same cross-section with RCDW, but there were no link elements between piles. The two individual piles were connected by a cap with 2m thick
Then, comparison and analysis of model P and model R were presented in terms of acceleration, displacement, EPWP ratio, total head and displacement of RCDW.

4.1. EPWP ratio of soil
EPWP ratio time histories of enclosed soil and far-field soil in the two models are compared in Figure 6. EPWP ratio time histories of the enclosed soil are quite different between model P and model R. In model P, the enclosed soil was liquefied at about 15s during the shaking time. However, liquefaction ($r_u \approx 1$) was not observed in model R. The EPWP ratio reached gently to maximum ($r_u \approx 0.95$) at about 35s during the shaking. At the end of shaking, EPWP of enclosed soil have a slight dissipation in both model R and model P. In the far-field, EPWP ratio time histories are similar in the two models. Far-field soil approached a liquefied state quickly at about 12s during shaking time, and kept the liquefied state during the remaining time. It can be indicated from the results that RCDW can mitigate liquefaction of the soil core (enclosed soil). Therefore, the soil core still can contribute vertical and lateral support to RCDW, resulting in less damage to supported buildings during earthquakes.

4.2. Total head of soil
Total head isochrones of far-field soil and the enclosed soil in model R and model P at different time (8s, 15s and 70s) are presented in Figure 7. In the soil core, total head of liquefiable layer in model R was smaller than that in model P at the beginning of shaking (8s). The total head of liquefiable layers in two models increased both with depth increasing and the liquefiable layers were not liquefied. In the middle of shaking (15s), liquefiable layer of model P was liquefied, while the liquefiable layer in model R was not liquefied. At the end of shaking time, the total head dissipated slightly in bottom of liquefiable layer and remained unchanged in non-liquefiable layer. In the far-field, total head of soil...
were similar in the model R and model P. The total head increased soon after the shaking began, and then approached the maximum (soil liquefied). At the end of shaking, the total head in the bottom of the liquefiable layer decreased slightly. Besides, the total head of liquefiable layer increased with the increase of soil depth. However, the total head of non-liquefiable layer kept unchanged with increase of soil depth, due its large permeability and vertical effective stress. It can be concluded from the comparison that RCDW can mitigate liquefaction of soil core. However, pile group is unable to mitigate liquefaction of the enclosed soil.

![Figure 7. Total head isochrones in model R and model P.](image)

4.3. Shear strain of soil

Shear strains of enclosed soil and far-field soil within liquefiable layer in the two models are compared in the Figure 8. In far-field, shear strains in model R and model P both showed a sharp increase after the shaking began and then basically remained unchanged. The sharp increase of shear strain was resulted from the soil softening caused by the accumulation of EPWP. The shear strain of far-field soil in model R was slight larger than that in model P. Compared with far-field, shear strain of enclosed soil was much smaller in both models due to the enclosure of foundations. Shear strain of soil core in model R was much smaller than that in model P. This difference indicated that RCDW can well restrain the soil shear deformation. This is the reason why RCDW is able to mitigate liquefaction of soil core subjected to earthquakes.

![Figure 8. Shear strain of different place in model R and model P.](image)
4.4. Lateral displacement of soil
Lateral displacements of soil in the near field and soil core (x=-6m, 0m and 6m) in the two models are presented in Figure 9. Lateral displacement was much smaller in the non-liquefiable soil and it increased quickly with depth decreasing in the liquefiable layer. Smaller lateral displacement was observed in the model R, indicated that RCDW have the advantage of resisting soil lateral spreading induced by liquefaction. This advantage is due to its huge stiffness and good structure integrity. Therefore, RCDW is able to perform well in the sloping liquefiable ground.

4.5. Stockwell transform of soil
Figure 10 shows the Stockwell transform of soil accelerations time histories in the enclosed soil and far-field in the two models. The Stockwell spectra combines elements of the windowed Fourier transform and the wavelet transform, to provide reasonable accuracy across all frequencies and a balanced resolution of the motion in both time and frequency. In this figure, the color scale is constant across all plots to facilitate comparison and analysis.
Significant deamplification of far-field soil accelerations occurred from the base toward the surface in liquefiable layer, due to soil softening. Obvious amplification of soil core accelerations was observed from the base toward the surface in liquefiable layer at the beginning of shaking, particularly in model R. This indicated that the soil core stiffness in model R was still very large, verifying the advantage of RCDW in liquefaction mitigation. Then, large EPWP generated in soil core in liquefiable layer, resulted in soil softening. Therefore, deamplification of accelerations occurred in soil core in liquefiable layer. Furthermore, the acceleration’s frequency content varied with depth in the liquefiable layer. At the bottom of liquefiable layer, the high frequency content increased in soil core and far-field of the model tests. In the top of liquefiable layer, the high frequency accelerations reduced apparently, especially in far-field. The low frequency accelerations increased more obviously in soil core in model R.

4.6. Displacement of RCWD and pile group

Figure 11 and figure 12 illustrated the displacement and deformation of foundations in the two models, respectively. The settlement of wall B was similar in the model R and model P. The fluctuation of RCDW was more obviously, due to its large stiffness. The displacement of wall A was larger in RCDW, which can be indicated that lateral displacement of RCDW may be larger than lateral displacement of pile group. In fact, the lateral displacement of RCDW was smaller than that of pile group. It can be seen from figure 16 that pile group had large flexural deformation, resulted from the liquefaction-induced lateral spreading and its small stiffness. The flexural rigidity of RCDW was much larger due to its good structural integrity. Therefore, RCDW is more suitable to be applied in the sloping liquefiable ground.

5. Conclusion

A series of centrifuge experiments were conducted to study the seismic behavior of RCDW in sloping liquefiable layered soil deposit. Based on the centrifuge experiments, a fully coupled solid-fluid nonlinear numerical model was performed on the OpenSees platform. Then, the model capabilities in capturing the response of soil and RCDW were evaluated by compared the numerical results with measured data from centrifuge experiments. The validated numerical model was used to investigate the differences between RCDW and pile group in sloping liquefiable layered deposit. Finally, the following conclusions can be made by analyzing the numerical results: a) RCDW can effectively mitigate liquefaction in soil core enclosed by it during the shaking time. However, the soil enclosed by
pile group liquefied soon after shaking began. Therefore, RCDW has the advantage of mitigating liquefaction; b) RCDW and pile group are able to resist the liquefaction-induced lateral displacement of soil. And RCDW is more effective than pile group due to large stiffness of RCDW; c) RCDW can restrain the shear strain of soil core which slowed down the generation and accumulation of EPWP, leading to liquefaction mitigation in soil core; d) The displacement and deformation of pile group and RCDW were quite different. The RCDW has larger settlement, and pile group have large lateral displacement. The deformation of piles is larger than RCDW; e) The advantage of RCDW in liquefaction-mitigation and resistance of soil lateral spreading are benefit from its large stiffness and structural integrity.

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