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A Numerical Study of Lateral Spreading Behind a Caisson-Type Quay Wall

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ABSTRACT

A series of centrifuge model tests were conducted at Rensselaer Polytechnic Institute to study the seismic response of a caisson-type waterfront quay wall system, and the liquefaction and deformation characteristics of the saturated cohesionless backfill. Using a nonlinear two-phase (solid-fluid) finite element program, a numerical study of the above centrifuge tests is performed. In this paper, the centrifuge tests and formulation of the employed finite element program are briefly described, and the numerical simulation results are compared to the experimental records. It is shown that the extent of liquefaction, the deformation pattern of the soil-wall system, and the magnitude of lateral spreading obtained from the computational code are similar to actual observations in the centrifuge tests. Computational parametric studies are then conducted by varying soil relative density and soil permeability to investigate the spatial extent of liquefaction in backfill material and its effect on the magnitude of ground lateral spreading. It is concluded that the dynamic properties and permeability of backfill material are among the most influential factors in dictating seismic performance of a quay wall system.

KEYWORS

Quay wall, Liquefaction, Centrifuge, Lateral Spreading, Soil Plasticity, Finite Element Analysis, Permeability, Earthquake

INTRODUCTION

Lateral spreading of saturated cohesionless soil behind a quay wall is one of the typical ground failure phenomena resulting from strong earthquake shaking. Extensive damage related to backfill liquefaction and quay wall failure has been observed in past earthquakes including Kobe and Taiwan. Recently, at Rensselaer Polytechnic Institute (RPI) a series of centrifuge model tests were conducted (Lee et al. 1999, Lee et al. 2000, Abdoun et al. 2001) to study the seismic response of a caisson-type quay wall system, and the liquefaction and deformation characteristics of the saturated cohesionless backfill.

Using a nonlinear two-phase (solid-fluid) finite element program, a numerical study of the above centrifuge tests was performed. In this paper, the centrifuge tests and formulation of the employed finite element program are briefly described, and the numerical simulation results are compared to the experimental records. It is shown that the liquefaction and deformation pattern of the soil-wall system, and the magnitude of lateral spreading obtained from the computational code are similar to actual observations in the centrifuge tests. Computational parametric studies are then conducted by varying soil relative density and soil permeability to investigate the spatial extent of liquefaction in backfill material and its effect on the magnitude of ground lateral spreading. It is concluded that the dynamic properties and permeability of backfill material are among the most influential factors in dictating seismic performance of a quay wall system.

DESCRIPTION OF THE TESTS

A series of three centrifuge model tests were carried out at the RPI 100 g-ton centrifuge facility (Elgamal et al. 1991). In the following, unless explicitly stated, all dimensions are in prototype scale. The model represents a prototype quay wall of 12m in height and 10m in width, supported on a loose sand foundation 6m in depth (Fig. 1). The lateral extent of the backfill is 74.6m, with the water table 1m above the ground surface. Nevada No. 120 fine sand at 40% relative density (Dr) was used as both backfill and foundation material. The D_{50} value of this sand is 0.15mm, with a permeability coefficient of 6.6×10^{-5} m/s (Lee et al. 1999). In an effort to investigate the time scaling of pore fluid dissipation within the sand, three different pore fluids were employed in the three tests, corresponding to a prototype permeability 120 times, 60 times, and 1 times that of water respectively (Lee et al. 1999, 2000).

In all three tests, the model was subjected to 20 cycles of in-plane sinusoidal base excitation at a frequency of 1Hz, with about 0.15g peak amplitude. Extensive instrumentation was deployed to record acceleration, displacement and excess pore pressure (u_e) histories in the soil, and earth pressure variation along the back and the base of the wall (Fig. 1). More detailed discussions on the experimental observations follow below. For a complete description of the tests, the reader is referred to the original experimention report (Lee et al. 2000).

NUMERICAL MODELING PROCEDURE

Modeling Background

In order to study the dynamic response of saturated soil systems as an initial-boundary-value problem, a numerical code *CYCLIC* is developed to couple these two phases. *CYCLIC* (Parra 1996, Yang 2000) is a general purpose two-dimensional (2D plane-strain and axisymmetric) Finite Element program, implementing the two-phase (solid-fluid), fully coupled numerical formulation of Chan (1988) and Zienkiewicz et al. (1990). *CYCLIC* has been employed extensively in numerical studies of post-liquefaction behavior of soil systems such as layered sloping ground and remediated earth embankments (as a liquefaction countermeasure).

CYCLIC incorporates a material constitutive model specially developed for liquefaction analysis (Parra 1996, Yang 2000). This model is based on the original framework of the multiple-yield-surface plasticity concept (Iwan 1967, and Mroz 1967), implemented by Prevost (1985) for frictional cohesionless soils. It was modified (Parra 1996, Yang 2000) from its original form (Prevost 1985) to model salient stress-strain features associated with post-liquefaction soil response. The model was previously calibrated (Parra 1996, Yang 2000) for Nevada sand at 40% relative density (the same material employed in the centrifuge quay wall test series) by extensive laboratory tests (Arulmoli *et al.* 1992) and centrifuge experiments (Taboada and Dobry 1993a, b). In this paper, the calibrated set of model parameters is adopted to represent the sand material behavior without additional modifications.

Modeling Procedure

A 4-node quadrilateral element was used for the solid as well as the fluid phases (Fig. 2). The input acceleration was prescribed at the base and side boundary nodes in the horizontal direction. The boundary conditions of the fluid phase are such that the base and two sides of the mesh are impervious, and prescribed fluid pressures were enforced along the surface nodes. Prescribed fluid pressures were evaluated depending on the water level at each individual surface node. Contact conditions between the quay wall and surrounding soil are such that the bottom of the wall is connected to the foundation soil both horizontally and vertically; the back of the wall is connected to the backfill horizontally, but vertically is free to move relative to the backfill. Friction between the wall and the soil was not modeled in this analysis. In all the numerical simulations, a Poisson's ratio of 0.33 was employed.

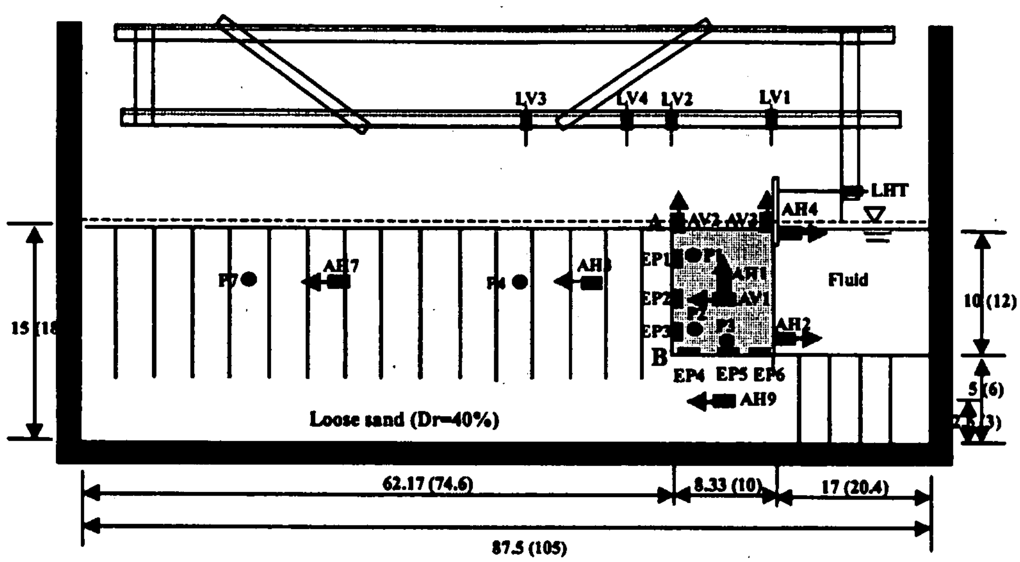

Fig. 1 Centrifuge quay wall model and instrumentation setup (model dimensions are in centimeters and prototype dimensions (in parentheses) are in meters, from Lee et al. 1999).

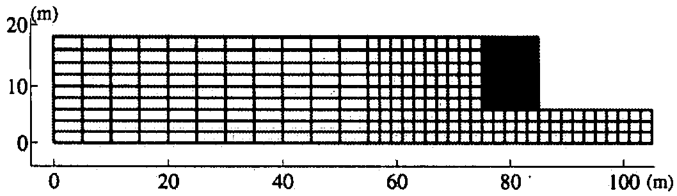

Fig. 2 Finite element mesh employed in the numerical analysis.

RESULTS AND DISCUSSIONS

In this section, the computational results employing the permeability coefficient of prototype Nevada sand (6.6×10^{-5} m/s) are presented and compared to those from the corresponding centrifuge experiment. Fig. 3 shows the permanent deformation pattern of the computational model after dynamic excitation (Fig. 4). As may be expected, more ground surface settlement is observed in the backfill near the wall than at the far field. A rigid body rotation of the wall (tilt) to the seaward direction is also clearly seen. Fig. 5 depicts the experimentally recorded and numerically computed lateral displacement of the ground surface right behind the quay wall. The recorded final permanent deformation is about 1.0m, which is only slightly underpredicted (by 5%) in the numerical simulation.

At the free field location P7, which is 47m away from the wall and 6m below the ground surface (refer to Fig. 1), both recorded and simulated pore pressure ratio r_u ($r_u = u_e / \sigma'_v$ where σ'_v is initial vertical effective confining pressure) reached 1.0 within only 2 or 3 cycles (Fig. 6). The corresponding acceleration histories (both recorded and computed) at the same place (Fig. 7) show significantly diminishing amplitudes after the first two cycles (due to liquefaction).

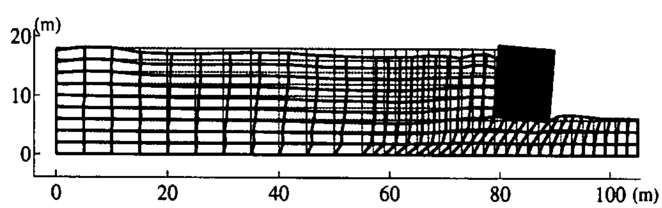

Fig. 3 Deformed mesh after numerical simulation (displacement not to scale).

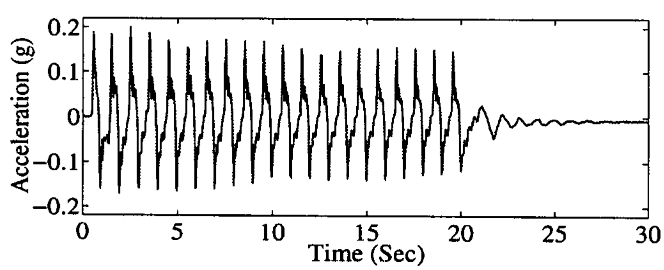

Fig. 4 Input base excitation.

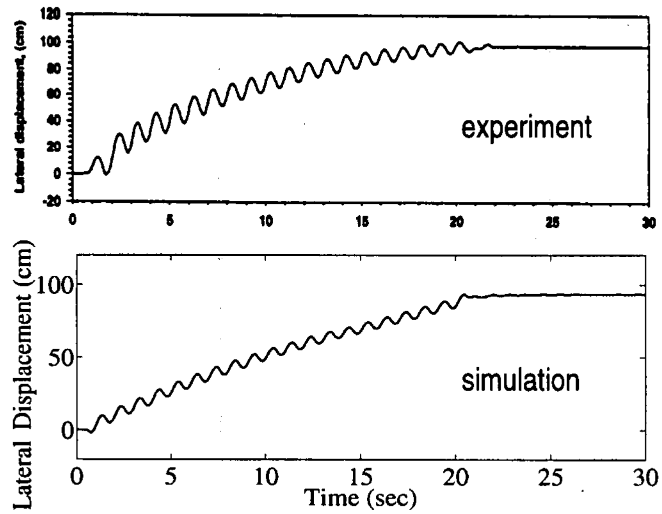

Fig. 5 Recorded vs. computed lateral ground surface displacement behind the quay wall.

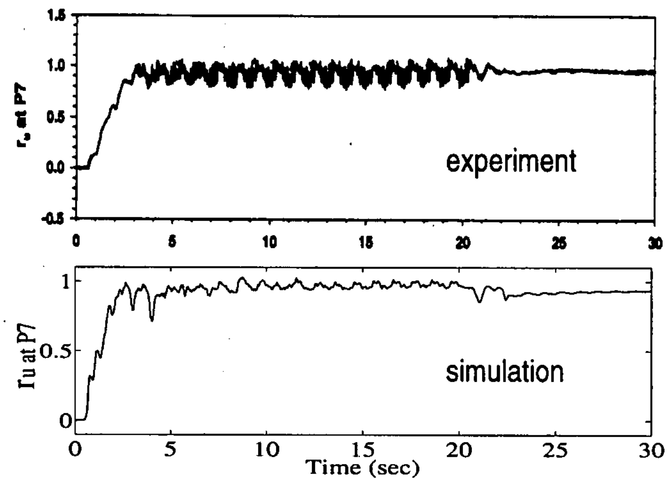

Fig. 6 Recorded vs. computed pore pressure ratio at free field location P7 (47m to the left of the quay wall).

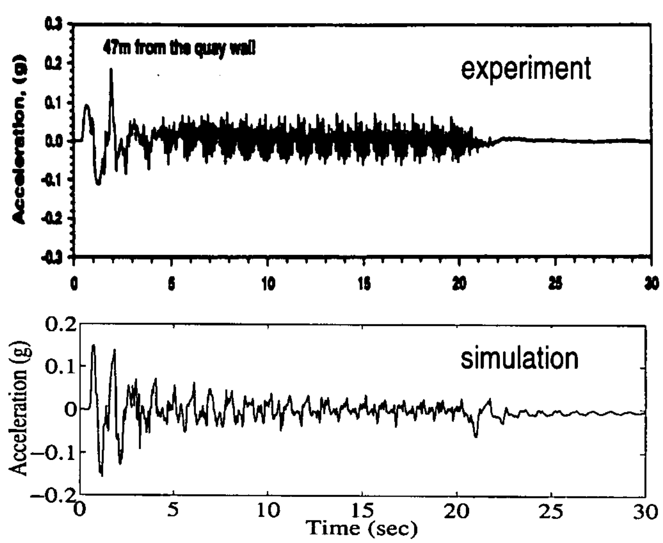

Fig. 7 Recorded vs. computed horizontal acceleration at far field location AH7 (47m to the left of the quay wall).

On the other hand, recorded and computed r_u behind the quay wall (Fig. 8) shows variation mainly within the range of –0.5 - 0.5. In addition, no significant amplitude reduction is seen (Fig. 9) in both recorded and computed acceleration histories underneath the wall throughout. Therefore, it may be concluded that liquefaction did not occur nearby (behind and under) the quay wall. In fact, even the computed acceleration history at location AH3, which is 10m away from the wall and 6m in depth (Fig. 10), does not show significant amplitude reduction indicating that no liquefaction occurred there as well (this is in agreement with the conclusion of Lee et al., 1999).

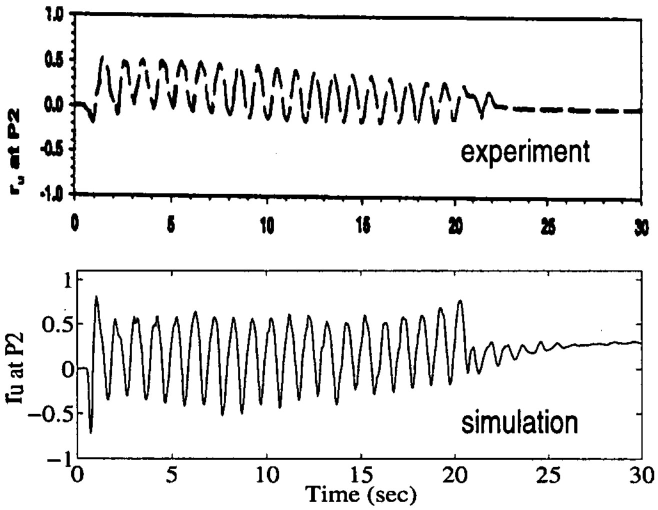

Fig. 8 Recorded vs. computed pore pressure ratio behind the wall (P2).

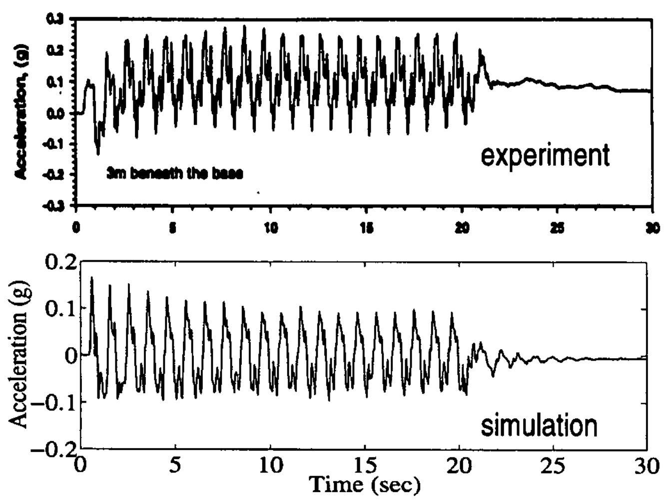

Fig. 9 Recorded vs. computed horizontal acceleration 3m below the wall (AH9).

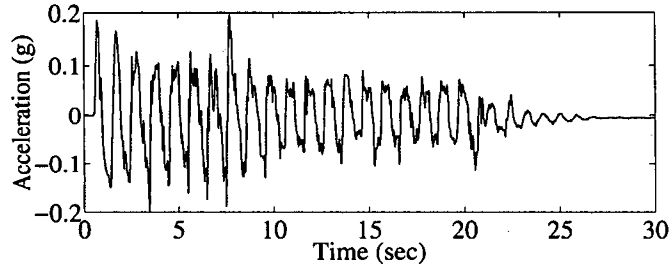

Fig. 10 Computed horizontal acceleration 10m behind the wall, at 6m depth (AH3).

As suggested by Lee et al. (1999), the difference in u_e buildup pattern between the far (free) field and near-wall field is mainly due to the fact that near the wall, soil experiences significant compression and extension alternately during the shaking (due to wall oscillation), causing u_e to oscillate between positive and negative with equivalent amplitude (Fig. 8). In the free field, soil mainly experiences shear during shaking, allowing for high u_e buildup and leading eventually to liquefaction.

PARAMETRIC STUDY

The parametric study below is focused on two factors that are directly related to liquefaction susceptibility of the soil, namely, soil relative density and permeability. Typically, u_e generation may be slower in denser sands, and u_e dissipation is faster in highly permeable materials. Therefore, a quay wall system consisting of dense backfill material with high permeability is less susceptable to liquefaction, and cosequently a better seismic performance may be expected.

Influence of Relative Density

Two additional sets of soil constitutive model parameters were selected for the backfill material, to represent medium-dense and dense sands. These two sets were selected (Elgamal et al. 1999, Yang et al. 1999) partially based on matching previously conducted cyclic laboratory tests on Nevada sand of Dr=60% (medium-dense) and Dr=90% (dense), and partially based on the authors' past modeling experience. Fig. 11 depicts the computed lateral displacement of the ground surface right behind the quay wall for the clean sand of Dr=60% and 90%, along with the response of Dr = 40% (the same as that in Fig. 5) discussed above. The final permanent deformations of the 60% Dr and the 90% Dr sands are only about one half and one quarter that of the 40% Dr, respectively.

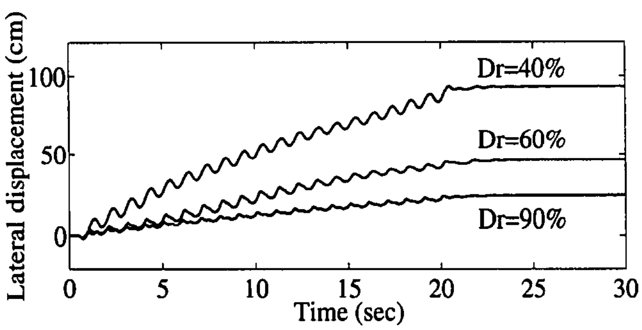

Fig. 11 Computed lateral ground displacement behind the quay wall for different backfill materials.

Fig. 12 depicts r_u histories 47m away from the wall and 6m in depth for all three materials. It is clearly seen that the denser the backfill, the slower the u_e accumulation. As mentioned earlier, the 40% Dr backfill liquefied in 2 cycles of shaking. On the other hand, Fig. 12 shows that the 60% Dr sand reached liquefaction (r_u =1) only towards the end of shaking. Finally, the 90% Dr material maintained a r_u less than 0.8 throughout. In addition, denser sands show more pronounced instants of u_e reduction, resulting from the strong tendency for dilation at large cyclic shear excursions (e.g., see Elgamal et al. 1998).

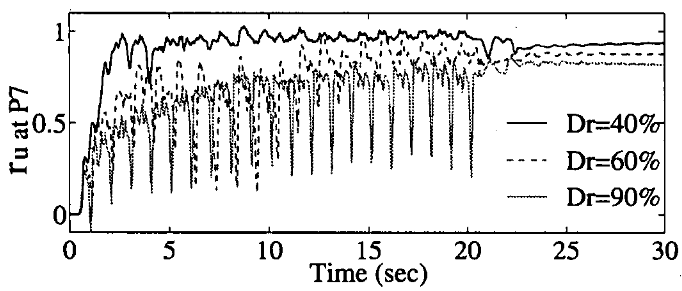

Fig. 12 Computed pore pressure ratios at far field location P7 (47m to the left of the quay wall) for three different backfill materials.

Influence of Permeability

In this case, only medium sand (40% Dr) material parameters were employed for the soil. Two additional permeability values were chosen for this parametric study, which are respectively 30 times (1.98×10^{-3} m/s, corresponding to sandy gravel) and 120 times (7.8×10^{-3} m/s, corresponding to gravel) the permeability (in prototype scale) of medium Nevada sand (as studied in the centrifuge test and numerical simulations above). Fig. 13 depicts the computed lateral displacement of the ground surface right behind the quay wall for the three permeability values. As expected, the higher the permeability, the less the accumulated permanent deformation.

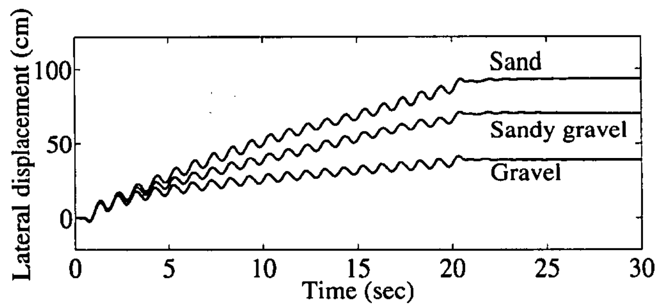

Fig. 13 Computed lateral ground displacement behind the quay wall for different permeabilities.

The recorded u_e histories at the free field location P7 (Fig. 14) show that in both the sand and sandy gravel cases, the free-field backfill quickly liquefied. However, after the shaking, u_e quickly dissipated in the sandy gravel, whereas in the sand no reduction in u_e appears long after the shaking. In the case of gravel, r_u only reached a maximum of 0.75, and the dissipation phase was completed soon after the shaking stopped.

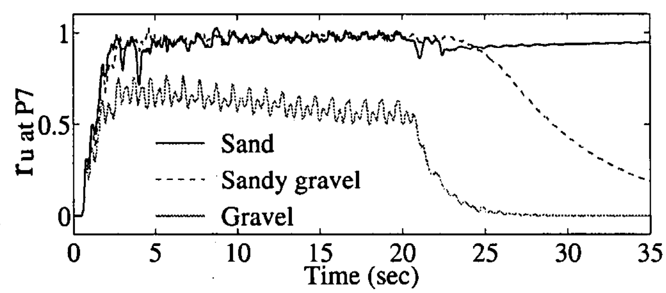

Fig. 14 Computed pore pressure ratio at far field location P7 (47m to the left of the quay wall) for three different permeabilties.

SUMMARY AND CONCLUSIONS

The procedure and results of a series of dynamic centrifuge tests on a caisson-type quay wall system were briefly described. Formulation of the finite element program employed in the numerical study was briefly outlined, along with the employed soil constitutive model. The numerical simulation results were compared to the experimental records. It is shown that the liquefaction and deformation pattern of the backfill-quay wall system, and the magnitude of lateral spreading obtained from the computational code are similar to actual observations in the centrifuge tests. Additional computational parametric studies were conducted by varying soil relative density and soil permeability to investigate the spatial extent of liquefaction and the magnitude of lateral spreading in the backfill material. It is concluded that the dynamic properties and permeability of backfill material are among the most influential factors in dictating seismic performance of the quay wall system. Increasing the relative density and/or permeability of backfill/base material can significantly improve the overall system behavior. Dense sand below and behind the quay wall may result in tolerable deformations of about 1cm for each cycle of 0.2g input excitation (in the investigated case). Free drainage (gravel) was also found to reduce deformations by a factor of 0.5 relative to a sandy soil. A combination of free drainage and high relative density would obviously be ideal. Additional experimental and numerical investigations to define the extent and required zone of remediation (by densification and/or drainage) for existing walls can be a basis for implementing liquefaction remediation efforts.

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