Container Boundary Effect on Seismic Earth Dam Response in Centrifuge Model Tests

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Abstract—**A numerical study is conducted to investigate container boundary effects on the dynamic response of earth dams in centrifuge model experiments. In particular, different container types (rigid vs. laminar) are used to show their respective influence on various dam response quantities including displacement, acceleration, and pore-water pressure. The dam model configurations and input excitation are obtained from actual physical model tests. These tests were designed to investigate the effect of foundation densification on the seismic behavior of a zoned earth dam with a saturated liquefiable sand foundation. The numerical procedure employs a solid-fluid fully coupled Finite Element code, incorporating a plasticity-based soil stress-strain model capable of simulating liquefaction and related deformations. This procedure was verified earlier through blind prediction of the above centrifuge tests. The computational results show that container type may introduce significant changes in the observed centrifuge model performance. Appropriate model containers (type and size) should be employed to reproduce field conditions with reasonable accuracy.**

Keywords—**Earth dam, earthquake, liquefaction, foundation remediation, centrifuge, Finite Element method, fully coupled analysis.**

INTRODUCTION

An important concern in physical model tests (e.g., centrifuge and 1g shake-table) is how the type (e.g., rigid vs. flexible) and size of model container affects the model response. Ideally, the resulting boundary conditions should represent actual field situations with reasonable accuracy. In an earlier paper [1], we studied numerically the size effect of a rigid centrifuge container on the dynamic response of two earth dam models. The employed dam models were based on a series of highly instrumented centrifuge tests recently conducted by Adalier and Sharp [2,3] at Rensselaer Polytechnic Institute. The preliminary results of that study suggest that, in earth dam models with a liquefiable foundation (subjected to moderate seismic loading), a larger rigid container may result in the following:

1) Noticeable reduction in acceleration and pore pressure response. This may be partially attributed to the fact that the excitation imposed by the lateral container boundaries on the dam body becomes smaller as the container size increases.

2) Insignificant change in displacements (lateral and vertical), since the reduced level of excitation effect appears to have been compensated for by the presence of a wider (and thus more flexible) foundation in the larger-container models.

In this paper, we continue to study the effect of container type on the dynamic response of four centrifuge earth dam models. The study is again based on the centrifuge test series by Adalier and Sharp [2,3]. This test series was designed to experimentally assess the performance of countermeasure techniques for liquefiable earth dam foundations. In the experimental series, seismic behavior of a zoned earth dam with a saturated sand foundation (Fig. 1) was investigated under moderate levels of dynamic excitation. The effect of various parameters on the seismic behavior of the dam, such as the thickness, width, and depth of liquefiable layer, was studied [2,3].

Blind numerical predictions were performed for four of the conducted tests [4], only knowing the physical model configuration and the input motion. A solid-fluid fully coupled Finite Element program [5,6,7] was employed in the numerical analysis. This program incorporates a soil stress-strain model that was partially calibrated earlier [5] for the same sand used in the physical model tests. The numerical prediction results, including displacement, acceleration, and pore-water pressure time histories, were generally in good agreement with the experimental data.

In the above blind numerical predictions (as well as in the physical models), a rigid centrifuge container was used. Herein, we perform a series of numerical simulations on the same set of models, with flexible lateral boundaries to mimic the effect of a laminar container (free field scenario). The computational results, including displacement, acceleration, and pore water pressure time histories, are presented and compared to the rigid-container counterpart.

CENTRIFUGE TESTING PROGRAM [2,3]

The centrifuge models (Fig. 1) simulated a prototype earth dam of 10 m in height and 39.5 m in base width, resting on a sand foundation deposit of 9 m thickness. The earth dam core was composed of Kaolin clay, and the embankment was composed of clean Nevada No. 120

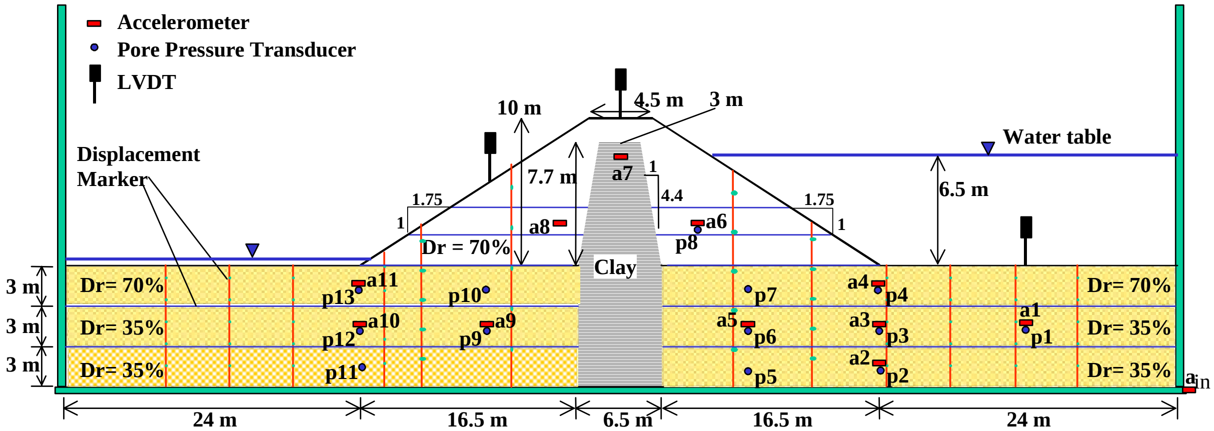

Fig. 1: Typical centrifuge model configuration (LLD case) and instrument deployment in prototype scale [2,3].

sand at a D_r of about 70%. This sand was also used as the foundation material, at a D_r of about 35% (for the non-densified zone) and 70% (for the densified zone). The foundation layer was saturated with a fluid at a prototype permeability of 1.3x10^{-4} m/s (D_r =35%) and 1.0x10^{-4} m/s (D_r =70%), within the range of fine sands. Water was used to saturate the upstream embankment and as the reservoir fluid (Fig. 1), resulting in a prototype permeability of about 6x10^{-3} m/s (within the range of coarse sands).

The four experiments (Fig. 1) were different only in thickness of the densified foundation layer, in order to evaluate overall performance of the dam-foundation system as a function of this parameter. The first case (LLL, where L stands for Loose) was the benchmark test with the entire foundation composed of loose sand (35% D_r). The other three models, LLD (D stands for Dense), LDD, and DDD, represented an increasingly thicker densified foundation layer (70% D_r) of 3 m, 6 m, and 9 m respectively (Fig. 1).

All models were subjected to similar lateral acceleration of about 30 cycles, 0.2g peak amplitude, and 1.5 Hz dominant frequency. Soil response during and after shaking was monitored (Fig. 1) by a large number of miniature accelerometers (in the horizontal direction), pore pressure transducers, LVDTs, and a dense mesh of displacement markers.

NUMERICAL MODELING PROCEDURE

Finite Element Model

The developed Finite Element (FE) program [8,9] implements the two-phase (solid-fluid) fully coupled FE formulation of Chan [10] and Zienkiewicz et al. [11]. The saturated soil system is modeled as a two-phase material based on the Biot theory [12] for a porous medium. A numerical formulation of this theory, known as *u-p* formulation (in which displacement of the soil skeleton *u*, and pore pressure *p*, are the primary unknowns [10,11]), was implemented ([4,5,6,7]). This implementation is based on the following assumptions: small deformation and rotation, density of the solid and fluid is constant in both time and space, porosity is locally homogeneous and constant with time, incompressibility of the soil grains, and equal accelerations for the solid and fluid phases.

The boundary conditions for the developed FE mesh of the dam-foundation system were [4]:

For the solid phase, lateral input motion was specified along the container base, as the recorded rigid container acceleration. Along two lateral boundaries, FE nodes at the same elevation were tied together laterally to simulate the effect of laminates in a flexible container.

For the fluid phase, the base and the two sides (i.e., the container boundaries) were impervious. The free water surface (phreatic surface) was assumed to vary linearly within the clay core between the upstream side and the downstream side. At each node along the model surface, a constant pore pressure was specified equal to the acting hydrostatic pressure.

A static application of gravity (model own weight) was performed before seismic excitation. The resulting fluid hydrostatic pressures and soil stress states served as initial conditions for the subsequent dynamic analysis.

Constitutive Model

The FE program incorporates a plasticity-based soil stress-strain constitutive model ([1,4,5,6,7]), in which a number of conical yield surfaces with different tangent shear moduli are employed to represent shear stress-shear strain nonlinearity and confinement dependence of shear strength [13,14]. This soil model was calibrated earlier for the same sand employed in the conducted centrifuge tests, at a $D_r \approx 40\%$. The calibration phase [5] included results of monotonic and cyclic laboratory sample tests [15], as well as data from level-ground and mildly inclined infinite-slope dynamic centrifuge model tests [16,17].

Liquefaction-induced deformation is among the most important criteria for evaluation/remediation of related hazards. In this regard, the employed soil constitutive model was developed with emphasis on simulating the liquefaction-induced shear strain accumulation mechanism in clean medium-dense sands [5,6,7]. Figure 2 depicts the model simulation results of an inclined liquefiable loose sand deposit ($D_r \approx 40\%$) subjected to cyclic loading. Fig. 2 shows: 1) an initial phase of gradual loss in effective confinement and thus gradual increase in pore pressure (shear-induced contraction), 2) considerable shear strain accumulating within each load cycle, as the effective confinement approaches zero (i.e., liquefaction), and 3) rapid increase in effective confinement at large cyclic shear strain excursions (shear-induced dilation), which causes increased shear stiffness and strength.

The main modeling parameters include typical dynamic soil properties such as low-strain shear modulus and friction angle, as well as calibration constants to control pore-pressure buildup rate, dilation tendency, and the level of liquefaction-induced cyclic shear strain. For loose Nevada sand (35% D_r), a set of parameter values similar to that calibrated for the 40% D_r Nevada sand [5] was used. For dense Nevada sand (70% D_r), no formal calibration phase was possible due to lack of appropriate data. Hence, the modeling parameters were chosen mainly based on engineering judgment. For the clay core, a simplified version of the same constitutive model was used, with the shear strength parameters chosen based on earlier laboratory testing results [2,3].

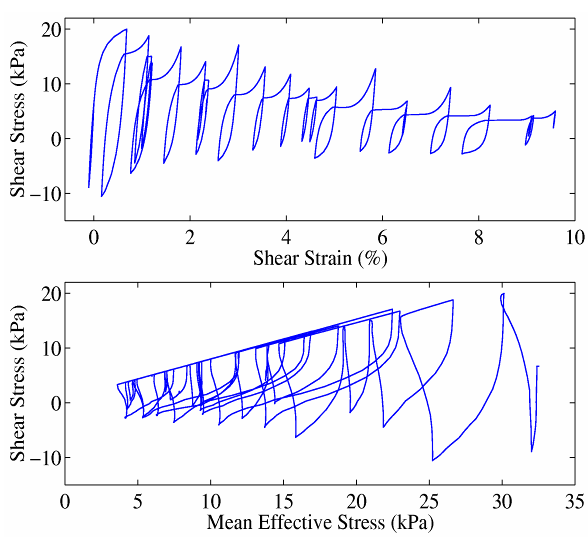

Fig. 2: Model simulation of inclined liquefiable soil deposit under undrained shaking conditions showing shear stress-shear strain and effective stress path.

RESULTS AND DISCUSSION

Deformation

Fig. 3 shows the final deformed configurations for all cases. Generally, due to the combined action of the imparted lateral excitation and weight of the dam body, foundation soil migrated laterally towards the free field. The LLL case shows significant lateral and vertical displacements in the entire loose foundation layer. The DDD case also shows uniform lateral displacements throughout the densified foundation layer, but of much smaller amplitude.

Different from the above two cases, the LLD and LDD show deformations concentrated in the loose lower layer. The densified upper foundation layer and the dam body remained essentially intact. Apparently, in these cases, the weak lower layer acted as a base-isolator effectively preventing the base shaking from upward propagation.

The final lateral displacements at the upstream and downstream dam toes from the experiments, the rigid-container model simulations (blind predictions), and the flexible-container model simulations, are shown in Fig. 4 for all cases. In Fig. 4, the sign convention is such that at the downstream toe, movement to the left is positive, whereas at the upstream toe, movement to the right is positive. It is noted that the final displacements were predicted reasonably well in the remediated cases, but were under-predicted by 40-50% in the benchmark (LLL) case. In this case, considerable lateral deformation was attributed to the shallow failure along the dam slopes (near the surface), which was not captured by the computational model [4].

The following observations can be made in Fig. 4:

1) In all cases (experiments and numerical simulations), foundation densification resulted in a significant reduction in lateral displacements.

2) In the experiments (using a rigid container), a similar level of displacement reduction was achieved in all remediated cases, regardless of the thickness of densification. In the flexible-container simulations, most reduction was achieved in the LLD and LDD, particularly at the upstream toe.

3) The displacements in the flexible-container models were much smaller than the rigid-container counterpart, for cases where a loose bottom layer was present (all except DDD).

The flexible container only allowed seismic energy to be imparted into the model through the base, much of which filtered by a loose bottom layer, as mentioned above. This is the reason why among all flexible-container models, the LLD and LDD displayed the least displacements. On the other hand, the rigid container allowed energy to be imparted not only from the base, but also from the side boundaries, resulting in stronger dynamic response (see below) and more permanent displacements.

Fig. 5 depicts measured vertical displacements (LVDT data) at the dam slope and crest, along with the corresponding simulation results (rigid- and flexible-container models). In all cases, settlement is seen to accumulate on a cycle-by-cycle basis, and the blind prediction results (rigid-container models) were in very good agreement with the experimental counterpart. Similar to the observations about the lateral displacements (Fig. 4), Fig. 5 shows that: 1) dam settlement was reduced significantly by foundation densification, and 2) the use of a flexible-container reduced settlement by about 40% (LLL) to 75% (LDD) in models with a loose bottom layer. In DDD, the settlements were almost identical to those with a rigid container.

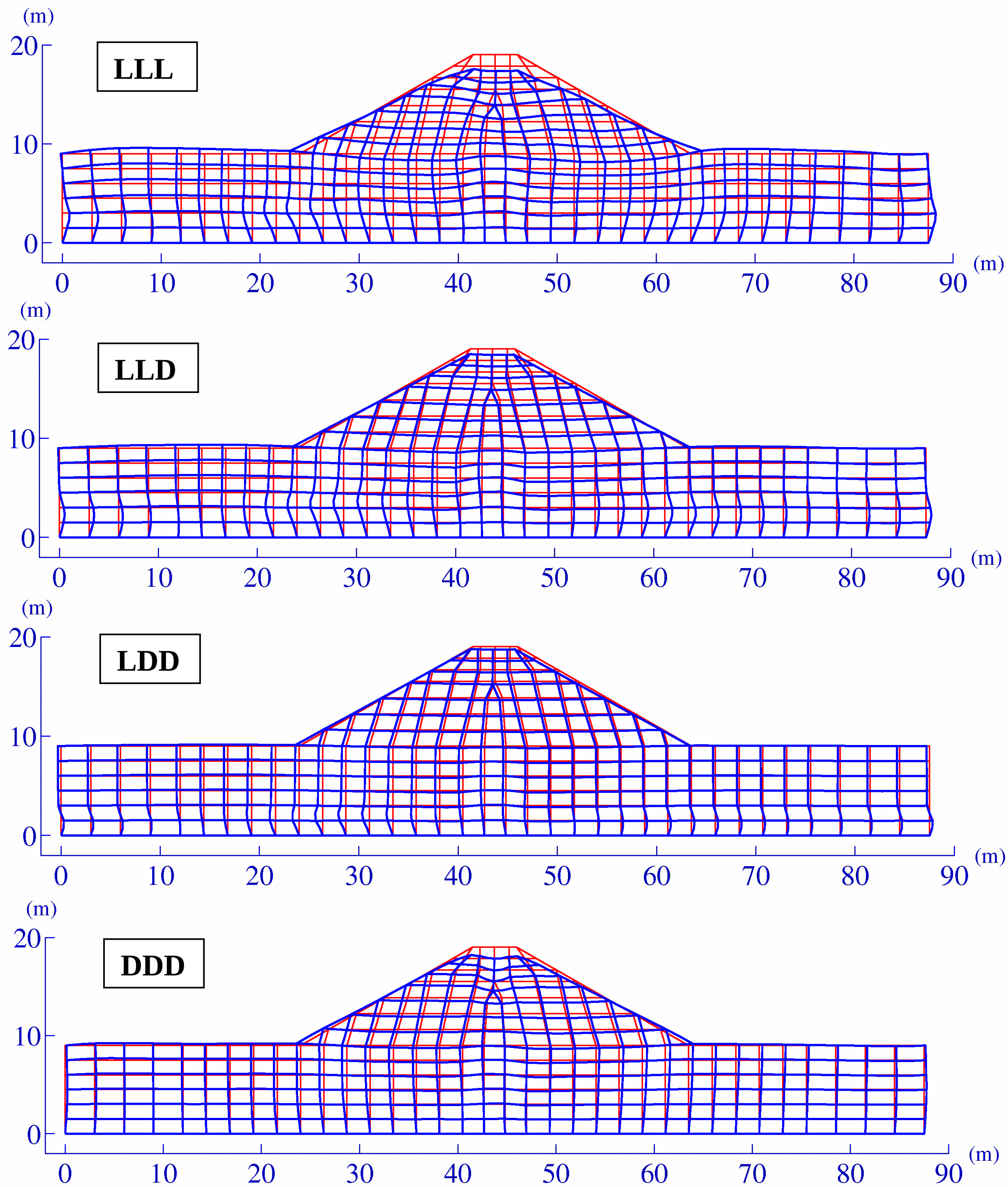

Fig. 3: Computed final deformed configurations for all flexible-container models.

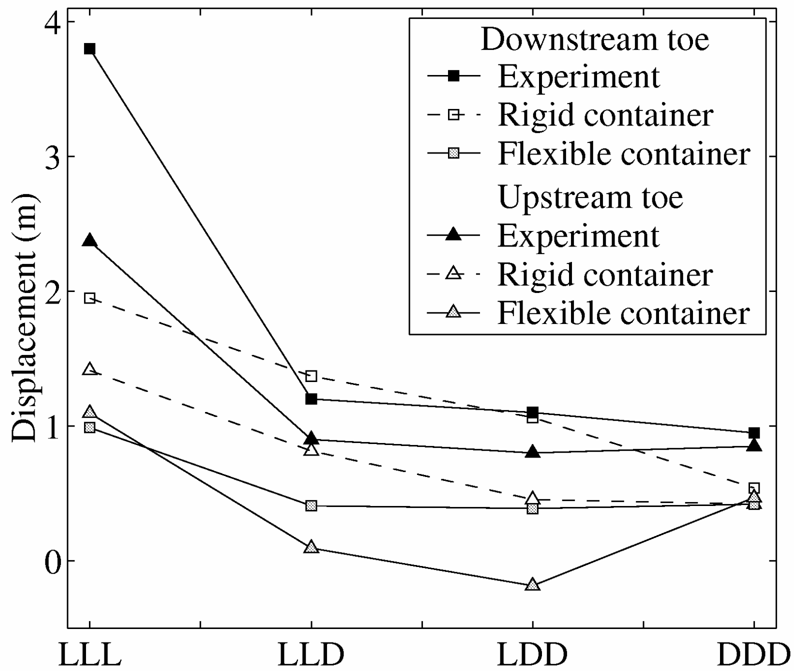

Fig. 4: Measured and computed lateral displacements at dam toes.

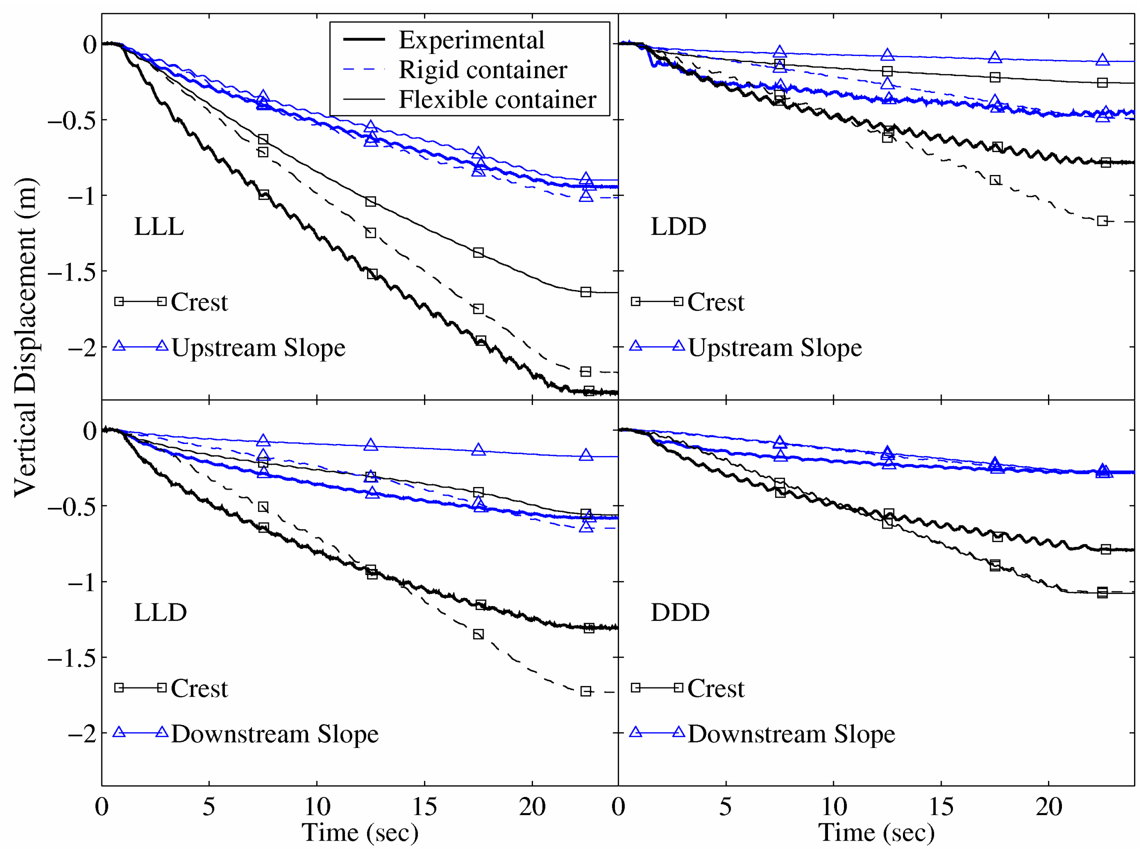

Fig. 5: Recorded and computed vertical settlement at dam slope and crest.

Acceleration and Excess Pore Pressure

Fig. 6 depicts the acceleration time histories near the dam crest (A7, Fig. 1) from the rigid- and flexible-container models. In the DDD case, the two different containers resulted in nearly identical acceleration response. In the other three cases, the flexible-container models recorded smaller acceleration amplitudes. The degree of reduction was more than 2/3 in LLD and LDD.

Fig. 7 depicts the computed excess pore pressure (u_e) response directly below the dam body (P7, Fig. 1). At this location, stretching of the foundation soil led to low or even negative u_e as manifested by the rigid-container models. In the flexible-container models, due to the reduced excitation (Fig. 6) and level of foundation lateral spreading (Fig. 4), no strong negative u_e is present. Finally, similar to all other response parameters (Fig. 4-6), the two container types resulted in essentially the same u_e response in the DDD case.

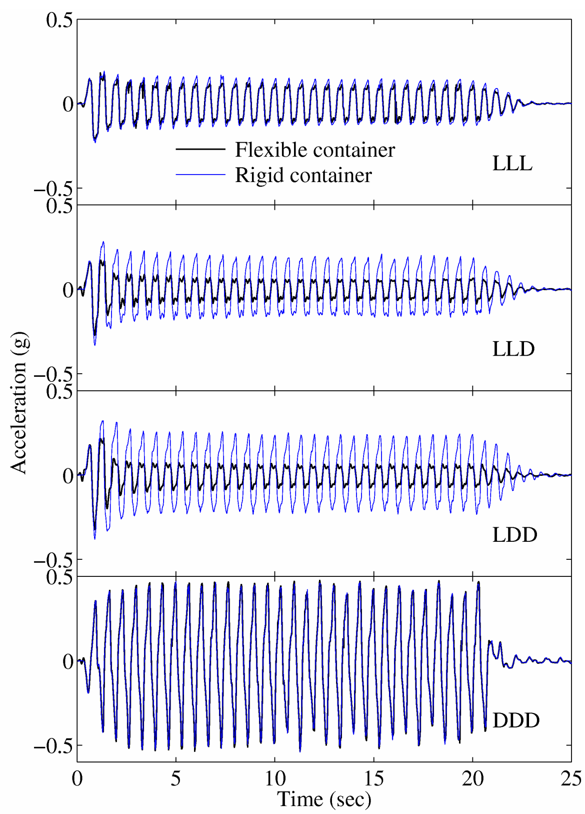

Fig. 6: Computed (rigid-container versus flexible-container models) lateral acceleration histories near dam crest (A7).

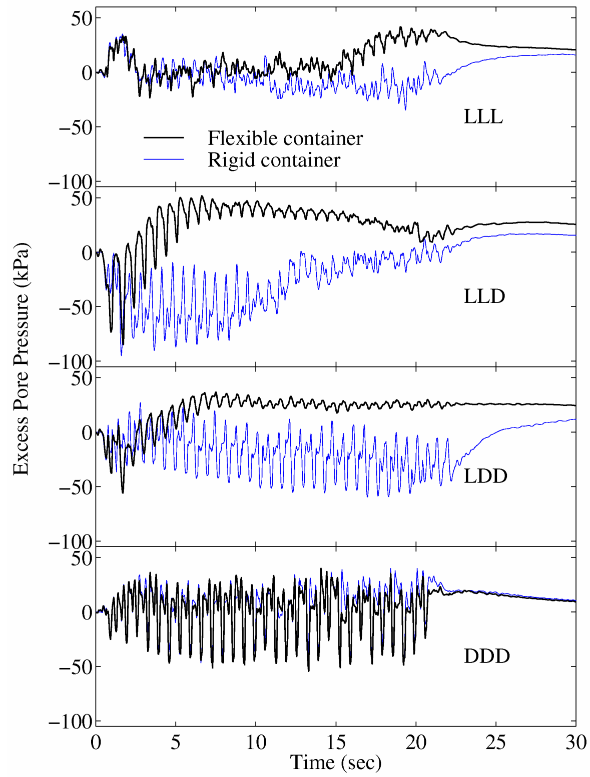

Fig. 7: Computed (rigid-container versus flexible-container models) excess pore pressure histories below upstream dam body (P7).

SUMMARY AND CONCLUSIONS

All results (Figs. 3-7) from the conducted numerical simulations consistently indicated distinctive model response patterns due to the use of a rigid container versus a flexible container. A flexible container only allows seismic energy to be imparted into the model through the base, whereas a rigid container allows energy to be imparted not only from the base, but also from the side boundaries. Therefore, although subjected to the same input motion, the flexible-container models experienced weaker vibration and smaller lateral and vertical displacements. Furthermore, in the flexible-container models, the presence of a loose bottom layer helped greatly in absorbing the upcoming seismic energy. Hence, the LLD and LDD model dams in a flexible container experienced the least displacement and acceleration. On the other hand, the rigid-container models showed reduced displacement and increased crest acceleration with the increase in foundation densification thickness.

Centrifuge models in a flexible container are typically regarded as more representative of field conditions. In this regard, the results using a rigid container are generally on the conservative side, and may be considered as an upper bound on the actual field behavior.

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