Remediation of liquefaction effects for a dam using soil-cement grids: Centrifuge and numerical modeling

La remédiation de liquéfaction effectue pour un barrage utilisant des grilles de ciment de sol: Centrifugeuse et modélisation numérique

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ABSTRACT: Numerical simulations of a centrifuge model test of an embankment dam on a liquefiable foundation layer treated with soil-cement walls are presented. The centrifuge model was tested on a 9-m radius centrifuge and corresponded to a 28 m tall embankment underlain by a 9 m thick saturated loose sand layer. Soil-cement walls were constructed through the loose sand layer over a 30 m long section near the toe of the embankment and covered with a 7.5 m tall berm. The model was shaken with a scaled earthquake having a peak horizontal base acceleration of 0.26 g, followed by a second event with a peak base acceleration of 0.54 g. Both events caused liquefaction in the loose sand layer. Crack detectors indicated that the soil-cement walls sheared through their full length in the second event. The results of the centrifuge model test and two-dimensional nonlinear dynamic simulations are compared. Capabilities and limitations in the two-dimensional simulations of soil-cement grid reinforcement systems, with both liquefaction and soil-cement cracking effects, are discussed. Implications for practice are discussed.

RÉSUMÉ : Les simulations numériques d'un test de modèle de centrifugeuse d'un barrage de quai sur une couche de base(fondation) liquéfiable ont traité avec des murs de ciment de sol sont présenté. Le modèle de centrifugeuse a été testé sur une centrifugeuse de rayon 9-m et correspondu à un quai de 28 m de haut underlain par une couche de sable desserrée(libre) saturée de 9 m d'épaisseur. Les murs de ciment de sol ont été construits par la couche de sable desserrée(libre) sur une section de 30 m de long près de l'orteil du quai et couverts avec une berme de 7.5 m de haut. Le modèle a été secoué avec un tremblement de terre pesé(mesuré) ayant une accélération de base horizontale maximale de 0.26 g, suivi par un deuxième événement avec une accélération de base maximale de 0.54 g. Les deux événements ont causé la liquéfaction dans la couche de sable desserrée (libre). Les détecteurs de première classe ont indiqué que les murs de ciment de sol tondus par leur durée totale dans le deuxième événement. Les résultats du test modèle de centrifugeuse et les simulations dynamiques non-linéaires bidimensionnelles sont comparés. Les capacités et les limitations dans les simulations bidimensionnelles de systèmes de renforcement de grille de ciment de sol, tant avec la liquéfaction que le ciment de sol fêlant des effets, sont discutées. Les implications pour la pratique sont discutées.

KEYWORDS: liquefaction, earthquake, dam, soil-cement, centrifuge, numerical analysis.

1 INTRODUCTION

Soil-cement grid and wall systems have been used to remediate embankment dams against the effects of earthquake-induced liquefaction in their foundations. Soil-cement treatments have the advantage that they can be constructed in a wide range of soils, including silty soils that can be difficult to treat by densification techniques. The seismic performance of soil-cement grids and walls have been studied using three-dimensional (3D) analysis methods (e.g., Fukutake and Ohtsuki 1995, Namikawa et al. 2007), but design practices generally rely on two-dimensional (2D) approximations with equivalent composite strengths for the treatment zones (e.g. Wooten and Foreman 2005, Barron et al. 2006, Kirby et al. 2010, Friesen and Balakrishnan 2012). Some common concerns in the design of soil-cement grids for liquefaction remediation include the potential for cracking and brittle failure, the ability of 2D analysis procedures to approximate the 3D response, and the lack of experimental or case history data to validate 2D or 3D numerical analysis methods.

This paper presents results of centrifuge model tests and numerical simulations of an embankment dam on a liquefiable foundation layer treated with soil-cement walls. The centrifuge model was tested on a 9-m radius centrifuge and corresponded to a 28 m tall embankment underlain by a 9 m thick saturated loose sand layer. Soil-cement grids were constructed through the loose sand layer near the toe of the embankment and covered with a berm. The model was shaken twice with a scaled earthquake motion; the peak horizontal base accelerations were 0.26 g and 0.54 g, respectively. Both events liquefied the loose sand layer. The soil-cement grids developed limited cracking in the first shaking event and sheared through their full length in the second event. Two-dimensional nonlinear dynamic analyses were performed using the finite difference program FLAC (Itasca 2011) and the user-defined constitutive model PM4Sand (Boulanger and Ziotopoulou 2015) for the sands. The treatment zone was represented with area-averaged properties as is common in design practice. The centrifuge model test and numerical simulation procedures are described, followed by comparisons of the measured and simulated responses. Implications of the results for practice are discussed.

2 CENTRIFUGE MODEL TEST

The centrifuge model was tested in a flexible shear beam container at a centrifugal acceleration of 65 g on the UC Davis 9-m radius centrifuge. Standard scaling laws are followed and results are presented in prototype units unless otherwise specified.

The centrifuge model configuration (Figure 1) consisted of a foundation layer of loose Ottawa F-65 sand (relative density of 42%), an embankment and berm of dry, dense, coarse Monterey sand (relative density of 85%), and a set of parallel soil-cement panels over a 30-m long section near the toe of the embankment. The pore fluid was a methylcellulose solution with a viscosity about 20 times that of water. The water table was above the top of the foundation layer and slightly above the tops of the walls. A thin layer of acquarium sand was placed at the water surface elevation to provide a capilary break during model construction.

The soil-cement walls were formed and cured in molds and then arranged in the model container prior to pluviation of the foundation sand layer. The soil-cement had an unconfined compressive strength of 2.0 MPa. The walls were instrumented with crack detectors.

The locations of accelerometers and pore pressure transducers along one section through the model are shown in Figure 2. Linear displacement transducers recorded vertical and lateral displacements of the embankment crest and toe berm.

The model was shaken twice with a scaled version of a recording from Port Island in the 1995 Kobe earthquake. The first shaking event had a peak base acceleration of 0.26 g, followed (after full dissipation of the excess pore pressures) by a second event with a peak base acceleration of 0.54 g.

A photograph of the deformed model after both shaking events is shown in Figure 3. The crest settled about 0.7 m and the toe berm displaced laterally about 1.3 m in the second shaking event, whereas movements in the first shaking event were between a quarter and half of those in the second event.

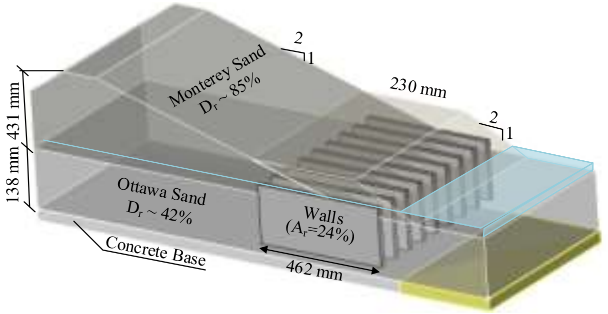

Figure 1. Model configuration with dimensions in model units (mm).

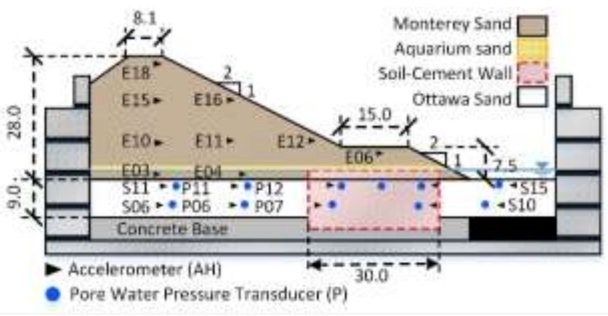

Figure 2. Cross-section of the model showing locations of accelerometers and pore pressure transducers; dimensions are prototype units (m).

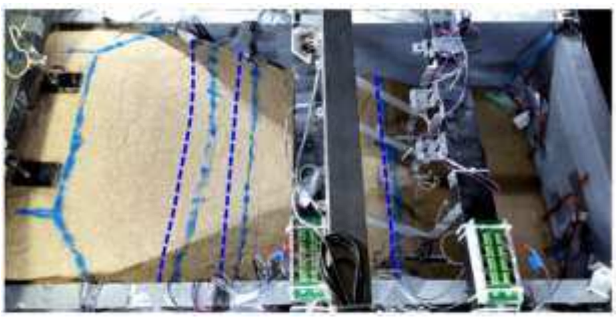

Figure 3. Photograph of the model surface after completion of the second shaking event; dashed blue lines show location of the blue colored sand markers prior to shaking.

Figure 4. Photographs of the soil-cement walls during model excavation after testing; embankment toe was to the left side of both photos.

A photograph of the soil-cement panels during model dissection is shown in Figure 4. Crack detectors indicate that only portions of the panels were cracked during the first shaking event, such that the majority of damage and the offsets along the cracks occurred during the second shaking event.

3 NUMERICAL SIMULATION MODEL

Two-dimensional (2D) nonlinear dynamic analyses were performed using the finite difference program FLAC (Itasca 2011). The mesh and material zones are shown in Figure 5 with the deformed shape after the second shaking event. Analyses used 0.5% Rayleigh damping at a frequency of 1 Hz.

The sands were modeled using the constitutive model PM4Sand, which is a stress-ratio controlled, critical state compatible, bounding surface plasticity model developed for earthquake engineering applications (Boulanger and Ziotopoulou 2015, Ziotopoulou and Boulanger 2016). The parameters for the dense coarse Monterey sand were the same as used by Armstrong and Boulanger (2015). The parameters for the loose sand layer were obtained by calibration against results of cyclic direct simple shear tests on Ottawa F-65 sand by Parra Bastidas et al. (2016). The cyclic stress ratio to cause a peak shear strain of 3% (or an excess pore pressure ratio of 100%) in 15 uniform loading cycles at a vertical effective consolidation stress of 400 kPa was 0.093 for virgin specimens (for the first shaking event) and 0.120 after one liquefaction event (for the second shaking event). The post-liquefaction reconsolidation after the first event causes a small increase in cyclic strength and density (D_r = 45% versus 42%). The calibrated model's stress-strain response to cyclic loading for the second event is illustrated by the single-element simulation in Figure 6.

The soil-cement was modeled using a Mohr Coulomb model with area-weighted cohesion and friction properties. The soil-cement area replacement ratio was 24%, the unconfined compressive strength was 2.0 MPa, and thus the equivalent composite shear strength for the treatment zone in the loose sand layer was taken as 0.25 MPa for the 2D numerical analyses.

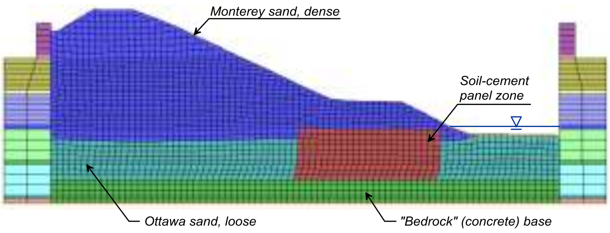

Figure 5. Finite difference mesh and zoning of materials shown with the deformed shape after the second shaking event.

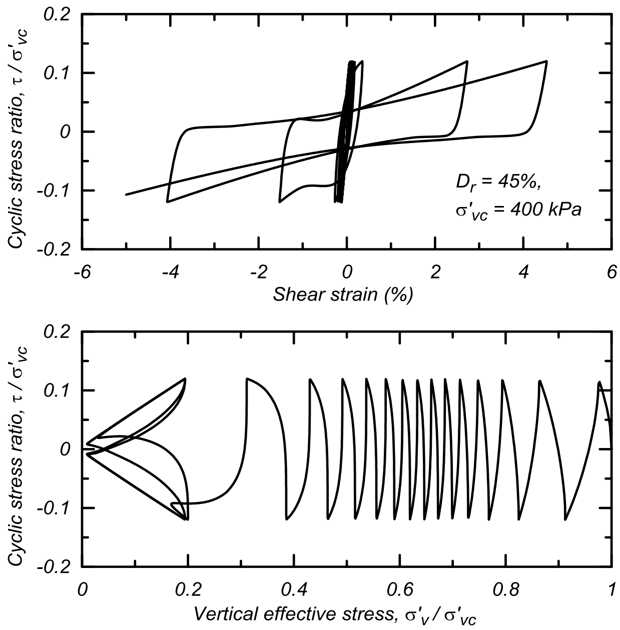

Figure 6. Simulated single-element direct simple shear response for the loose Ottawa F-65 sand subjected to uniform undrained cyclic loading.

The flexible shear beam container was modeled using linear elastic materials. The mass and lateral stiffness of the 2D model, per unit width of soil, was equated with that of the full model container by relative-volume-weighting of the density and stiffness properties of the rings and rubber layers.

4 SIMULATION RESULTS

Numerical simulations were compared to measured responses for both shaking events in terms of the accelerations, pore pressures, displacments, deformation patterns, and soil-cement damage patterns. Results for the second event using one baseline set of input parameters are presented to illustrate these comparisons.

Contours of shear strain on the deformed mesh are shown in Figure 7. The computed deformation patterns and magnitudes are in reasonable agreement with those measured during testing and model dissection. The simulation predicts that the soil-cement panels would be sheared through near their base, which is consistent with the observed damage patterns (Figure 4).

Simulated and measured accelerations are compared in Figure 8 for several points in the embankment, toe berm, foundation layer, and base. The measured accelerations in the loose sand layer exhibit some high frequency spikes that are associated with the cyclic mobility behaviors; the computed responses capture this cyclic mobility response but underestimate the magnitude of the acceleration spikes. The simulations are otherwise in reasonable agreement with the recordings throughout the emankment and foundation.

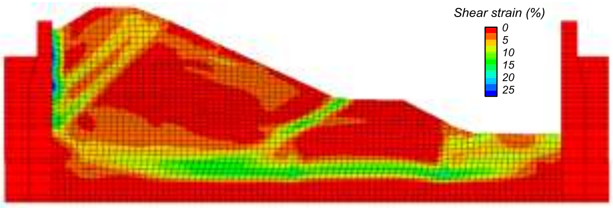

Figure 7. Contours of shear strain on the deformed mesh after the second shaking event.

Simulated and measured pore pressures for several points in the loose sand layer are compared in Figure 9. The measured excess pore pressures are far greater under the embankment (left two columns in Figure 9) than in the free field beyond the toe (right column in Figure 9), reflecting the differences in overburden stresses at these points. The pore pressures rise to values equal to the estimated overburden stresses at these points, indicating that excess pore pressure ratios of, or near, 100% were triggered thoughout the loose sand layer. The simulations reasonably track the rise in excess pore pressures, their peak values, and their dissipation rates after the end of strong shaking.

The 2D analyses are in reasonable agreement with the overall measured response, despite the 2D approximation of the treatment zone. The simulation results for the first shaking event correctly predict that the panels are not expected to shear through along their base, but are expected to during the second shaking event. The modeling of the composite behavior of the treatment zone does not account for crack propagation, strength loss with crack growth and offsets, pore pressure migration into the cracks within the panels, flow of liquefied soil between walls, or the nonlinearity of soil-cement strengths with varying confining stress. The reasonable agreement obtained with the present approximation of composite strengths suggests that these approximations may be sufficient for design purposes.

Numerical simulations were also performed for the case without soil-cement walls. The computed deformations exceeded several meters during strong shaking before the simulation was stopped due to excessive element distortion. These results illustrate that the soil-cement walls were effective in reducing embankment deformations, even though the panels were extensively damaged.

Further analyses of this model will examine the impacts of a more detailed representation of cracking effects as well as sensitivity to the other modeling parameters.

5 CONCLUSIONS

Two-dimensional nonlinear dynamic analyses were presented for a centrifuge model of an embankment dam on a liquefiable foundation layer treated with soil-cement walls. The model corresponded to a 28 m tall embankment underlain by a 9 m thick saturated loose sand layer. Soil-cement walls were constructed through the loose sand layer over a 30 m long section near the toe of the embankment with a replacement ratio of 24%. The model was shaken with scaled earthquake motions having peak horizontal base accelerations of 0.26 g and 0.54 g. The numerical simulations were in reasonable agreement with the recorded dynamic responses, including the triggering of liquefaction in the loose sand layer during both events. The simulations reasonably approximated the observed deformation magnitudes and patterns, and correctly predicted that the soil-cement walls would shear through their full length in the second event. The results of these comparisons provide support for the use of these numerical modeling procedures, including the representation of a treatment zone with area-weighted properties, for analyses of embankment dams with soil-cement treatment of liquefiable soils in their foundations.

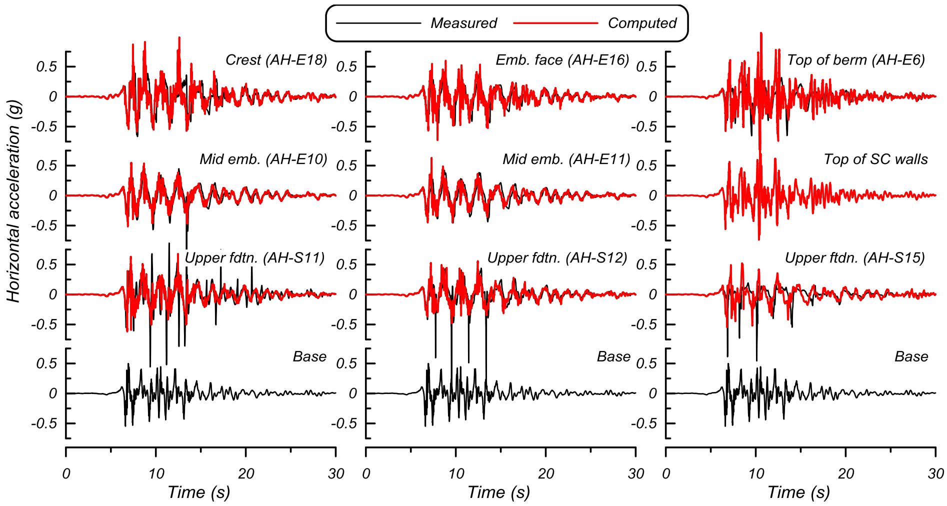

Figure 8. Accelerations along columns at the crest (left column), mid face (middle column), and berm (right column) for second shaking event.

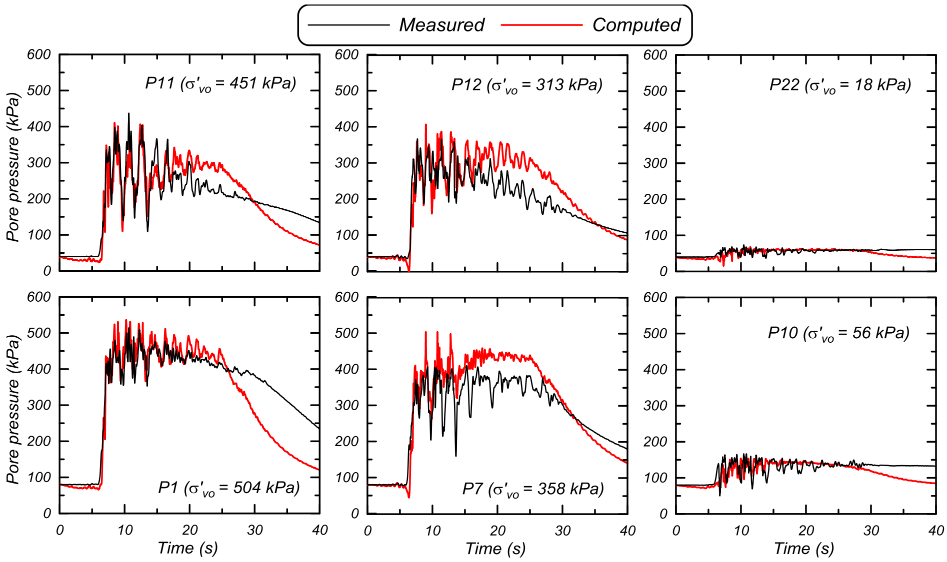

Figure 9. Pore pressures in the upper (top row) and lower (bottom row) part of the loose sand layer beneath the embankment crest (left column), mid embankment face (middle column), and free-field below the embankment toe (right column) for the second shaking event.

6 ACKNOWLEDGEMENTS

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