COMPDYN 2011
III ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

ON ANALYSIS OF LIQUEFACTION-INDUCED DISPLACEMENT IN A CAISSON QUAY WALL

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Keywords: Caisson, Earthquake, Elastoplasticity, Displacement, Finite Element, Liquefaction, Quay Wall.

Abstract. *A series of finite element analyses have been conducted to assess the capability of en open source finite element code in dynamic analysis of a caisson quay wall with liquefiable backfill and foundation soils. A typical quay wall of Rokko island that suffered large displacements and rotations during the 1995 Earthquake in Kobe was considered as a case study. The nonlinear fully-coupled finite element analyses led to the following observations. Earthquake-induced excess pore water pressures have caused significant stiffness degradation in the backfill and foundation soil. This is a likely cause of large lateral displacements and settlements of the caisson wall and the backfill soil. Vertical displacements of the wall and the backfill soil were under-predicted by the finite element simulations. However the numerical simulations produced correct displacement modes, and the order of magnitude of lateral displacement was also correctly simulated.*

1 INTRODUCTION

In the past two decades, there has been considerable progress in development of advanced constitutive and numerical modeling techniques for seismic analysis of geostructures containing liquefiable granular soils. An accurate estimation of earthquake-induced permanent deformations in these structures is an essential component of a performance-based design approach that is currently becoming the prevailing design concept in geotechnical earthquake engineering. In this paper, a caisson quay wall that suffered large displacements and rotations during the 1995 Earthquake in Kobe (Iganaki, et al., 1996) is considered as a case study (Figure 1). The wall is sitting on rubble material underlain by a medium dense sandy soil. The backfill soil is also a sandy soil underlain by an alluvial clay layer. Iganaki et al. (1996) have presented a detailed study of the engineering properties of the backfill and foundation soils.

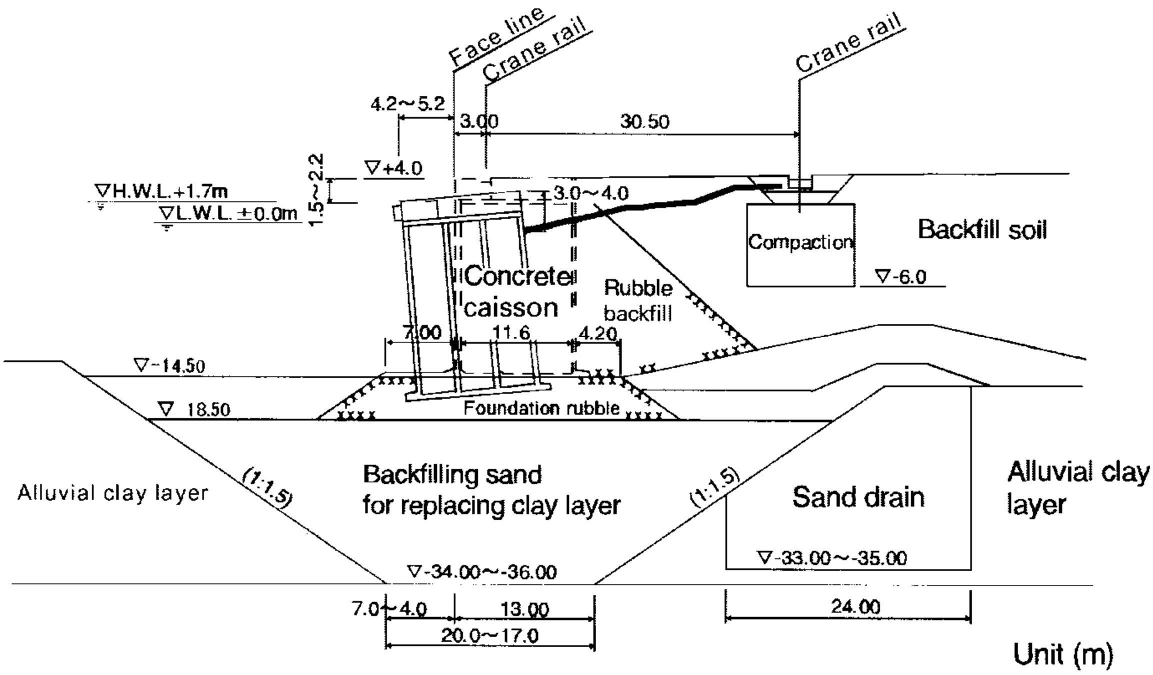

Figure 1. A typical section of the caisson quay-wall in Rokko Island that sustained significant damage and permanent deformations during 1995 Kobe Earthquake (Iganaki, et al., 1996)

Iai et al. (1996) conducted detailed finite element simulations of this quay-wall structure by using a multi-mechanism strain-space plasticity model. The results presented by Iai et al. (1996) show good agreement with the observed response of the wall. In this paper, the results of a series of fully-coupled nonlinear finite element analyses are used to examine the capabilities and limitations of an advanced constitutive and numerical modeling platform (Opensees, http://opensees.berkeley.edu/index.php) in seismic analysis of this quay wall system that contains liquefiable soils as backfill and foundation soils. An outline of the numerical simulations and the results obtained in these analyses is presented in this paper.

2 FINITE ELEMENT MODEL

A detailed finite element model of the wall, its foundation and the surrounding soil was developed, and the material models used for different parts of the model were calibrated based on the experimental data available for the site (Iganaki, et al., 1996). A typical finite element mesh used in the analyses is shown in Figure 2. In order to consider wall-water interaction, the body of water in front of the wall was also included in the finite element model (elements shown in red).

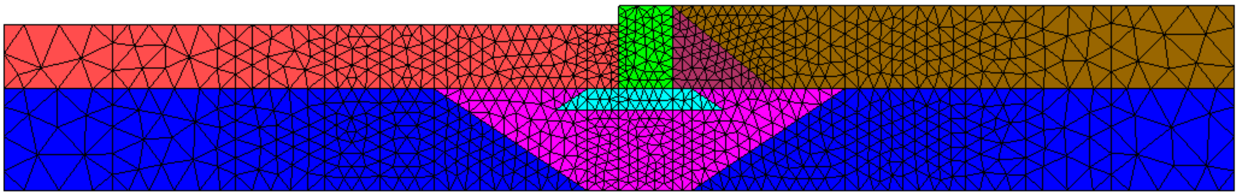

Figure 2. A Typical Finite Element Mesh used in the Analyses

Interfaces of the quay wall with the foundation and backfill soils were also modeled using appropriate interface elements. No-tension zero-length elements were used in the interface between the wall and foundation rubble, the wall and backfill rubble, and the wall and water. No friction was considered between the wall and backfill. Friction between the wall and foundation rubble was modeled using zero-length elements.

3 CONSTITUTIVE MODELS

The multi-surface plasticity models implemented in Opensees platform (Elgamal et al., 2003) were used in the finite element simulations. The pressure dependent multi-surface plasticity model was used for sands and rubbles while the pressure-independent multi-surface plasticity model was used for the alluvial clay. The models were calibrated against the results of the triaxial tests conducted on the soil samples obtained from the site (Iganaki et al., 1996). Calibration efforts showed that it was difficult to match the observed response of the soil at all strain levels and for all shear stress ratios.

Figure 3 shows a typical comparison of the model predictions with the experimental data. In this Figure, the number of cycles to reach a double amplitude axial strain of 5% is plotted against shear stress ratio (i.e. deviatoric stress, τ_d, divided by initial vertical effective stress in the specimen). The calibration efforts showed that it is possible to match the experimental data for any specific level of shear stress ratio. However, when the applied stress ratio is changed to a different level, the predicted number of cycles using the same model constants (parameters) shows some discrepancies with the observed response in the experiments.

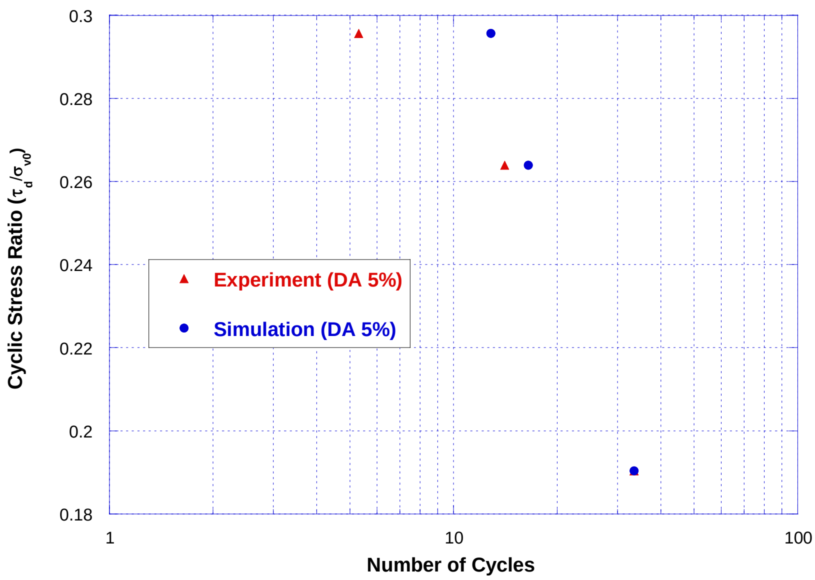

Figure 3. Comparison of the Number of Cycles Predicted by the Calibrated Constitutive Model against the Experimental Data Reported by Iganaki et al., (1996) for the Foundation Sand.

4 ANALYSIS STEPS

The finite element simulations were conducted in two steps. First, initial stresses and internal variables of the constitutive models were initialized by using an incremental gravity turn-on technique. Since the site consisted of both cohesionless and cohesive soils, a quasi-static fully-coupled effective stress analysis was conducted to ensure that both the initial pore water pressure and initial effective stresses were fully accounted for before the system was subjected to seismic loading.

Next, the complete soil-wall-water system was subjected to the horizontal and vertical ground motions recorded near the site during the 1995 earthquake (Iganaki, et al., 1996). Mesh sensitivity analyses were conducted to evaluate the accuracy of the numerical solutions. All the simulations were conducted by using the same parameters that were obtained from calibration of the constitutive models.

5 RESULTS

The numerical simulations showed that the earthquake caused significant pore water pressure generation in the backfill sand (Figure 3) and created a near liquefaction state in the foundation sand (Figure 4).

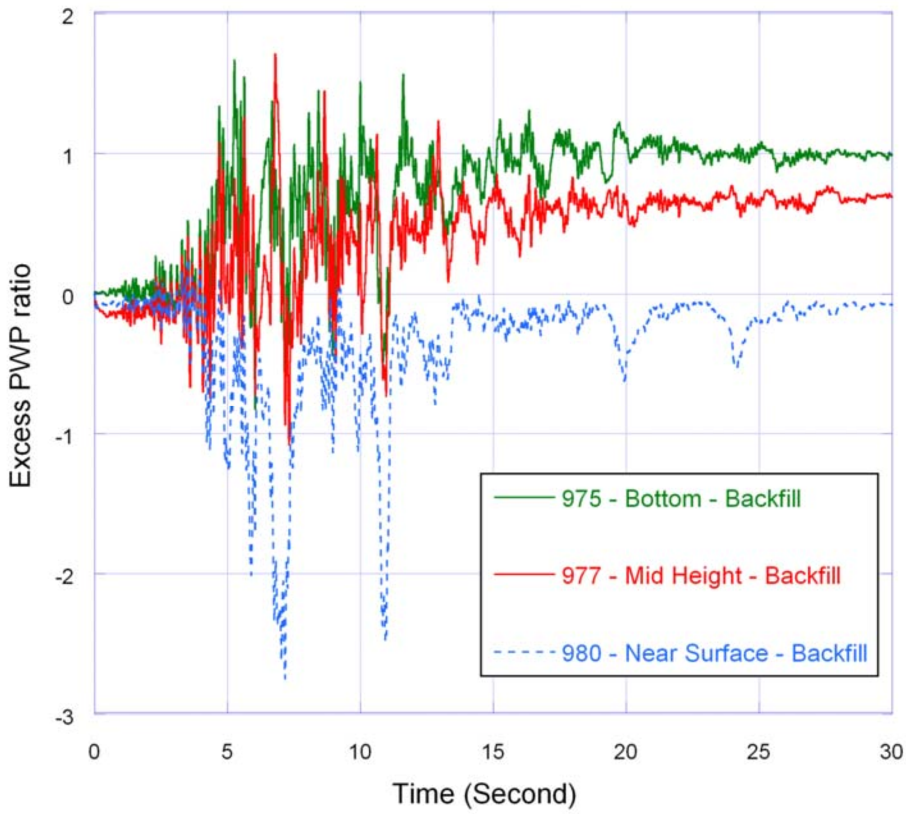

Figure 3. Excess Pore Pressure Ratios Computed at Different Elevations in the Backfill Soil

Figure 3 shows that very large negative excess pore pressure ratio is developed at a soil element located near ground surface in the backfill soil. This is mainly caused by the low confining stress in the soil which makes the soil highly dilative. Much less dilative response is seen in the mid height and bottom of the backfill sand (i.e., near the interface with the alluvial clay) where the higher initial confining stresses make the soil more contractive. It is also observed that the large excess pore pressure at the bottom of the backfill soil brings the soil to a near liquefaction state. The excess pore pressure ratio remains at its high level even when the earthquake ceases.

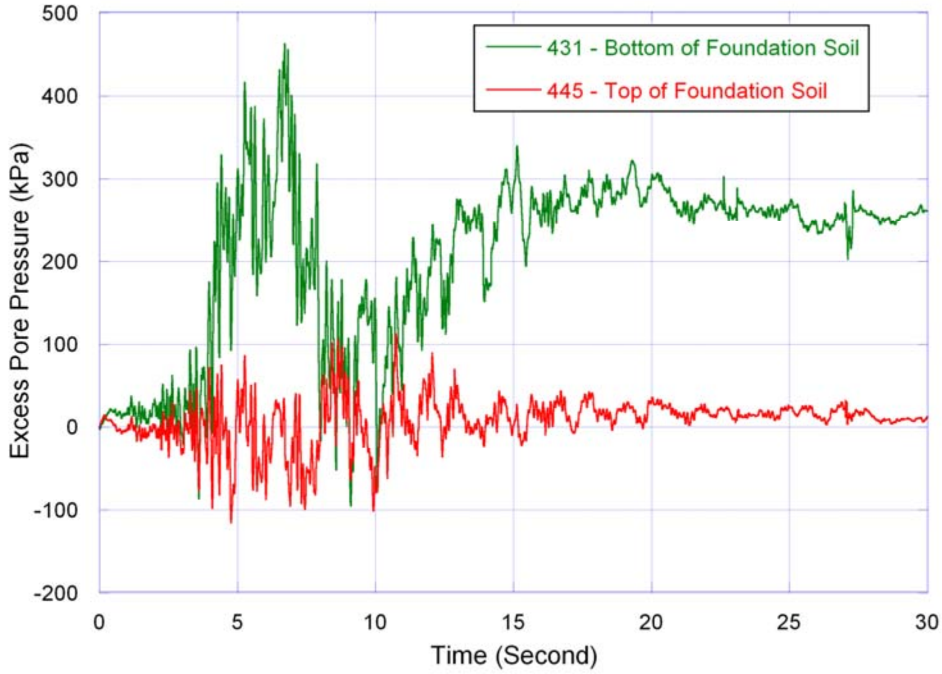

Figure 4. Excess Pore Pressure Computed at Different Elevations in the Foundation Soil

Figure 4 shows that large positive excess pore pressure is generated at an element near the bottom of the foundation sand. This excess pore pressure significantly degrades the stiffness of the foundation soil and contributes to the rotation of the quay wall. A soil element near the top of the foundation soil (just below the foundation rubble), however, shows highly dilative response. This is likely due to the large shear stresses imposed by the movement of the wall on a soil element that is under a modest confining stress.

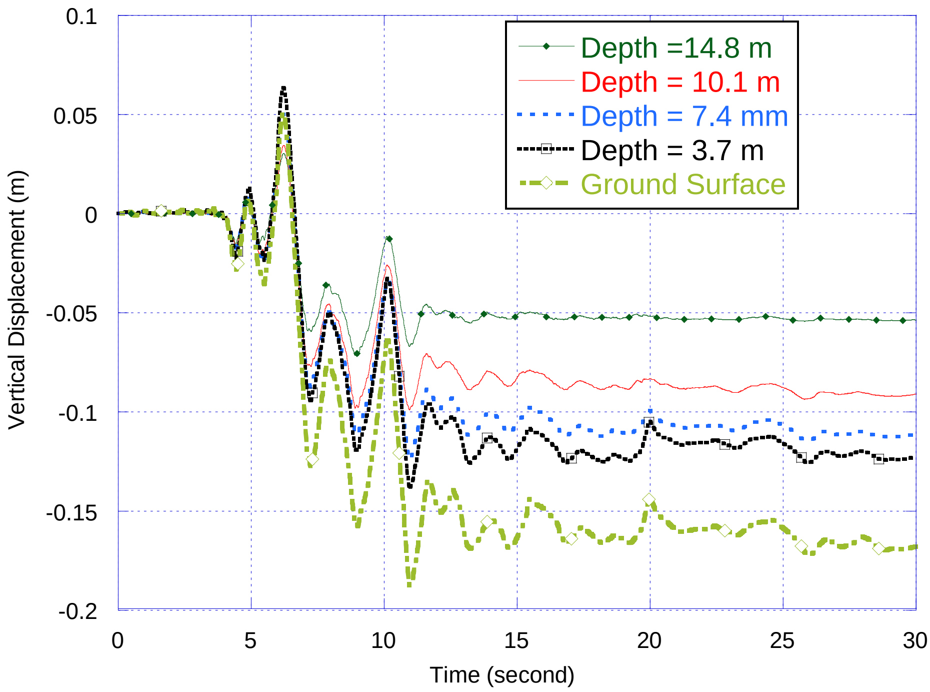

Figure 5. Vertical displacements in the backfill soil far away from the wall

Figure 5 shows the computed vertical displacements in a column of the backfill soil far away from the wall. The backfill soil has experienced significant settlements due to the large excess pore pressure generation caused by the earthquake.

The analysis results also indicate that despite the significant pore pressure generation in the foundation soil, the soil has amplified the base motion and the wall has experienced large horizontal accelerations at its crest. Figure 6 shows a comparison of the acceleration time histories computed at the ground surface and at the wall crest with the assumed base acceleration.

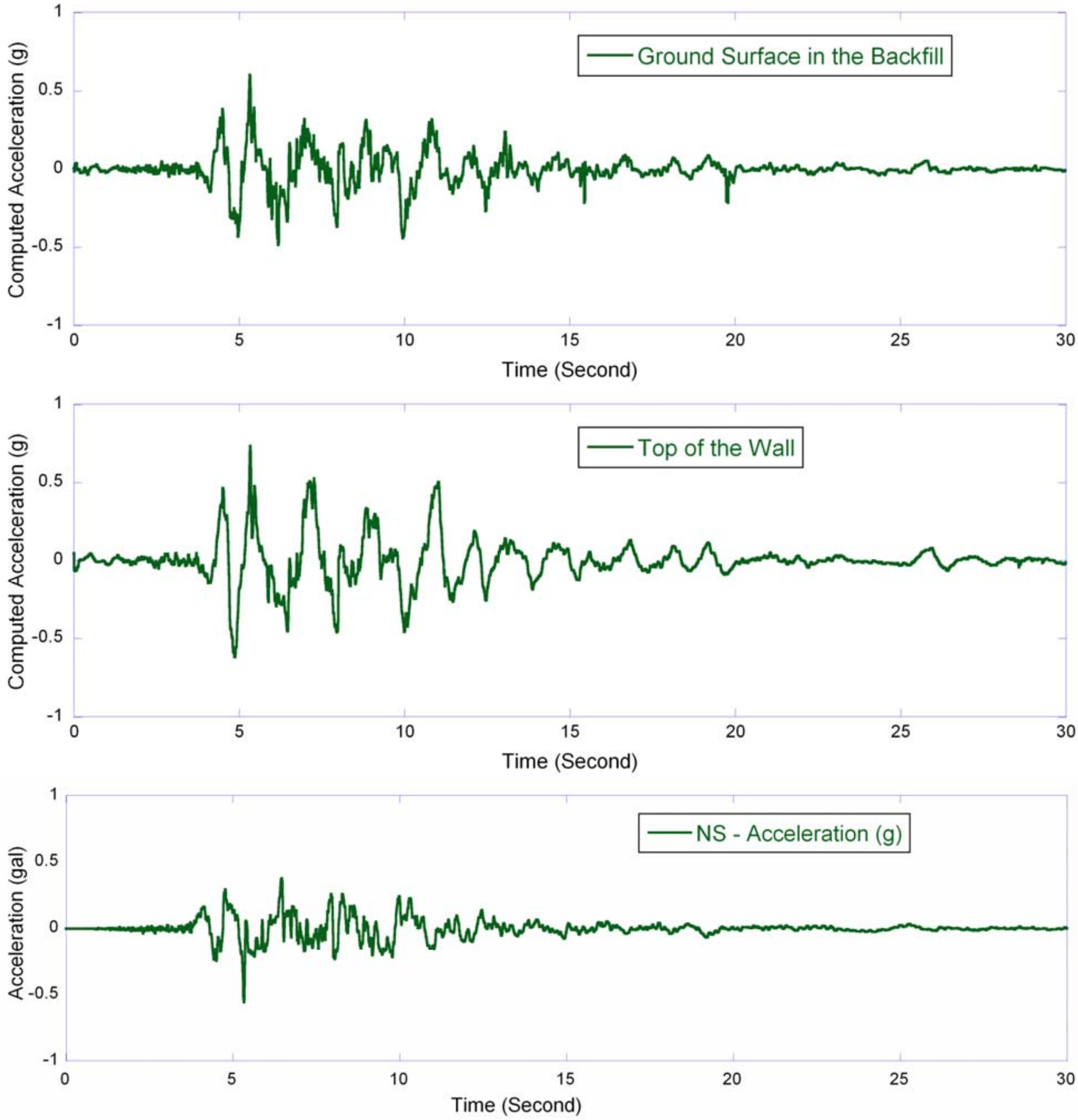

Figure 6. Comparison of the Acceleration Time Histories Computed at the Ground Surface in the Backfill (top) and at the Wall Crest (middle) with the Base Acceleration Time History (bottom).

The finite element simulations show a lateral displacement of about 2 meters for the wall and a settlement of 1 meter in the backfill soil. The wall tilted about 2.5^{o} counter-clockwise.

The simulated displacements and deformation patterns are in general agreement with the observed response of the wall in the field. However both lateral and vertical displacements are under-predicted. Figure 7 shows the earthquake-induced displacements of the quay wall and its surrounding soils at the end of the earthquake loading.

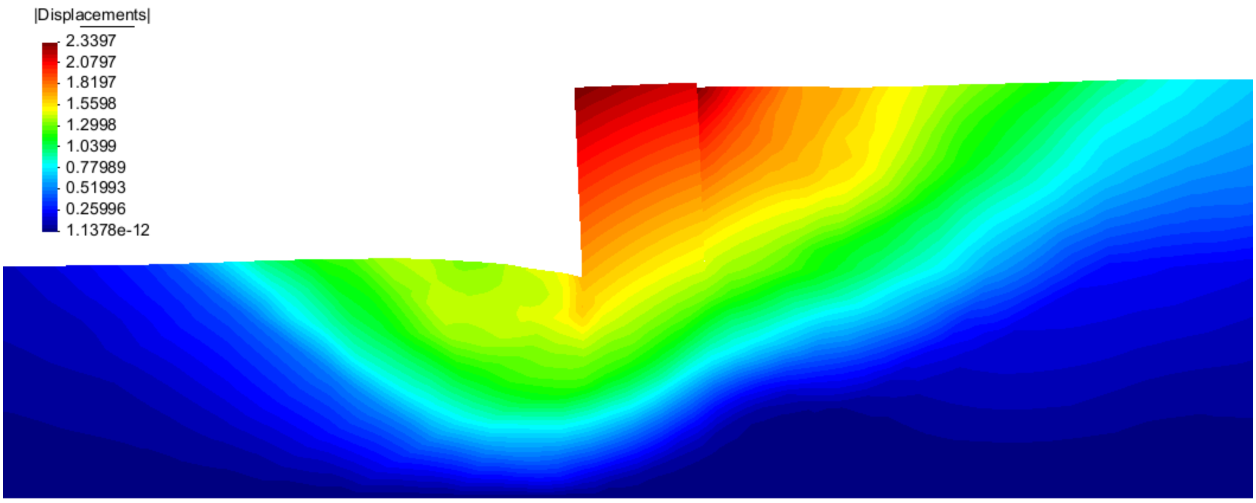

Figure 7. Earthquake-induced Permanent Displacements in the Quay Wall and Surrounding Soils

6 CONCLUDING REMARKS

Based on the finite element simulations presented in this paper, the following observations are made:

- Relatively large excess pore water pressures were developed in the backfill and foundation soils. These excess pore pressures have likely led to significant stiffness degradation in these soils. This is a likely cause of large lateral and vertical displacements and in the caisson quay wall and the backfill soil.
- Vertical displacements of the wall and backfill were under-predicted by the finite element simulations. However, the numerical simulations produced correct displacement patterns, and the order of magnitude of lateral displacements was also correctly simulated.

A key factor contributing to the large rotation of the wall appears to be the local failure of the foundation soil. Given the inherent deficiency of classical continuum formulation in capturing the post-failure response of granular soils, it is difficult to model the local failure in the foundation soil by using the constitutive models that are based on classical continuum mechanics. An enhanced constitutive model with an internal length scale (e.g., Manzari, 2004) is necessary for proper modeling of the settlements and rotations caused by the local failure in the foundation soil.

7 ACKNOWLEDGMENT

The authors are grateful for the support provided by the National Science Foundation via Geomechanics and Geohazard programs directed by Dr. Richard Fragaszy. The first and fourth authors wish to acknowledge the REU supplemental funds for the participation of the fourth author during her undergraduate studies at the George Washington University. Professor Susumu Iai, Dr. Atsushi Nozu, Port and Airport Research Institute, and the Kobe City Government graciously provided the ground motion records used in the simulations.

REFERENCES

[1] Inagaki, H., S. Iai, T. Sugano, H. Yamazaki, and Inatomi, T. (1996). "Performance of caisson type quay walls at Kobe Port," *Soils and Foundations,* Special Issue on Geotechnical Aspects of the January 17 1995 Hyogoken Nanbu Earthquake, Japanese Geotechnical Society, pp.119-136.

[2] Iai S., Sugano T., Ichii K., Morita T., Inagaki H., and Inatomi T. (1996). "Performance of Caisson Type Quay Walls." The 1995 Hyogo-ken Nanbu Earthquake-Investigation into Damage to Civil Engineering Structures. Japan Society of Civil Engineers, 181-207.

[3] Elgamal, A., Yang, Z., Parra, E., and Ragheb, A., (2003). "Modeling of Cyclic Mobility in Saturated Cohesionless Soils," *International Journal of Plasticity*, Pergamon, Elsevier Science Ltd., 19(6): 883-905.

[4] Manzari, M.T. (2004). "Application of Micropolar Plasticity to Post-failure Analysis in Geomechanics Problems." International Journal of Numerical and Analytical Methods in Geomechanics, 28(10), 1011-1032.