Seismic Performance of a Layered Liquefiable Site: Validation of Numerical Simulations Using Centrifuge Modeling

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Abstract

Effective mitigation of liquefaction requires a reliable evaluation of liquefaction triggering and its consequences in terms of excess pore pressures, accelerations, and displacements. However, reliable prediction of all these key response parameters remains challenging in the past. In this paper, the results of a centrifuge experiment modeling of a layered soil profile, including a liquefiable layer of Ottawa sand, are used to evaluate the predictive capabilities of two state-of-the-art constitutive models. The models were first calibrated using the same set of monotonic and cyclic triaxial tests and were then used to simulate the seismic performance of a layered soil deposit to a horizontal earthquake motion. This paper presents the systematic calibration process adopted for each constitutive model, followed by a comparison of the numerical results with centrifuge recordings. This effort aims to provide insight into the strengths and limitations of the adopted models.

INTRODUCTION

Earthquake-induced liquefaction continues to cause significant damage to the built environment. Although great progress has been made in understanding and predicting the phenomenon of liquefaction and its effects via advanced soil constitutive models, the simultaneous prediction of accelerations, generation and dissipation of excess pore pressures, and post-liquefaction settlements due to reconsolidation remains a challenge, even under level-ground free-field conditions and 1D horizontal shaking, and without the complexities of soil-structure-interaction.

Reliable prediction of the triggering and consequences of liquefaction requires performing a nonlinear site response analysis, because soil properties under large strains as well as excess pore pressures leading to liquefaction cannot be simulated using a linear or equivalent-linear method. A number of soil constitutive models have been developed in the past to simulate the seismic response of saturated sand under earthquake loading (Elgamal et al. 1998; Papadimitriou et al. 2001; Dafalias and Manzari 2004; Boulanger and Ziotopoulou 2015; Gao and Zhao 2015). The predictive capabilities of these models have been evaluated individually either based on element level tests or centrifuge results typically with a uniform sand layer. For example, Manzari and Arulanandan (1993), Parra (1996), Elgamal et al. (2002), Taiebat et al. (2007) and Tasiopoulou et al. (2015) assessed the predictive capabilities of various soil constitutive models and numerical approaches for modeling of a centrifuge experiment with a homogenous layer of saturated sand. A systematic comparison of the capabilities of different soil constitutive models in capturing distinct features of site performance (i.e., in terms of accelerations, excess pore pressures, and settlements) in a layered, level, liquefiable deposit can provide valuable insight to guide research and practice. Here, three-dimensional (3D), nonlinear, soil-fluid, fully-coupled, effective stress, dynamic finite element analyses were performed using the OpenSees platform (Mazzoni et al. 2007) to assess the capabilities of two soil constitutive models to predict seismic site performance: 1) the PDMY02 soil model developed by Elgamal et al. (2002) and Yang et al. (2008); 2) a modified version of Manzari-Dafalias constitutive model (Rahimi-Abkenar and Manzari 2016).

This paper describes the calibration for the aforementioned two constitutive models using the same set of monotonic and cyclic triaxial tests on Ottawa sand F65. The calibrated soil models are subsequently employed to predict the response of a layered liquefiable soil profile undergoing horizontal earthquake shaking in centrifuge. Although the experiment was conducted before the simulations, the predictions were essentially blind, as the modelers did not have access to the experimental results until after their simulations. The numerical results are then compared with experimental measurements in terms of accelerations, excess pore pressures, and settlements during the first major motion to better evaluate and compare predictive capabilities and limitations of the two models.

EXPERIMENTAL PROGRAM

Triaxial Tests. Drained and undrained monotonic and cyclic triaxial compression tests were conducted on Ottawa sand specimens (70mm in diameter and 140mm in height). The tests were performed in accordance with Head (1986), ASTM D 5311, and ASTM D 4767. All soil specimens were reconstituted by the Air Pluviation technique (AP) at three relative densities ($D_r \cong 40$, 60 and 90%).

Samples were saturated first with carbonic dioxide for approximately 10 min. De-aired water was then flushed into the specimens from the bottom drain lines. De-aired water was allowed to flow through the specimen until an amount equal to ten times the void volume of the specimen was collected in a beaker through the specimen's upper drain line. For all samples tested, the average B-values of the specimens, after back pressure saturation, was higher than 0.96.

For drained and undrained triaxial compression tests, all specimens were isotropically consolidated to various levels of effective stress ranging from 50 to 300 kPa. For the undrained cyclic triaxial tests, specimens were reconsolidated isotropically under an effective confining stress (σ'_c) of 100 kPa. Cyclic axial loading, under the strain-controlled mode, was applied to the sample through uniform sinusoidal cycles of 1 Hz frequency.

Centrifuge Experiments. A series of centrifuge experiments were performed at the University of Colorado Boulder (CU Boulder) to evaluate site performance and soil-structure interaction on liquefiable ground. Only the centrifuge experiment that approximately modeled free-field conditions was selected for this study, due to its simplicity. Figure 1 shows the configuration of the experiment and the layout of the instrumentation. It is acknowledged that centrifuge experiments in general are simplified and do not represent the complexities of soil and ground motion characteristics found in the field. However, considering their limitations, they can still provide valuable insight and data for the validation of numerical models.

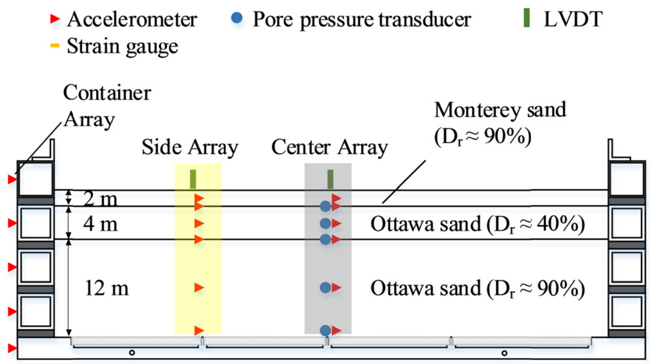

Figure 1. Schematic of centrifuge experiment simulating free-field site response (dimensions are presented in prototype scale at 70g of centrifugal acceleration)

The properties of Ottawa sand were measured at CU Boulder as: maximum void ratio (e_{max}) = 0.81, minimum void ratio (e_{min}) = 0.53, uniformity coefficient (c_u) = 1.56, and specific gravity of solids (G_s) = 2.65. The dense bottom layer of Ottawa sand with a thickness (H) of 12m in prototype scale was dry pluviated to achieve a relative density (D_r) of approximately 90%. This dense layer was overlaid by a looser liquefiable layer of Ottawa sand with $D_r \approx 40\%$ and H = 4m. Finally, a layer of Monterey sand (e_{max} = 0.84 and e_{min} = 0.54) with $D_r \approx 90\%$ and H = 2m was dry pluviated as the non-liquefiable surface layer. The pluviation technique in the centrifuge mimicked the technique used in preparing the triaxial specimens. Table 1 provides the initial measured conditions of the materials used for the experiments and the numerical analysis.

The soil specimen was prepared in a flexible-shear-beam container to reduce the effects of reflecting waves observed in containers with rigid walls. A solution of methylcellulose in water (Stewart et al. 1998) was used to saturate the soil under vacuum with a viscosity 70 times that of water to satisfy the scaling laws. The model was subsequently spun to 70g of centrifugal acceleration, after which a number of 1D horizontal earthquake motions were applied to its base in flight. The results and simulations presented in this paper are all in prototype scale units, unless stated otherwise. Table 1 illustrates the initial measured conditions of the materials used for the experiments and the numerical analyses.

The instrumentation was planned to measure the accelerations and pore water pressures at the boundaries between layers and also at their mid-depth (Figure 1). In addition, linear variable displacement transducers (LVDTs) were placed on the surface of the Monterey sand,

and the top and bottom of the liquefiable layer, to monitor the vertical displacement in the liquefiable layer. Table 1 illustrates the initial measured conditions of the materials used for the experiments and the numerical analysis.

Table 1. Initial soil properties in the centrifuge experiment

Thickness / %D_r / Layer	Void Ratio	Saturated unit weight (kN/m^3)	Hydraulic conductivity (m/s)
2m / 90% / Monterey Sand 030	0.570	19.81	5.30e-04
4m / 40% / Loose Ottawa Sand F65	0.698	19.05	1.41e-04
12m / 90% / Dense Ottawa Sand F65	0.557	19.89	1.19e-04

This paper focuses on the response of soil to one of the motions, the horizontal component of the 1995 Kobe earthquake registered at the Takatori station, which was scaled to a peak ground acceleration, PGA, of 0.3 g and applied to the base of the container in flight. This motion is referred here as “Kobe-L”. Figure 2 shows the acceleration time history, Arias Intensity time history, and the acceleration response spectrum (5%-damped) of the Kobe-L motion.

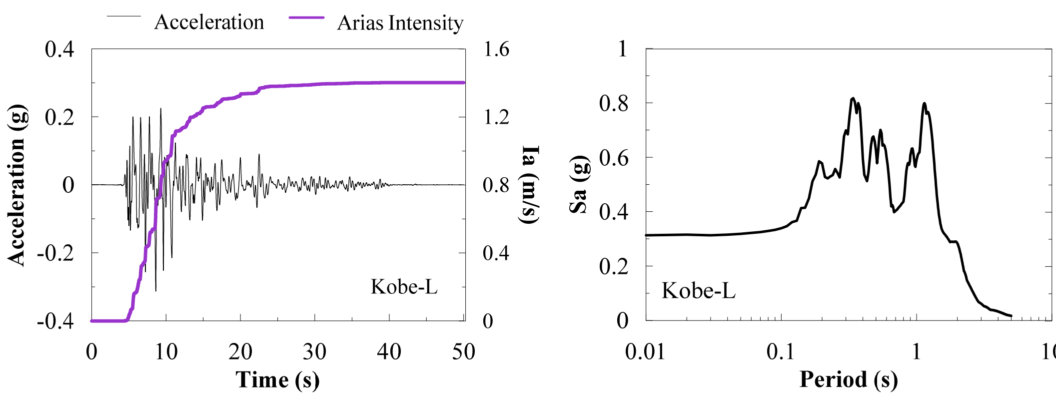

Figure 2. The acceleration and Arias Intensity (Ia) time histories and response spectrum (5%-damped) of the Kobe-L earthquake motion recorded at the base of the container in centrifuge and used as input to the single soil column numerical analyses.

NUMERICAL SIMULATIONS

Fully-coupled, effective stress, finite element analyses were conducted to simulate the response of a single soil column in 3D using two different constitutive models (PDMY02 and Modified M&D) implemented in OpenSees. Both constitutive models were developed to simulate the stress-strain behavior of cohesionless soils under static and dynamic loading for drained, partially drained, and undrained conditions. In all simulations, solid-fluid 8-node hexahedral BrickUP element were used. This element has 4 degrees of freedom at each node, 3 for displacement in different directions, and 1 for fluid pressure.

In the PDMY02 model, the yield criteria are defined by open conical-shaped yield surfaces that have a common apex at the origin of the principal stress space. This model was developed to capture the cyclic response of pressure-dependent soils such as dilatancy and non-flow liquefaction. The model follows a non-associated flow rule to simulate volumetric dilation and contraction under shear. No plastic change of volume is predicted by this model under a constant stress ratio. Further, the model requires parameter calibration separately for different soil relative densities. Using the model requires defining a set of parameters that characterize the soil's elastic and plastic behavior, the evolution of excess pore pressures with time, and the coupling between shear and volumetric strains.

In the modified M&D model (Rahimi Abkenar and Manzari 2016), the constitutive model proposed by Dafalias and Manzari (2004) has been modified to capture the prediction of flow liquefaction and cyclic mobility. The model is cast within a two-surface plasticity platform and uses state parameter to link the stress-strain-strength properties of the soil to its evolving void ratio and stress conditions as it approaches the critical state. The modified model proposes revised expressions for shear modulus and hardening moduli.

Model Calibration. The soil parameters in the PDMY02 and Modified M&D models were first calibrated to capture the response of Ottawa sand as observed during both drained and undrained monotonic and undrained cyclic triaxial tests. The shear strain versus number of cycles required to cause liquefaction (here defined as $R_u = 0.99$) was also compared between the numerical simulations and cyclic triaxial tests during model calibration.

The numerical simulations at the element level were carried out in OpenSees by using a single brickUP element with the initial conditions (mean effective stress and density) corresponding to the isotropically consolidated samples. The elements were then subjected to undrained cyclic shearing. The PDMY02 model parameters required separate calibration for different soil relative densities, whereas the Modified M&D model resulted in a single set of parameters for different relative densities. In the PDMY02 model, the maximum shear modulus was computed based on the equation proposed by Hardin and Black (1968) as a function of void ratio at 1 atm of mean effective stress. Other key parameters in the PDMY02 model, such as the friction angle and phase transformation angle were obtained from the set of monotonic drained and undrained triaxial tests performed at CU Boulder. These define the outer yield surface and the boundary between the contraction and dilative behavior, respectively. The yield surfaces in the PDMY02 model were defined manually as discrete points for pairs of shear modulus ratio (G/G_{max}) versus shear strain, in order to describe better the shear stress-strain response (as suggested by Hashash et al. 2015). Contractive and dilative parameters were set to reasonably match the simulated results with triaxial tests in terms of stress-strain response and number of cycles to reach liquefaction, as shown in Figures 3 and 4 for Ottawa sand.

The Modified M&D model was calibrated to capture the cyclic triaxial tests and to ensure that number of cycles to cause liquefaction was reasonably reproduced by the model. As shown in Figures 3 and 4, the PDMY02 model seemed to generally provide excessive damping and a faster generation of excess pore pressures in undrained cyclic triaxial tests compared to the experimental results, and the opposite behavior was observed in the Modified M&D model. However, both models reasonably captured the shear strain versus number of cycles to liquefaction as observed in the laboratory, as shown in Figure 4.

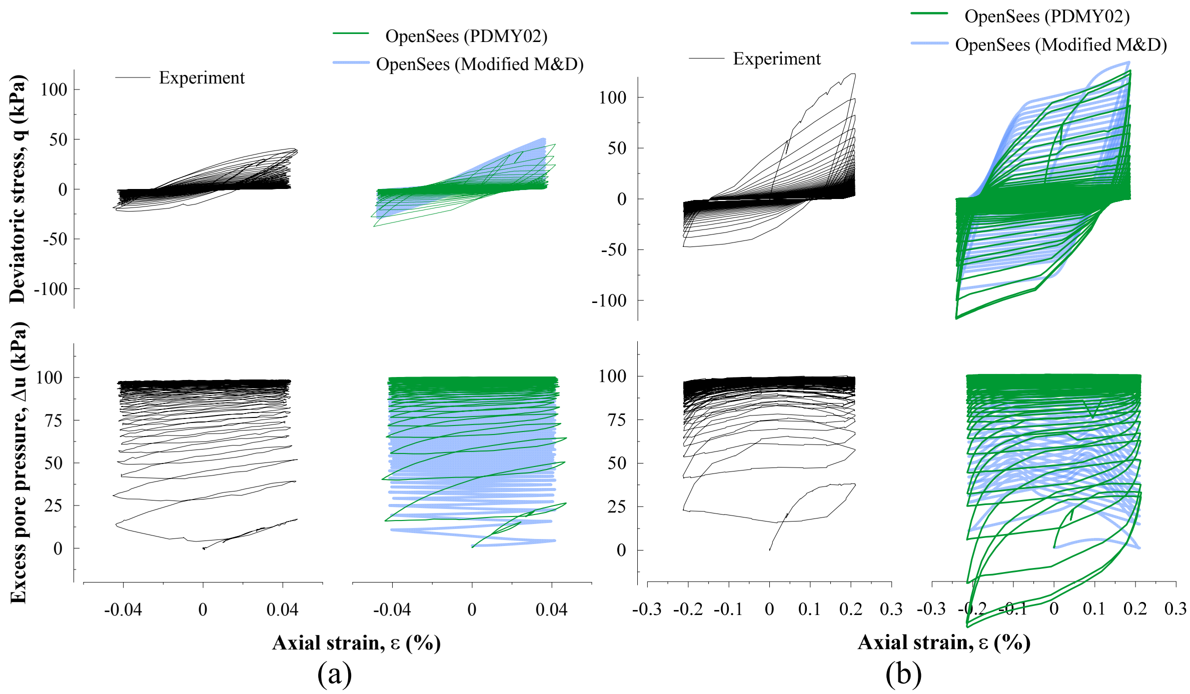

Figure 3. Comparison between experimental and numerical simulated response for Ottawa sand F65: (a) D_r = 40% and a cyclic shear strain amplitude of 0.063%; and (b) D_r = 90% and a cyclic shear strain amplitude of 0.32%.

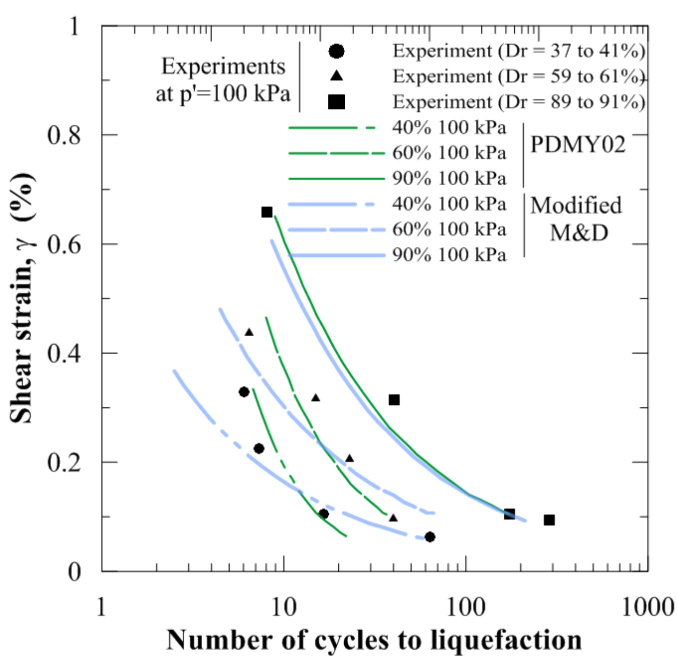

Figure 4. Relation between number of cycles required to achieve liquefaction (R_u≈0.99) and cyclic shear strain amplitude, γ, in numerical simulations and cyclic triaxial experiments on Ottawa sand F65.

Comparison of Numerical Predictions with Centrifuge Measurements. A 3D single soil column was modeled in OpenSees that represented the soil profile in centrifuge in the prototype scale. The column was defined by a total of 23 BrickUP elements, the size of which was determined based on the shear wave velocity of the soil column and the frequency range of interest (Seed and Idriss, 1970; Bardet 1993; Menq 2003). Nodes located at the same depth were tied to move together in all directions (equal degrees of freedom). This contraint serves to roughly simulate the conditions in the centrifuge. The container's aluminum base was simulated as a rigid base, and the acceleration time history of Kobe-L motion recorded at the base was applied to the base nodes. Only the top nodes were set as pervious to allow upward water flow. In the simulations with the PDMY02 model, a constant hydraulic conductivity (k) was assumed during and after the end of the base motion, whereas in the simulation with the Modified M&D model, k is automatically updated and increased at each time step in each element as a function of the excess pore pressure ratio. Using the PDMY02, there is a faster dissipation of the pore pressure because the coefficient of consolidation, cv, is overestimated due to an underestimation of coefficient of volumetric compressibility, m_v (as it shown by Howell et al. 2015).

Figure 5 shows the experimentally measured and numerically computed acceleration time histories, response spectra (5% damped), and Arias Intensity time histories at different depths along the soil profile. The Modified M&D model had a lower material damping compared to the PDMY02 model and triaxial results, leading to larger deviatoric stresses. The lower damping in this model led to an over-prediction of acceleration amplitudes, particularly within the looser layer of Ottawa sand and at lower periods (or higher frequencies). The PDMY02 model was able to capture the first three cycles of motion very well at all depths. However, as large excess pore pressures generated followed by significant volumetric settlement and densification, the predicted results showed less favorable comparison with the centrifuge results.

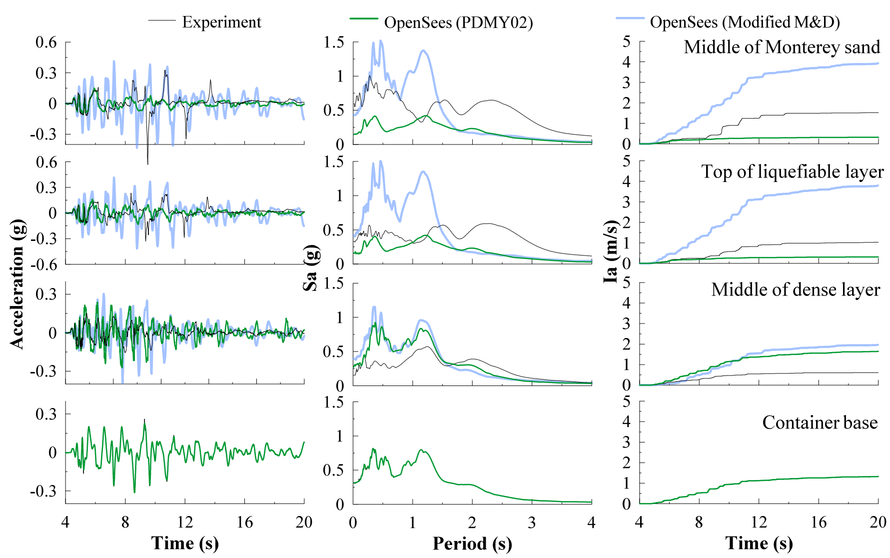

Figure 5. Experimentally measured and numerically computed acceleration time histories, response spectra (5%-damped), and Arias Intensity time histories at different depths during the Kobe-L motion.

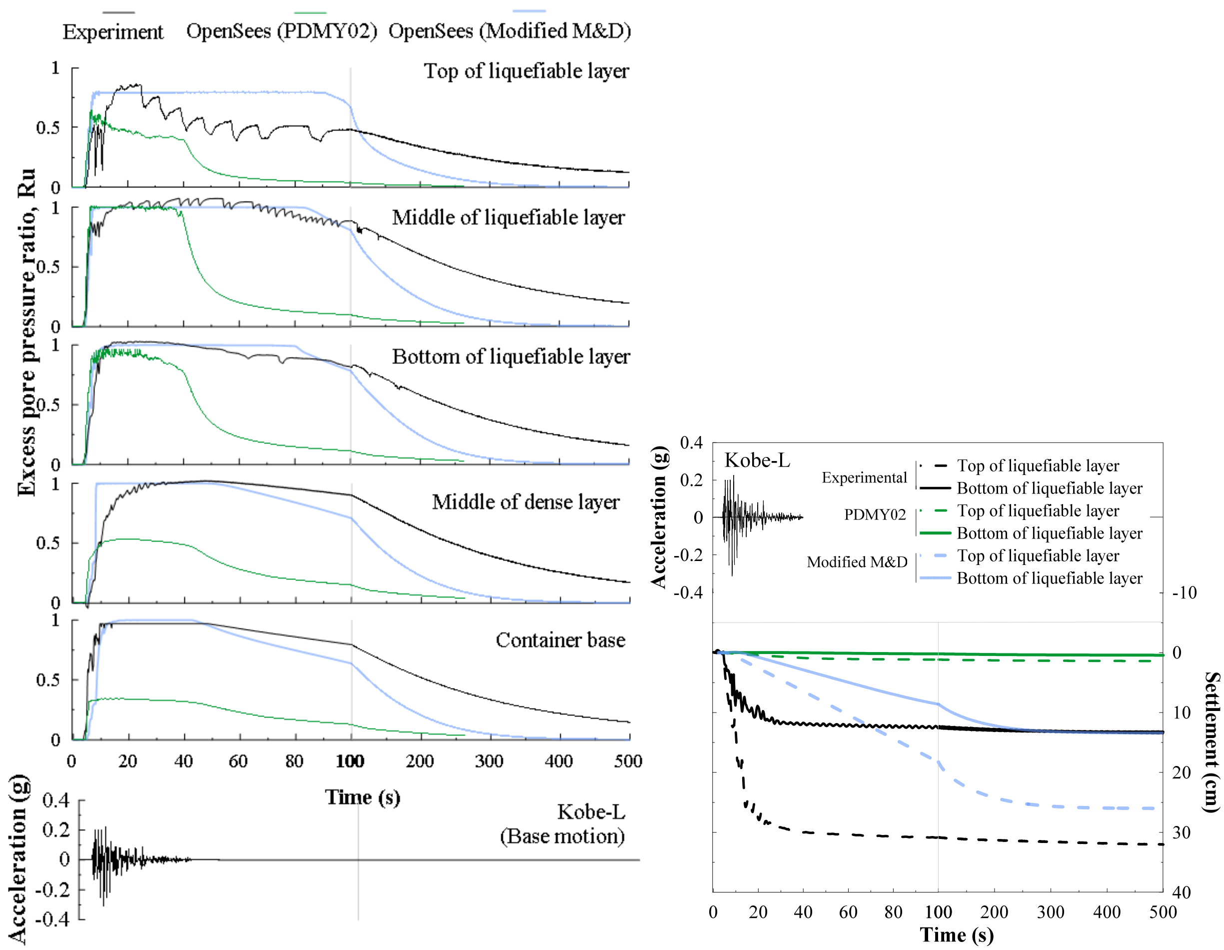

Figure 6. Experimentally measured and numerically computed excess pore pressure ratio (R_u) and settlement time histories at different depths during the Kobe-L motion.

Figure 6 compares the experimentally measured and numerically computed excess pore pressure ratios (R_u) and settlements during the Kobe-L motion. Both models captured the magnitude and timing of peak R_u well within the liquefiable layer, but the PDMY02 model overestimated material damping and hence, underestimated the extent of excess pore pressure generation, particularly in the lower dense layer of Ottawa sand. In general, the Modified M&D model showed reasonably good agreement with the recorded excess pore pressures both during and after earthquake loading at all depths, in particular in the lower dense layer, which also appears to have led to an improved prediction of pore water pressure dissipation in the upper layers. The Modified M&D showed a good prediction of the final permanent volumetric strains or settlements, as shown in Figure 6. However, the rate of settlement predicted by the Modified M&D model was still slower than that observed experimentally, even with the variable k. The PDMY02 model significantly underestimated settlements. Similar observations were reported by Karimi and Dashti (2015, 2016).

SUMMARY AND CONCLUSIONS

The capabilities of two different constitutive models (PDMY02 and Modified M&D) are compared in this paper in capturing the acceleration, excess pore pressure, and settlement response of a layered soil profile, including a liquefiable layer. Although both constitutive models were calibrated to simulate the number of cycles required to cause liquefaction in triaxial tests, when comparing the time history of results both in triaxial and centrifuge experiments the PDMY02 and Modified M&D models tended to overestimate or underestimate material damping, respectively. As a result, the amplitudes of accelerations were often underestimated by PDMY02 and overestimated by Modified M&D. The Modified M&D model generally provided a better prediction of excess pore pressure generation and volumetric settlements compared to the PDMY02 model. Further Class-C1 simulations will investigate ways to improve the responses produced by both models.

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