Dynamic analysis of single pile in liquefied sands modeled as fluid

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ABSTRACT

Case histories have shown that liquefaction-induced lateral spreading is one of the main causes of damage to pile foundations subjected to seismic loading. This study will investigate the pile-liquefied soil interaction based on fluid mechanics method in which the liquefied soils will be modeled as Newton fluids. A numerical simulation on a single pile embedded in a fully saturated sand stratum was conducted and then the numerical results was compared with shake table test results. The results show that the numerical simulation can well capture the general features of soil response, pile displacement/moment and soil pressure imposed by liquefied soil on the pile after liquefaction occurs. The lateral soil pressure distribution in liquefied sand increases with depth in a nonlinear manner.

Keywords: single pile; liquefied sands; newton fluid; finite element simulation, seismic loading

1 INTRODUCTION

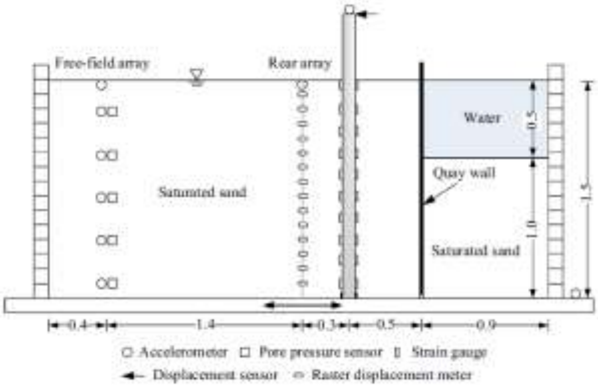
In recent years, liquefaction resulting from seismic events has become a major concern due to its impact on structures, buildings and other infrastructure during and after an earthquake. Liquefaction-induced ground failure has become one of the leading causes of infrastructure damage during an earthquake. Under seismic loading, the rapid increase in pore water pressure quickly decreases the shear strength of the unconsolidated sediment, possibly triggering large shear deformation. Flow failure of the ground during an earthquake may be caused by either the dynamic force due to the seismic acceleration or the static gravity force due to the topography of the ground (Tamate and Towhata 1999). Pile foundations have been widely used to support bridges, ports, and harbor facilities that are located in liquefiable soils. Flow deformation of liquefied soil could impose net lateral pressure on the pile, which may cause damage to pile and the supported structure. It is crucial to consider the possible load imposed by liquefied soil in pile design.

In this paper, numerical simulation was performed to study the dynamic responses of single pile in liquefied sand which is modeled as incompressible fluid. Compared with existing experiment, the pile-soil interaction was studied based on the numerical simulation results. Displacement and bending moment of the pile and soil response were analyzed.

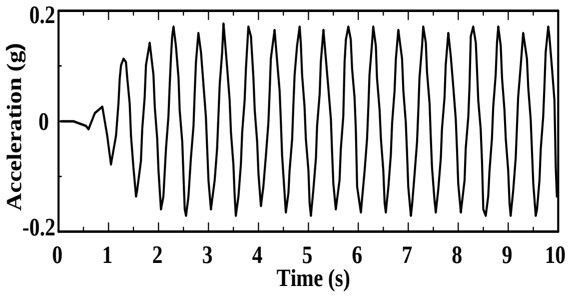
2 FINITE ELEMENT MODEL DETAILS

A pile and liquefied soil interaction problem was analyzed using the iterative coupling method in ADINA finite element software. The numerical model was established based on a shake table test conducted by Su et al. (2016). As shown in Figure 1, a pile was located behind a quay wall in the physical model. The quay wall acted as a soil retaining structure in a port. The quay wall and the pile were placed in a loose saturated sand layer with a thickness of 1.5 m and a relative density of 45%-50%. The water table is at the ground surface. The shear box had a dimension of 3.5 m $\times$ 1.7 m $\times$ 2.2 m. The pile was a 1.95 m long steel pipe with a wall thickness of 0.6 mm and an outer diameter of 0.088 m. The quay wall was a steel plate and was free to rotate about its base through a pin connection. The model dimensions were shown in Figure 1(a). See details of the model and test results in Su et al. (2016). The shake table test model was subjected to ground motion as presented in Figure 1(b).

The model dimensions in the physical experiment were used in the numerical simulation and the soil was simulated by viscous fluid. In the numerical model, the liquefaction process was not simulated. The simulation can be regarded as starting from the liquefaction stage, where the whole sand layer has fully liquefied. This simplification is acceptable since the interest is on the post-liquefaction dynamic responses of the pile and soil-pile interactions.



(a) Shake table model plot



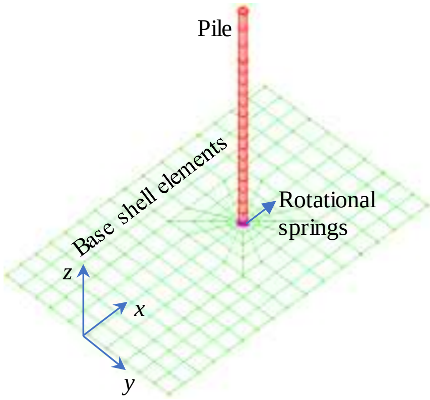
(b) Acceleration history of input motion.

Fig. 1. Shake table model test performed by Su et al. (2016)

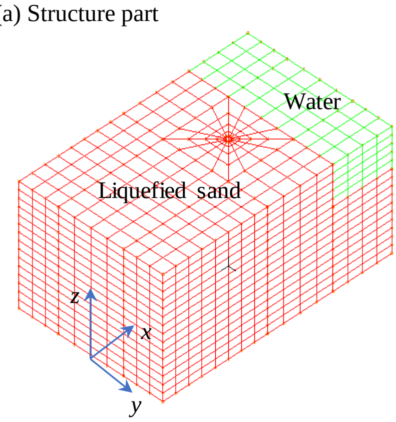
The liquefied saturated sand was simulated as an incompressible fluid with an equivalent viscosity. Pile was modeled by 3D solid elements with equivalent elastic properties. Water in front of the quay wall was also simulated as an incompressible fluid. The bottom of the numerical model was a layer of shell elements without thickness to simulate the shear box base in the shake table test. 32 rotational springs, connecting the pile and the bottom shell, were installed with an equivalent rotational stiffness of 3.75 kN·m/rad. This bottom shell boundary was assumed to be fixed in the transversal (y) and vertical (z) directions and free in the longitudinal (x) direction. For the fluid part, the front and rear boundaries in the transversal (y) direction were assumed to be fixed. The longitudinal fluid velocity (v_x) on the left and right boundaries was set zero. The fluid surface was set as free surface. The numerical model was shown in Figure 2. Each node of the pile element has three degree of freedoms (DOFs) for translation and three DOFs for rotation. The liquefied sand elements around the pile are finer than those far from the pile. A fluid-solid coupling surface was set up between the structure and the liquid. The ground motion as described in Figure 1(b) was applied to the model bottom shell nodes. The material parameters are shown in Table 1.

Table 1. Parameters used for the simulation

Parameters	Pile	Liquefied Soil	Water
Modulus of elasticity (GPa)	190	-	-
Density (kg·m^{-3})	2500	1800	1000
Possion's ratio	0.29	-	-
Viscosity (kPa·s)	-	20	0.001



(a) Structure part



(b) Fluid part

Fig. 2. Finite element discretization of the numerical model.

3 DISPLACEMENT AND BENDING MOMENT OF PILE

Figure 3 shows that the peak lateral displacement of the pile head in each cycle and its variation with time are generally consistent with the experimental results, except for in the first 3 seconds. However, the numerical curve fluctuates in a wider range than in the experiment. Based on the experiment, the seismic responses of the system can be divided into three stages: Stage 1 (0 – 2.3 s) prior to liquefaction, Stage 2 (2.3 – 6.8 s) liquefaction-induced lateral spreading, and Stage 3 (6.8 – 10 s) no net further lateral spreading development observed. Relative displacement between the liquified soil and pile was present in all the three stages and reached the maximum in Stage 3. Before the soil begun to liquefy (t=2.3 s) in experiment, the soil in the numerical simulation was treated as having liquefied and therefore showed lower shear strength than in experiment. Consequently, in Stage 1, the peak lateral displacement in each cycle increased sharply in experiment (non-liquefied soil) but this increase is much gentler in the simulation (viscous fluid).

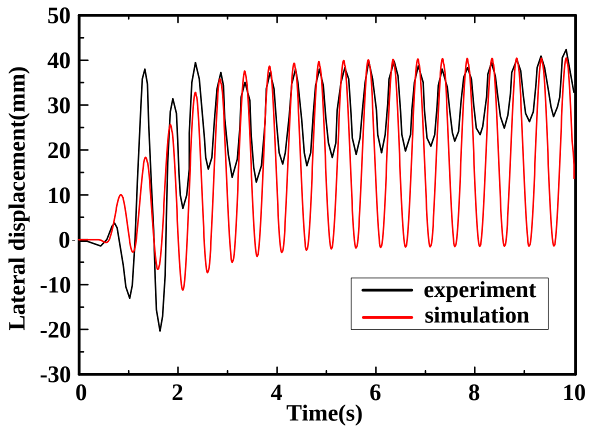


Fig. 3. Lateral displacements of pile head.

Figure 4 presents the displacement and bending moment of the pile compared with the experimental results when the pile head displacement was the maximum. Overall, the bending moment gradually increases along the depth, and the maximum bending moment occurs near the base, as expected in this cantilever beam configuration. The lateral displacement at the pile head is the maximum and the lateral displacement decreases with depth nonlinearly. The maximum pile head displacement (at the pile head) and moment (at the pile base) are close in the physical and numerical models. Although not quantitatively well matched along the whole depth, the patterns of displacement and bending moment distributions are generally consistent between the experimental results and the simulated ones.

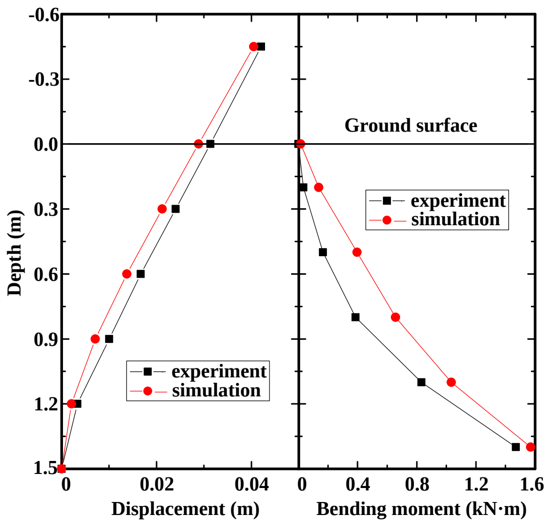


Fig. 4. Pile responses compared with the experimental results

4. SOIL RESPONSES

Figure 5 depicts the free-field accelerations in the numerical simulations compared with the experimental results. See the free-filed array of accelerometers in Figure 1(a). In the experiment, the acceleration history had gone through three different stages. In Stage 1, the amplitude of soil acceleration increased with cycle numbers. Buildup of pore pressure within the sand stratum was very fast, rapidly arriving at the initial liquefaction state. In Stages 2 and 3, the acceleration attenuated significantly as the sand stratum liquefied and then remained constant at a low level, indicating that the liquefied sand had a very low shear strength. In Stage 1, the peak accelerations at different depths are very close. In Stage 2, the peak acceleration increases significantly along the depth. From the experimental results, it can be found that the number of seismic cycles required for liquefaction at different depths is different. For deep soil layer, it is difficult to liquefy, and more seismic cycles are needed. The numerical modeling does not well simulate the acceleration responses of the soil layer along the depth in Stages 1 and 2 but is generally consistent with experiment in Stage 3 when the soil has liquified. In both the simulation and experiment, the post-liquefaction soil in Stage 3 behaves in a similar manner to a viscous fluid and the fluid can bear great shear strain and consume the energy of seismic waves, leading to a low amplitude of the acceleration. Therefore, it is effective to simulate liquefied sand as viscous fluid.

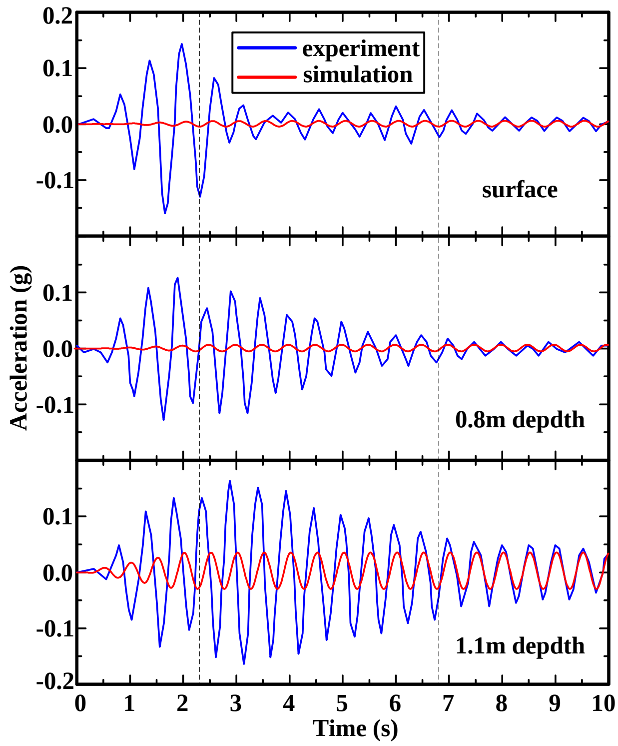


Fig. 5 Free-field accelerations compared with the experimental results.

5. LATERAL SOIL PRESSURE

Two simplified lateral soil pressure distribution models have been widely used to consider the interactions between liquefied soil and pile in design practice. The first one is proposed by Dobry et al. (2003) recommending a uniform soil pressure distribution. The second one suggested by Japan Road Association (2002) recommends a triangular soil pressure distribution and a lateral pressure coefficient of 0.3 is recommended. Besides, Su et al. (2016) proposed a relatively complicated Beam on Nonlinear Winkler Foundation (BNWF) model. The BNWF model employs an elastic beam to simulate the pile and nonlinear *p-y* spring elements to represent the behavior of soil–pile interaction. American Petroleum Institute (2000), Li et al. (2009), and Guo et al. (2014) further developed the BNWF model by incorporating some modifications (Su et al. 2016). The above mentioned three models are compared with the results in our numerical simulations in Stage 3, as shown in Fig. 6. A uniform soil pressure of 4.8 kPa for Dobry's model and a coefficient 0.7 for the triangular soil pressure distribution were calibrated to approximate the trend of the observed bending moment, but with an underestimation of the soil pressure at the base. See details of the model parameter calibration in Su et al. (2016). Figure 6 shows that the BNWF model resembles our simulation results and such nonlinear increase of soil pressure with depth is consistent with experimental results by Su et al. (2016). Figure 6 further illustrates the inadequacy of employing a triangular or uniform soil pressure distribution.

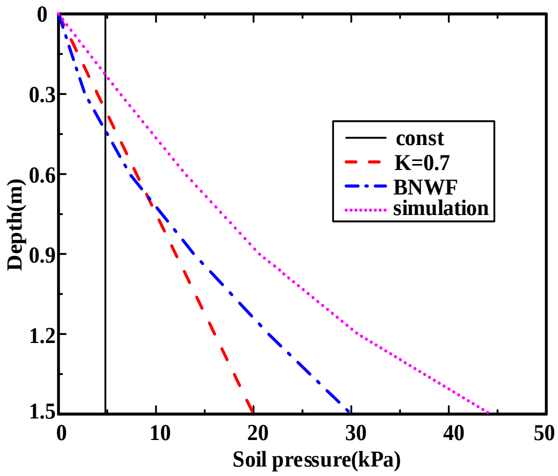


Fig. 6 Comparisons of soil pressure models.

6 CONCLUSIONS

The results of a numerical simulation of pile-liquefied sand interaction is compared with the shake table experiment to study the dynamic behavior of a single pile subjected to liquefaction-induced lateral relative movement. In view of fluid mechanics, the model of pile-soil interaction was discussed. Based on the investigated scenario, the following conclusions can be drawn:

(1) Considering liquefied sand as a fluid, the numerical simulation can well capture the general features of soil response, pile displacement/moment and soil pressure imposed by liquefied soil on the pile, indicating that post-liquefaction soil behaves in a similar manner to a viscous fluid.

(2) The lateral soil pressure distribution in liquefied sand increases with depth in a nonlinear manner. A triangular or uniform soil pressure distribution is not adequate to describe the soil pressure.

(3) More shake table experiments and numerical simulations with various scenarios should be conducted to investigate other issues, such as time-varying viscosity, and effects of overburden pressure and earthquake parameters, which are also significant in the seismic design of pile foundations.

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