Numerical Simulation of Dynamic PSSI System Considering Liquefaction

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Abstract

Dynamic soil-structure interaction considering liquefaction is a very important topic in the field of earthquake engineering. A three-dimensional numerical analysis of pile-soil-structure interaction (PSSI) system in liquefiable site is carried out. General finite differential program FLAC3D is adopted in the analysis. Boundary condition of the model, soil nonlinearity, nonlinearity on the soil-structure interface and pore water pressure build-up process are taken into account in the calculation model. The computational model and analysis method is verified through comparison study between the calculation and the shaking table test results. The distribution of the pore pressure ratio in sand soil and the acceleration of PSSI system are discussed in this paper. At last, some important findings are concluded.

Keywords: liquefaction; numerical simulation; pore water pressure; shaking table test

1. Introduction

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased in pore-water pressure and reduced effective stress (Marcuson et al. 1978). Earthquake ground motions causing soil liquefaction can be a major cause of failures of earth structures, dams, slopes, and foundations. Liquefaction analysis has been developed since the 1980s by many researchers. The field equations of current liquefaction analysis are based on Biot's porous media theory (Biot MA, 1962). Most precedent studies, however, relied upon 2-D analysis to assess linear structures. Few investigators developed dynamic 3-D soil-water coupled analysis based on effective stress concept (Ohtsuki et al. 1994). Finn and Thavaraj (2001) validated their 3-D soil-water coupled analysis method for the results of shaking table tests with group-piles in liquefiable ground under gravitational and centrifugal field. The dynamic behavior of group-piles in liquefiable ground depends on the nonlinear material properties of liquefied soil and piles. Therefore, general findings cannot be inferred from a limited number of experiments. Further study of dynamic soil behavior in liquefied ground through simulations of model tests and case histories, along with soil-water coupled analysis are required.

In order to investigate the effects of soil liquefaction on the dynamic pile-soil-structure interaction (PSSI) system, shaking table model tests on free field and pile-soil-structure interaction system in liquefiable site were done in the State Key Laboratory of Disaster Reduction in Civil Engineering at Tongji University, in China. These tests have provided a significant number of experimental data. On the basis of these shaking table model test data, combined with the results of the general finite differential program FLAC3D (Itasca Inc. 2002), 3-D effective stress analysis on PSSI system in liquefiable site is presented in this paper.

2. Brief Description of Shaking Table Model Test

In shaking table tests, the model soil should be held in a box of reasonable size. Because of wave reflection on the boundary and variation in the vibration mode of the system, an error called boundary effects will affect the test results. To reduce the boundary effect, a flexible container and proper constructional details were designed for the model test, and the ratio of the ground plane diameter to the structural plane diameter was set at 5 by controlling the size of the structural plane. The cylindrical container was 3000 mm in diameter, and its lateral rubber membrane was 5 mm thick. Reinforcement loops of 4 mm diameter, spaced at 60 mm, were used to strengthen the outside of the container in the tangential direction. The model soil was composed of two layers. The top layer consisted of silty clay, and the bottom layer was saturated sand.
The top clay depth of the model was 0.2 m, and the bottom sand depth was 1.3 m. For the foundation of the superstructure, nine piles of a pile group were used. The superstructure was a 12-story reinforced concrete frame, with a single bay and a single span. Two 1/10 scale tests of free field and PSSI system were designed according to well-established similitude relations (Li et al. 2008).

The measuring points in PSSI system model test were arranged as shown in Fig.1. Accelerometers and strain gauges were used to measure the dynamic responses of the superstructure, the foundation, and the soil. Pore pressure gauges were used to measure the pore water pressure in the process of the vibration. Strain gauges were used to measure the strain of the piles. Pressure gauges were used to measure the contact pressure between piles and the surrounding soil. Ground shaking was simulated as unidirectional (X-axis direction of shaking table) motions. The records selected for the study included the 1940 El Centro wave and Shanghai bedrock wave. Five levels of excitation were used in this study. From level 1 to level 5 the values for peak accelerations were, respectively, 0.131g, 0.375g, 0.75g, 1.125g, and 1.5g. The time interval was 0.003266 s.

The emphasis of this paper is mainly put on the numerical simulation of the PSSI system model test in liquefiable site.

3. Modeling Method
3.1 Simulation of Soil Nonlinearity
The equivalent-linear method has been in use for many years to calculate the seismic response in soil and rock at sites subjected to seismic excitation. However, the method does not directly capture any nonlinear effects because it assumes linearity during the solution process; strain-dependent modulus and damping functions are only taken into account in an average sense. In order to approximate effects of nonlinearity, fully nonlinear method is employed in this analysis with the hysteretic damping.

The hysteretic damping allows strain-dependent modulus and damping functions obtained from the dynamic test to be incorporated directly into the FLAC3D program. Modulus degradation curves, as illustrated in Fig.2., imply a nonlinear stress-strain curve. If we assume an ideal soil, in which the stress depends only on the strain (not on the number of cycles, or time), we can derive an incremental constitutive relation from the degradation curve, described by

$$\tau = M_s \gamma$$

Where $\tau$ is the normalized shear stress, $\gamma$ the shear strain and $M_s$ the normalized secant modulus.

$$M_s = \frac{d\tau}{d\gamma} = M_t + \gamma \frac{dM_t}{d\gamma}$$

where $M_t$ is the normalized tangent modulus. The incremental shear modulus in a nonlinear simulation is then given by $GM_s$, where $G$ is the small-strain shear modulus of the material.

Hardin model of the hysteretic damping is used to simulate nonlinearity of the soil which has the following function:

$$M_s = \frac{1}{1 + \gamma/\gamma_{ref}}$$

Where $\gamma_{ref}$ is reference shear strain, which may be determined according to the modulus reduction curve.

3.2 Simulation of the Boundary Condition
The behavior of the flexible container in the shaking table test should be included in the modeling of shaking table model test on PSSI. The base plate of the container was rigidly bolted to the shaking table. Crushed rock was attached to the base plate by epoxy resin to create a rough interface between the soil and the base during the test. This ensured a negligible relative slip between the soil and the bottom surface of
the container and justified the fixed-base assumption in the computer model. Reinforcement loops outside the container were used to provide radial rigidity to the system and to permit the soil to deform as a series of horizontal shear layers during the test. In the computer model the reinforcement loops were modeled that the nodes at the same height along the container perimeter have the same displacement in the excitation direction (X-axis direction of shaking table), which was realized by the FISH programming language embedded within FLAC3D.

3.3 Pore-pressure Model

Dynamic pore-pressure generation can be modeled with the Martin-Finn-Seed model. Noting that the relation between irrecoverable volume-strain and cyclic shear-strain amplitude is independent of confining stress, Martin (1975) supplies the following empirical equation that relates the increment of volume strain, \( \Delta \varepsilon_{vd} \), to the cyclic shear-strain amplitude, \( \gamma \), where \( \gamma \) is presumed to be the engineering shear strain:

\[
\Delta \varepsilon_{vd} = C_1 (\gamma - C_2 \varepsilon_{vd}) + \frac{C_3 \varepsilon_{vd}^2}{\gamma + C_4 \varepsilon_{vd}}
\]

(4)

where \( C_1, C_2, C_3 \) and \( C_4 \) are constants, \( \varepsilon_{vd} \) is accumulated irrecoverable volume strain. In this paper: \( C_1 = 0.8, C_2 = 0.79, C_3 = 0.45 \) and \( C_4 = 0.73 \).

3.4 Simulation of Nonlinearity on the Soil-Structure Interface

Because of the different characteristics of the soil and the concrete of the foundation, sliding and separation may occur at the soil-structure interface when the interface stress reaches a certain limit. The contact between piles and the soil is realized by pileSELs in FLAC3D. Piles interact with the soil via shear and normal coupling springs. The normal coupling springs can simulate the effect of the soil squeezing around the pile. The shear coupling springs can simulate the effect of the frictional and cohesive behavior of the pile-grid interface. The contact between the bearing platform and the soil is realized by interfaces in FLAC3D. Interfaces have the properties of friction, cohesion, normal and shear stiffnesses, tensile and shear bond strength. By the interface constitutive model, the absolute normal penetration and the relative shear velocity can be calculated during each timestep.

3.5 Meshing

Both the frequency content of the input wave and the wave speed characteristics of the system will affect the numerical accuracy of wave transmission. Kuhlemeyer and Lysmer (1973) show that, for accurate representation of wave transmission through a model, the spatial element size, \( \Delta l \), must be smaller than approximately one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave:

\[
\Delta l \leq \left( \frac{1}{8} - \frac{1}{10} \right) \lambda
\]

(5)

Where \( \lambda \) is the wavelength associated with the highest frequency component.

Fig.3. shows the meshing of the PSSI test model satisfying the above requirements.

4. Calculation Results

In this paper, pore pressure ratio and acceleration response of the system will be discussed in details.

4.1 Pore Pressure Ratio

Fig.4. and Fig.5. show the comparison of pore pressure ratio time histories at different measuring points under excitation of EL2 and EL3 (the figure is after correction of zero line). Excitation of EL2 and EL3 represent the excitation of the El Centro wave, with a peak acceleration of 0.375g and 0.75g, respectively. From these figures, we can draw the rules as follows.

(1) The pore pressure ratio in the sand soil layer increases quickly with the increasing of the acceleration value. The pore pressure ratio mainly dissipates after the excitation and the velocity of the dissipation tends to slow gradually from top to bottom.

(2) The pore water pressure and the pore pressure ratio increase with the increasing peak acceleration of the input excitation except the measuring points H2 and H3 of which the pore pressure ratio reaches to 1.

(3) It can also be seen that the transient negative pore pressure occurs when the initial acceleration reaches the peak value. The main reason is that the sand soil layer performs transient swelling effect when the initial acceleration reaches the peak value. So it is not pressure but suction at the moment and the transient value obtained by the pore water pressure gauge is negative.
Fig. 4. Comparison of Pore Pressure Ratio between Calculation and Test Results (EL2 case)

Fig. 5. Comparison of Pore Pressure Ratio between Calculation and Test Results (EL3 case)
(4) The value of the pore pressure ratio has relation with the depth and the location whether the measuring points lie below the foundation. For the measuring points at the same depth, the value of the pore pressure ratio lying below the foundation is obviously larger than that lying outward the foundation. It indicates that the existence of the foundation and the superstructure has influence on the increasing of the pore water pressure. The pore pressure ratio of point H2 and H3 with shallowly buried depth reaches 1.0 and the sand soil liquefies under the excitation of EL3.

(5) The pore water pressure does not dissipate immediately after the vibration stops. However, it may continue to increase in a short time (The input seismic wave lasts about 8.8s. The time to acquire the data by pore water pressure gauge is about 294.3s). The soil stays in instable non-linear deformation and the soil surface sinks under the excitation. As the vibration stops, the instability can not stop immediately and the deformation continues. That is the reason that the pore water pressure continues to increase after the earthquake.

4.2 Acceleration

S5, S6, S7, S8, S9 and S10 are measuring points in soil with the same plane position while depth increases gradually and SD is a measuring point on the container base. Fig.6. and Fig.7. show the comparison of the acceleration time history between calculated and test results for the measuring points. From these figures, we can draw the rules as follows.

(1) The calculated and test results basically agree well, although there is certain difference on wave
shape and wave amplitude.

(2) In the figure, the acceleration amplitude of the measure points first decreased and then increased as the distance between the measuring points and the container base increased. The response of the middle point is the smallest.

(3) Magnification or reduction of vibration transferred by soil is related to soil characteristic, excitation magnitude, spectral characteristics of excitation, and so on. Sand magnifies vibration under small earthquakes. Under medium earthquake and strong earthquake, however, it will damp vibration. Because sand soil liquefies, non-linear behavior develops and stiffness declines because of the increasing of the pore water pressure.

5. Conclusions

A three-dimensional research on modeling of PSSI system based on shaking table tests in liquefiable soil has been conducted with the FLAC3D software. By comparison study between the calculation and the tests, it's verified that the modeling methods are rational and the developed model is suitable for the numerical analysis of the PSSI system in liquefiable site.

Issues drawn from the calculation, which are consistent with those drawn from the tests, are as follows. 1) The pore water pressure in the sand soil layer increases quickly with the increasing of the input acceleration. 2) The soil layer with shallowly buried depth liquefied more easily. 3) The value of the pore pressure ratio has relation with the depth and the location whether the measuring points lie below the foundation. 4) The pore water pressure does not dissipate immediately after the vibration stops. 5) As the distance between the measuring points and the container base increased, the acceleration amplitude of the measure points first decreased and then increased. The response of the middle point is the smallest. 6) Sand soil can filter and isolate vibration.

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