COMPUTER SIMULATION ON DYNAMIC SOIL-PILE-SUPERSTRUCTURE INTERACTION SYSTEM CONSIDERING LIQUEFIABLE FOUNDATION

Hongmei Ren[1], Xilin Lu[2] and Peizhen Li[3]

[1]*PH.D student, State Key Lab. for Disaster Reduction in Civil Eng., Tongji Univ., Shanghai, 200092, China*
[2]*Professor, State Key Lab. for Disaster Reduction in Civil Eng., Tongji Univ., Shanghai, 200092, China*
[3]*Associate Professor, State Key Lab. for Disaster Reduction in Civil Eng., Tongji Univ., Shanghai, 200092, China*
Email: hmren2005@yahoo.com.cn

ABSTRACT:
A three-dimensional numerical analysis of shaking table tests on dynamic soil-pile-superstructure interaction (SSI) system considering soil liquefaction is presented in this paper. The finite difference method FLAC^{3D} is used in the analysis. A 12-story reinforcement concrete frame structure is employed to simulate the superstructure. Nonlinearity of the soil and pore water pressure build-up process, and the interface behavior between piles and surrounding soil under seismic input is taken into account in the model. With the established model, variation process of dynamic pore water pressure of the soil is discussed and some important findings are obtained. By comparing the results of the numerical analysis with the data from shaking table tests, the computational model is validated.

KEYWORDS: Liquefaction, Pore water pressure, Seismic response, Numerical simulation

1. INTRODUCTION

Liquefaction analysis has been developed since the 1980s by many researchers. However, most precedent studies relied upon 2-D analysis to assess linear structures such as embankments, quay walls, etc. 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 soil structure interaction (SSI), shaking table model tests on free field and soil-pile-structure considering soil liquefaction were carried out in the State Key Laboratory for Disaster Reduction in Civil Engineering at Tongji University, in China (Li *et al.*, 2008). A three-dimensional numerical simulation of shaking table tests on SSI considering soil liquefaction is presented in this paper. The general finite differential program FLAC^{3D} (Itasca Consulting Group, Inc., 2002) is used in the analysis, in which nonlinearity of the soil and generation of pore water pressure are considered.

2. MODELING METHOD

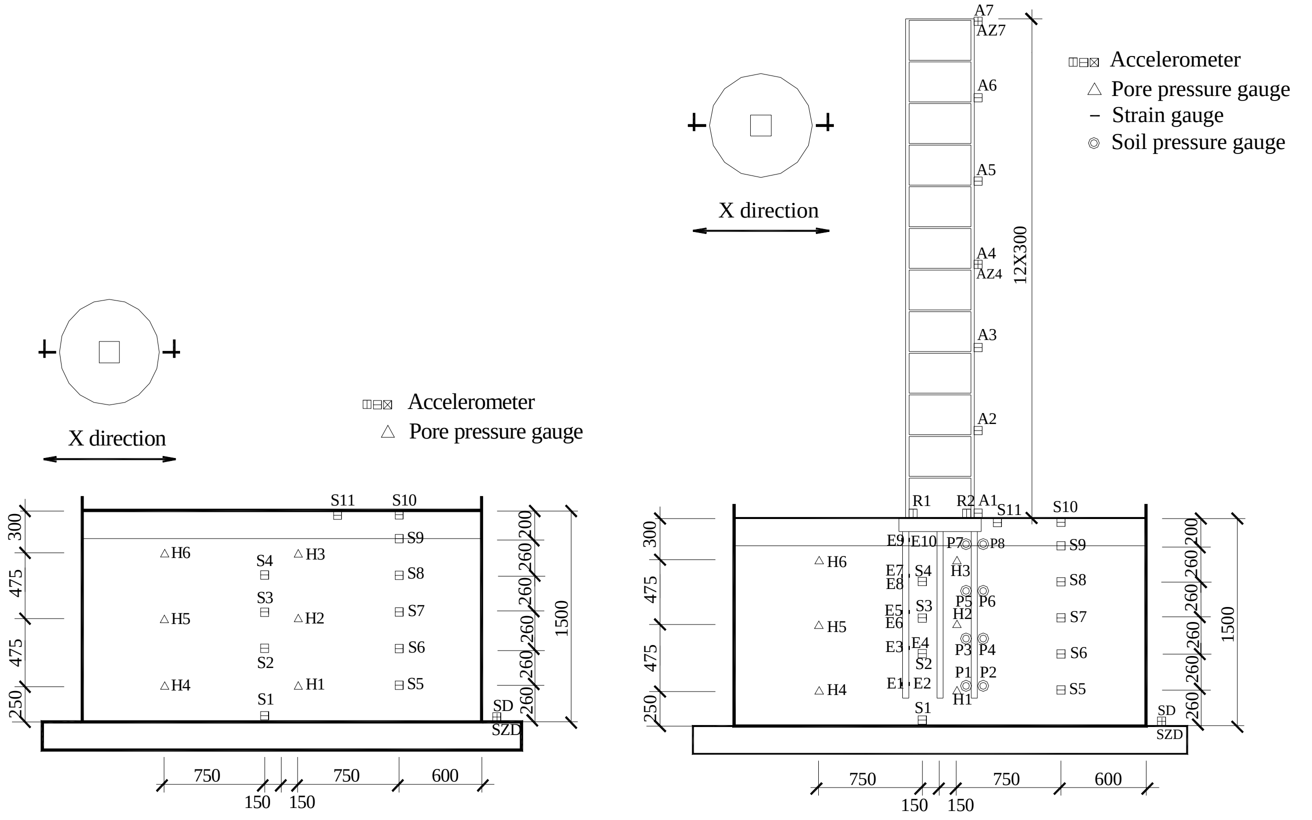
2.1. Brief description of shaking table tests

In the shaking table test, the model soil should be held in a box of reasonable size. Because of wave reflection on the boundary 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 (i.e., *D*/*d*) 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 soil model 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 models of free field (FF10L) test and the soil-pile-superstructure system (PS10L) test were designed according to well-established similitude relations (Sabnis *et al*., 1983; Lu *et al*., 1999).

The measuring points in shaking table tests were arranged as shown in Figure1. Accelerometers were used to measure the dynamic responses of the system, and pore pressure gauges were used to measure the pore water pressure in the process of the vibration.



(a) Free field (b) Soil-pile-superstructure interaction system

Figure1 Arrangement of measuring points in shaking table tests

Ground shaking was simulated as unidirectional (*x* direction of the shaking table) motions. The records selected for the study included (*i*) a record from the 1940 El Centro earthquake; and (*ii*) a 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, 0.131, 0.375, 0.75, 1.125, and 1.5g. The time interval for the 1/10 scale test was 0.003266 s (Li *et al*., 2008).

2.2. Modeling steps

Prior to performing mechanical-groundwater flow coupling analysis, an initial force-equilibrium state must be obtained. The initial stress of the model in vertical direction could be calculated when gravity was applied. Underground water was taken into account by applying static water pressure. After a mechanical equilibrium and steady-state flow state was reached, mechanical-groundwater flow coupling analysis could be performed.

2.3. Boundary conditions

2.3.1. Simulation of the flexible container

The behavior of the flexible container in the shaking table test should be included in the modeling of the SSI system. 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 as follows. The nodes at the same height along the container perimeter have the same displacement in the excitation direction (*x* direction of the shaking table), and this was realized by the FISH programming language embedded within FLAC^{3D}.

2.3.2 Dynamic boundary conditions

In mechanical-fluid coupling stage, we should take the dynamic boundary conditions into account. In FLAC^{3D}, the dynamic input can be applied in one of the following ways: (a) an acceleration history; (b) a velocity history; (c) a stress (or pressure) history; or (d) a force history. In this simulation the acceleration histories of El Centro and Shanghai bedrock wave were applied to the bottom of the model. Lateral and bottom boundaries were set impermeable in FLAC^{3D}.

By comparing calculated and test results, it was verified that the modeling method adopting the above boundaries was suitable for the numerical analysis of SSI considering soil liquefaction.

2.4. Parameters of the model soil

The elastoplastic model was employed to simulate the nonlinearity of the soil. A resonant column test and a cyclic triaxial test were carried out for the remolded soil and we can get the following soil parameters. For the clay: density of the clay is 1.842 g/cm^3, pore pressure ratio is 0.902, saturation degree is 93.8%, dynamic shear modulus is 71.56 MPa, passion ratio is 0.42 and water content is 31.0%. For the sand soil: density of the sand soil is 1.96 g/cm^3, pore pressure ratio is 0.574, saturation degree is 83.3%, dynamic shear modulus is 33.83 MPa, passion ratio is 0.32 and water content is 17.9%.

2.5. Dynamic pore-pressure model

FLAC^{3D} can calculate pore-pressure effects, with or without pore-pressure dissipation. Also, dynamic pore-pressure generation can be modeled by accounting for irreversible volume strain in the constitutive model. This is done with the built-in constitutive models: the Finn model.

The relation between pore-pressure increment and irrecoverable volume-strain increment under undrained condition can be expressed as:

$$\Delta u = \bar{E}_r \Delta \varepsilon_{vd} \tag{1}$$

Where Δu is pore-pressure increment, $\bar{E}_r$ is rebound modulus of the soil, $\Delta \varepsilon_{vd}$ is irrecoverable volume-strain increment.

Noting that the relation between irrecoverable volume-strain and cyclic shear-strain amplitude is independent of confining stress, Martin et al. (1975) supply the following empirical equation that relates the increment of volume strain, $\Delta \varepsilon_{vd}$, to the cyclic shear-strain amplitude, γ, where γ is presumed to be the engineering shear strain:

$$\Delta\varepsilon_{vd} = C_1\left(\gamma - C_2\varepsilon_{vd}\right) + \frac{C_3\varepsilon_{vd}^2}{\gamma + C_4\varepsilon_{vd}} \tag{2}$$

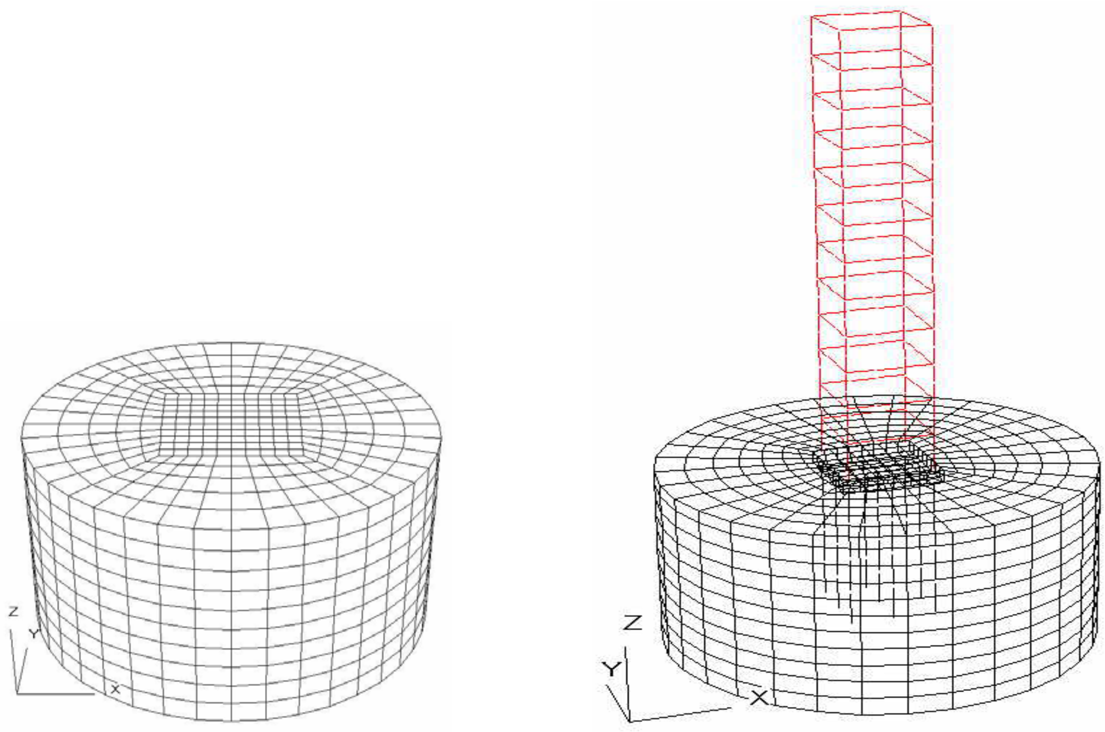
where C_1, C_2, C_3, and C_4 are constants, ε_{vd} is accumulated irrecoverable volume strain. In this paper: $C_1 = 0.8$, $C_2 = 0.79$, $C_3 = 0.45$, and $C_4 = 0.73$.

2.6. Wave transmission

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, Δl, must be smaller than approximately one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave - i.e.,

$$\Delta l \le \left(\frac{1}{8} \sim \frac{1}{10}\right)\lambda \tag{3}$$

Where λ is the wavelength associated with the highest frequency component.



(a) Free field (b) soil-pile-superstructure interaction system

Figure 2 Meshing of FF10L and PS10L test models

For dynamic input with a high peak velocity and short rise-time, the Kuhlemeyer and Lysmer requirement may necessitate a very fine spatial mesh and a corresponding small time step. With finer mesh and more degrees of freedom, precision is higher and the time taken for the calculation is longer. Thus, a proper grid size should be used. Figure 2 shows the meshing of FF10L and PS10L test models, which satisfies the above requirements.

2.7. Mechanical damping model

FLAC^{3D} supplies three forms of mechanical damping model in dynamic analysis: Rayleigh damping, Local damping and Hysteretic damping. Local damping is originally designed as a means to equilibrate static simulations. However, it has some characteristics that make it attractive for dynamic simulations. It can be expressed as:

$$\alpha_L = \pi D \tag{4}$$

where α_L is the local damping coefficient, and D is the critical damping.

Local damping appears to give good results for a simple case because it is frequency independent and needs no estimate of the natural frequency of the system being modeled. Due to the saturated sand and simple boundary conditions, local damping was used to simulate soil liquefaction in this paper.

3. CALCULATION RESULTS

The change of pore water pressure in the sand soil was obtained by nine high-sensitivity pore water pressure gauges embedded in the sand soil (The arrangement of the measuring points is shown in Figure 1). In this section, some data of the pore water pressure were simulated and changing rules of the pore pressure ratio were studied.

Figure 3~Figure 6 show the comparison of pore pressure ratio time history at different measuring points in both FF10L and PS10L test models 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) From time-history curves of pore pressure ratio, we can see calculated and test results agree well, which demonstrates the modeling method is rational.

(2) From these figures it can be seen that the pore water pressure in the sand soil layer increases quickly with the increasing of the acceleration value. And there is not obvious dissipation of the pore water pressure within the time period recorded because of the limited time for recording (The inputted seismic wave lasts about 8.8s. The time to acquire the data by pore water pressure gauge is about 11.7s). 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.

(3) Both test and calculated results increase gradually as the depth decreases under the seismic excitation, which indicates the soil layer with shallowly buried depth liquefied more easily. It agrees well with test phenomenon and actual seismic damage investigation. The main reason is the effective stress of the soil decreases and the seismic response of the soil layer increases with soil depth decreasing, and pore pressure ratio is proportional to the effective stress of the soil and inversely proportional to the seismic response of soil layer.

(4) It can be seen from the figures that the pore pressure ratio of all points does not reach 1.0 and the sand soil does not liquefy under excitation of EL2. The pore pressure ratio of Point H6 with shallowly buried depth reach 1.0 and the sand soil liquefy under excitation of EL3. It is just at this point that we observe the phenomenon of sand boiling. Furthermore, the sand soil at the bottom liquefies gradually as the excitation increases. The reason may be that pore water at the bottom transfers up easily while the pore water at the top of the sand soil is difficult to drain out through the clay soil layer (the drainage path of the clay soil layer is blocked). And there is certain supplement from the pore water at the bottom. The result is basically consistent with the saturated soil test done before.

(5) The time history curves indicate that there are almost the same rules of pore pressure ratio increase for the measuring points with the same depth at the same horizontal plane.

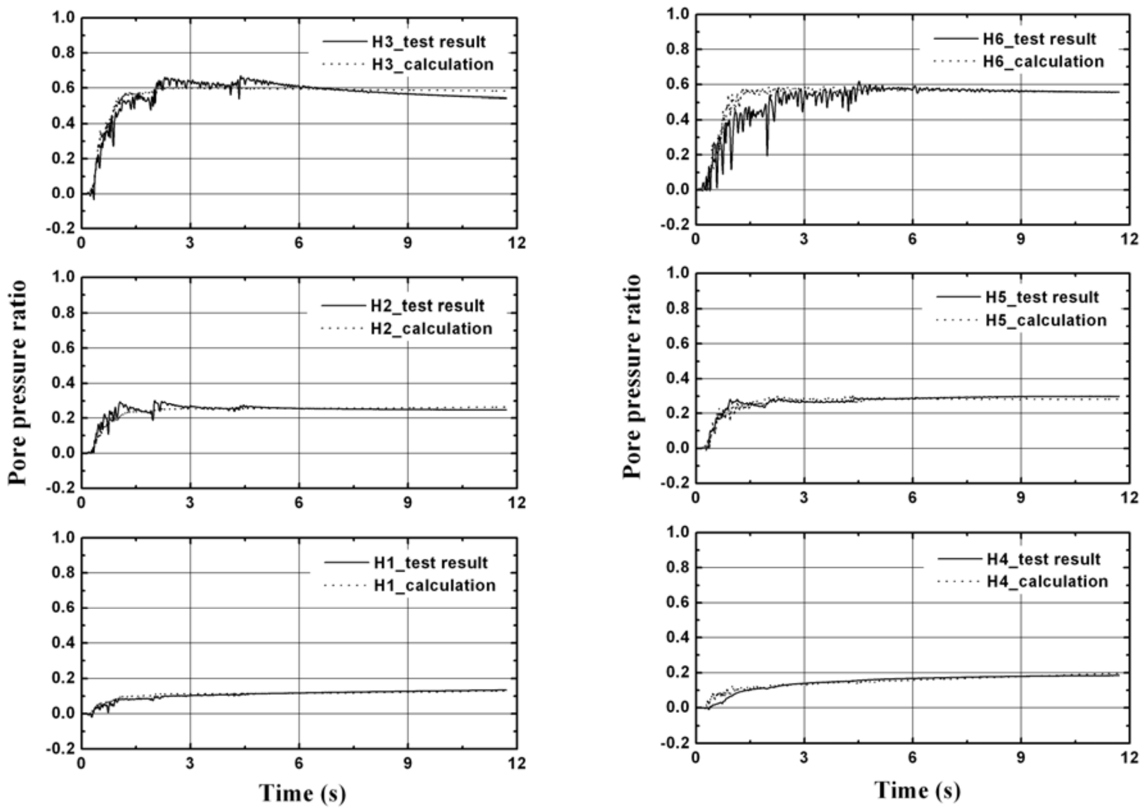


Figure 3 Comparison of pore pressure ratio between calculated and test results (EL2, FF10L test model)

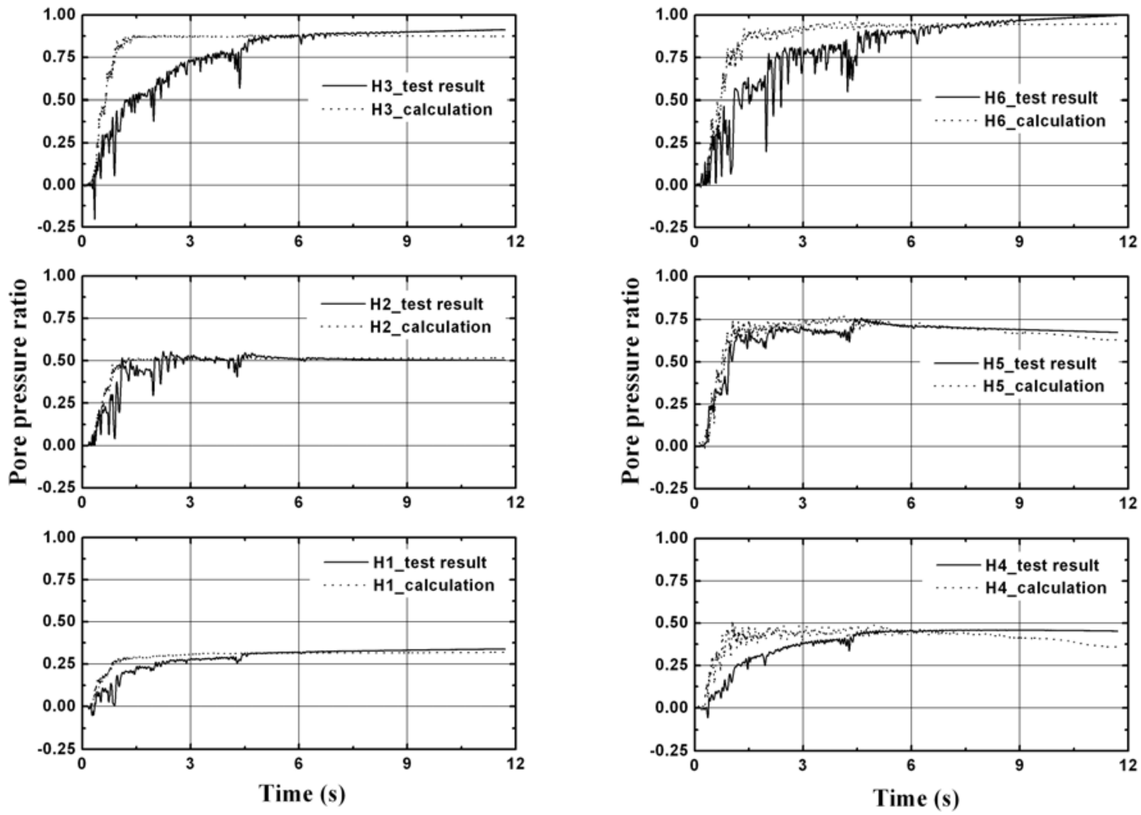


Figure 4 Comparison of pore pressure ratio between calculated and test results (EL3, FF10L test model)

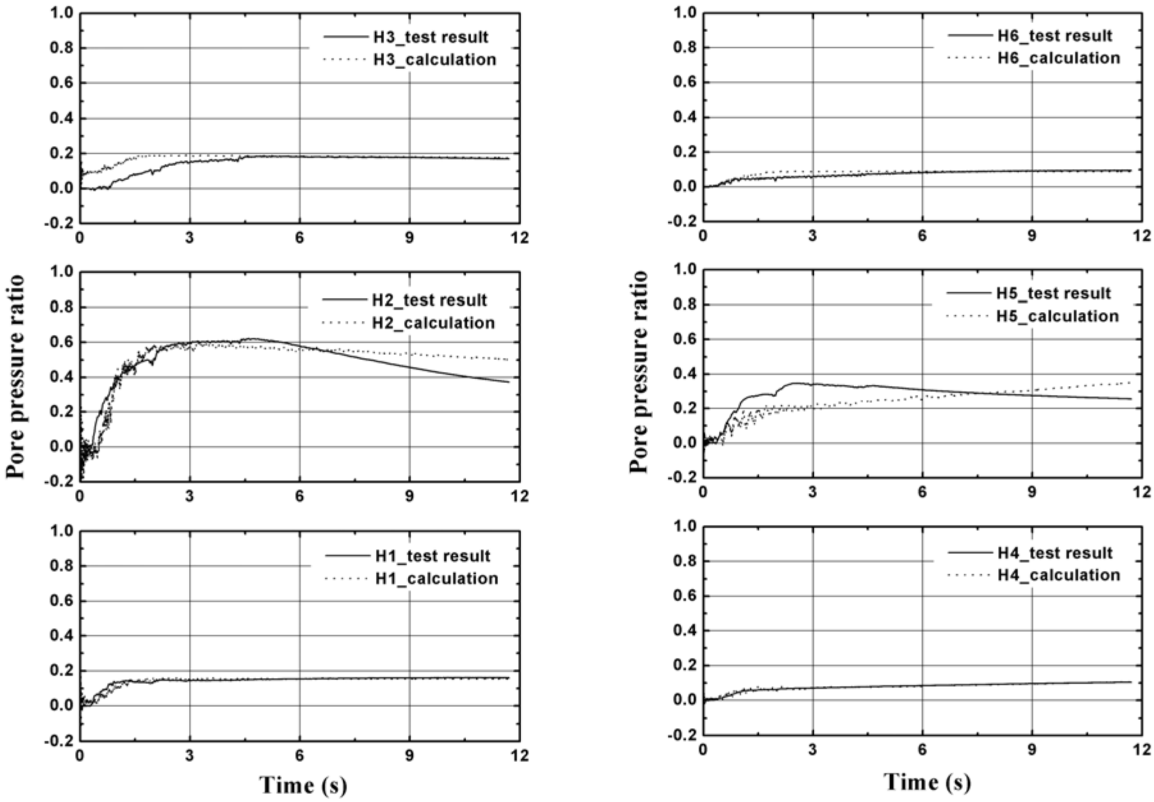


Figure 5 Comparison of pore pressure ratio between calculated and test results (EL2, PS10L test model)

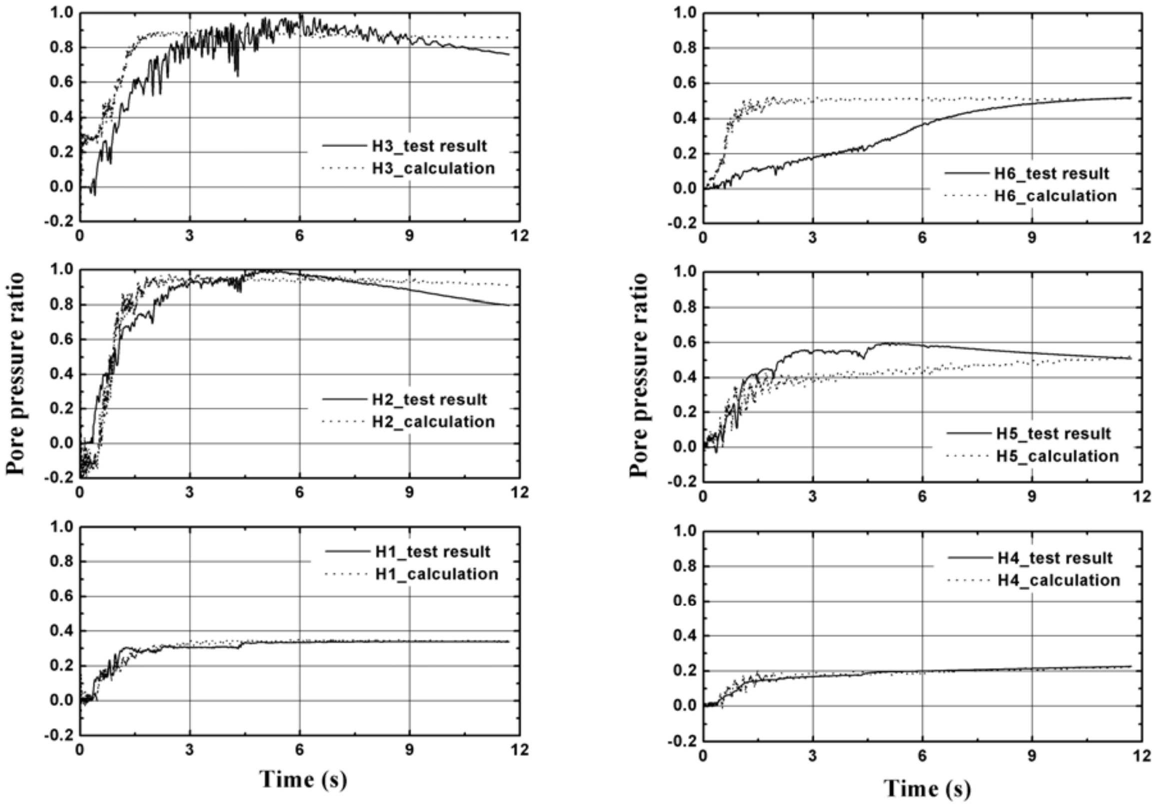


Figure 6 Comparison of pore pressure ratio between calculated and test results (EL3, PS10L test model)

4. CONCLUSIONS

A three-dimensional simulation of shaking table tests on SSI in liquefiable soil was conducted with the FLAC^{3D} software. The Finn pore-pressure model and the elastoplastic constitutive model were used to simulate the soil. Pore pressure ratio of the soil were calculated under seismic excitation. By comparing calculated and test results, it was verified that the modeling method is rational. The simulation method developed in this paper can be used to establish the 3D analytical model for the seismic response of SSI system in liquefiable site.

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