Validation of numerical model based on large-scale shaking table test of liquefied site

Kemin Jia¹, Chengshun Xu¹* Pengfei Dou¹, Jia Song²

¹ Key Laboratory of Urban Security and Disaster Engineering of Ministry of Education (Beijing University of Technology, Beijing, 100124, China)
² School of Civil Engineering, (North China University of Technology, Beijing 100144, China)

First author’s email: 245442602@qq.com
*Corresponding author’s e-mail: xuchengshun@bjut.edu.cn

ABSTRACT: Based on the completed large-scale shaking table test of liquefaction free field, the OpenSees computational platform was used to establish a numerical model of liquefiable free-field shaking table test in this study. In the numerical modeling, simulation of soil displacement and pore pressure using two-phase fully coupled u-p form. The comparison of shaking table test results shows that the established numerical model can reasonably simulate the seismic response of liquefiable soils, and further verify the reliability of the numerical model. At the same time, the distribution characteristics of soil excess pore water pressure at the critical moment were analyzed. This paper provides an effective numerical method for dynamic response analysis of liquefied soil.

1. Introduction
Foundation failure caused by soil vibration and liquefaction often results in structural damage, which is a theoretical and practical research subject in the field of geotechnical earthquake engineering (2013) [1]. Free-field seismic response analysis is the basis of soil-structure seismic response analysis. Because the seismic response of saturated sand is complex and has many influencing factors, it is necessary to conduct research on seismic response of liquefaction free field.

Seismic simulation shaking table test and finite element numerical simulation are effective methods to study the seismic response of liquefiable sites. For example, Varghese R.Dobry (2014,2011) [2,3] conducted a free-field 1 g shaking table test of a liquefaction site to study the dynamic response of free-field soil. Zhou (2017) [4] conducted a centrifuge shaking table test on the liquefaction induced lateral spreading site based on the LEAP project, providing high-quality comparative data for numerical analysis of liquefied soil. D.Chang (2013) [5] studied the structural response in a liquefaction site using the finite element method. Shaking table test is expensive, the period is long, and the test conditions are not easy to control. It is not realistic to carry out a large number of test. It is necessary to develop a reliable and effective numerical simulation method to study the seismic response of liquefied soil.

In this study, a two-dimensional finite element model of a liquefaction free-field shaking table was established, based on the completed large-scale shaking table tests and the open source finite element numerical simulation platform OpenSees. The validity of the proposed numerical model is verified by comparing the test results with the numerical results.
2. Shaking table test
Using the large laminal shear box at the State Key Laboratory of Building Safety and Environment, China Academy of Building Research, the overall size of the model foundation was 3.2-m long, 2.4-m wide, 2-m high. The corresponding soil stratum configuration and sensor layout were shown in Figure 1. The model consisted of a 0.5m dense sand layer, 1.2m loose sand layer, and 0.3m clay cover layer from ground surface to bottom, and the soil stratum was constructed by sand deposition in water. After the soil stratum was prepared, it was allowed to stand for 24 hours to consolidate naturally, and the water level was kept flush with the ground surface. The test mainly measured the acceleration and pore pressure response. This shaking table test used the Wenchuan earthquake and Wolong earthquake records as the input ground motion, with a peak value of 0.3g. Acceleration time history and spectrum characteristics are shown in Figure 2.

Soil surface conditions before and after the shaking were shown in Figure 3, it was found that the soil surface was flat before the test and there was no stagnant water as shown in Figure 3(a). There were obvious waterspouts and sand boils at the ground surface after shaking as shown in Figure 3(b). This phenomenon provides extremely strong evidence for occurrence of sand liquefaction.
3. OpenSees numerical model

The 2D numerical model of shaking table test was built by using OpenSees framework, which is an open source finite element software, and the post-processing was performed with GID. The discretization of the numerical model is illustrated in Figure 4. In the numerical model, the saturated soil was simulated using QuadUP elements. This element simulated saturated sand as a two-phase material based on the Biot’s theory for porous media. In numerical calculations, the Biot formula is discretized into a u-p form, where u is the soil particle displacement, and p is the pore water pressure. The u-p form can consider the interaction between the pore water pressure and the soil skeleton. Saturated sand adopted a multi-yield surface plastic constitutive model that was sensitive to changes in pore water pressure, corresponding to the PressureDepend MultiYield02 (PDMY02) constitutive model in OpenSees. This model can better simulate the dilatancy and flow of sand under cyclic loading., Reproducible shear deformation accumulation and liquefaction characteristics during dynamic process. The clay adopted a multi-yield surface plastic constitutive model that was insensitive to changes in pore water pressure, that is, changes in pore water pressure have little effect on the shear properties of the soil. This model corresponds to the PressureInpendentMultiYield (PIMY) constitutive model in OpenSees material. The parameters needed for the constitutive soil model as presented in Table 1 were selected based on the measured parameters from the shake table experiment.

![Figure 3. Soil surface condition before and after the shaking](image)

![Figure 4. FE model discretization of shaking table test.](image)

| Parameter                                 | Clay layer | Loose sand layer | Dense sand layer |
|-------------------------------------------|------------|------------------|------------------|
| Mass density \( \rho/\text{t/m}^3 \)       | 1.5        | 1.7              | 1.9              |
| Reference low-strain shear modulus \( G/\text{kPa} \) | 20000      | 60000            | 90000            |
| Bulk modulus \( B/\text{kPa} \)            | 50000      | 160000           | 190000           |
| Maximum octahedral shear strain \( \gamma_{\text{max}} \) | 0.1        | 0.1              | 0.1              |
| Reference effective confining pressure \( p/\text{kPa} \) | 100        | 101              | 101              |
| Pressure dependency coefficient \( n \)    | 0          | 0.5              | 0.5              |
| Number of yield surface                   | 20         | 20               | 20               |
The ground surface was set to free drainage boundary condition, the bottom and sides of the model are set to undrained. Soil nodes on both sides of the same plane are salved together to form the shear condition of laminar boxes in shaking table tests. The finite element analysis process consists of the following steps:

Step 1: a free-field saturated soil model of a shaking table was established, and a quadrilateral plane strain element was used to mesh the saturated soil with finite elements. Set the permeability coefficient of the saturated soil layer to 1 for fast consolidation of the soil. Elastic and plastic gravity analyses were performed separately to generate initial stress fields.

Step 2: set the soil deformation at the stage of gravity analysis to 0, and update the permeability coefficient of the saturated soil to the set value. Earthquake loading was applied to the nodes at the base of the soil and container meshes using the UniformExcitation command in OpenSees. Solved using the Newmark integration method. An assumed 3% Rayleigh damping for the soil was used to ensure the stability of the calculation at low strain level.

4. Numerical results analysis and model reliability verification
The validity of the numerical model was verified by comparing the shaking table test results of the free field seismic response of the liquefaction site with the numerical results of the model, focusing on the acceleration response of the soil and the pore pressure development mechanism.

Figure 5 displays the experimental and computed time histories of accelerations at typical measurement points of different depth. It can be observed that the experimental values and the calculated values show a basically consistent change law. In the experimental results, the acceleration time history and peak value of the measurement point SAA2-0 of the dense sand layer was closest to the base input, indicating that there was no relative sliding phenomenon between the dense sand layer and the bottom of the shear laminar box. From the measured acceleration peaks of SAA2-0 and SAA2-2 in the experiment, it can be seen that compared with the dense sand layer at the bottom, the acceleration peak in the loose sand layer has a significant attenuation effect, and the numerical calculation can better simulate this peak attenuation of acceleration in loose sand.

The computed and experimental time histories of excess pore water pressures at different depths are shown in Figure 6. As seen from Figure 6, the excess pore water pressure accumulation was rapid. The results of the numerical model reproduced the phenomenon of the increase of excess pore water pressure during the seismic load input process. The experimental and numerical results of the excess pore water pressure time history response trends were basically consistent, and the time of the peak appearance was basically consistent. Compared with the numerical solution, the excess pore water pressure of the experimental solution decreased slightly after reaching the first peak value. The pore water pressure time history response showed a "double hump" growth and dissipation characteristic. The numerical solution of the excess pore water pressure at the W9 measurement point was larger than the experimental solution. However, the numerical and experimental solutions after 45s were more consistent. This difference in results may be due to the uncertainty of the numerical model parameters and the characteristics of the experiment itself. For example, the constitutive model parameters and the soil boundary conditions during the experiment. Water leaked from the surface of the clay layer where the sensor cable was placed, which changed the drainage conditions of the soil.
Figure 5. Numerical and experimental acceleration time history along soil depth

Figure 6. Experimental and numerical excess pore water pressure time history

The distribution of excess pore water pressure in saturated sand layers at the critical moment of seismic load input is shown in Figure 7. It can be seen that the distribution of excess pore water pressure along the depth gradually increases. As the amplitude of input ground motion decreases, the value of the super pore pressure in the bottom dense sand layer and the loose sand layer gradually decreases, but at the interface between the loose sand layer and the clay layer, remains almost unchanged.

Figure 7. Distribution of excess pore water pressure in the saturated sand at key time points

5. Conclusion

Based on the completed large-scale shaking table test of the liquefied site and the open source finite element program OpenSees calculation platform, the interaction between the pore water pressure of the saturated soil and the soil skeleton are considered comprehensively. A numerical model of the saturated liquefied free field was established. The dynamic response analysis was performed and the following conclusions were obtained.
1) The established numerical calculation model can better simulate the seismic dynamic response of liquefied free fields, and realistically reproduce the complex mechanical characteristics of liquefied soils, which provides a certain reference for similar free field simulations.

2) After the soil was liquefied, in the middle of the liquefied soil, the acceleration peak appears a more obvious attenuation effect. Dissipation of soil excess pore water pressure in liquefied site begin at the bottom of the soil.

References
[1] Vessia G, Venisti N. Liquefaction damage potential for seismic hazard evaluation in urbanized areas [J]. Soil Dynamic and Earthquake Engineering, 2013, 31 (8): 1094－1105.
[2] VARGHESE R, LATHA G. Shaking Table Studies on the Conditions of Sand Liquefaction[C]. Geo-Congress 2014 Technical Papers, 2014(234):1244－1253.
[3] Dobry R, Thevanayagam S, Medina C, Bethapudi R, et al. Mechanics of lateral spreading observed in a full-scale shake test. Journal of Geotechnical and Geoenvironmental Engineering 2011;137(2):115－29.
[4] Y. Zhou, Z. Sun, Y. Chen, Zhejiang University benchmark centrifuge test for LEAP-GWU-2015 and liquefaction responses of a sloping ground, Soil Dyn. Earthq. Eng. 113 (2018) 698－713.
[5] Chang D, Boulanger R, Brandenberg S, et al. FEM analysis of dynamic soil-pile structure interaction in liquefied and lateral spreading ground [J]. Earthquake Spectra, 2013,29(3):733-755.