3D Dynamic Numerical Modeling for Soil-Pile-Structure Interaction in Centrifuge Tests

Modélisation numérique dynamique en 3D de l'interaction sol-pieu en centrifugeuse

Kwon S.-Y., Kim M.-M.
Seoul National University, Seoul, Republic of Korea

Kim S.g-H.
Ministry of Land, Transfer and Maritime Affairs, Seoul, Republic of Korea

Choi J.-I
University of California, Los Angeles, Los Angeles, CA, USA

ABSTRACT: 3D dynamic analysis based on the finite difference method was performed, to simulate the dynamic behavior of a soil-pile-structure system under seismic loading, which was observed from dynamic centrifuge tests. For the centrifuge tests, model piles were placed in a so-called equivalent shear beam box. The acceleration time histories of 12 sine waves and 10 scaled earthquake events were used as input motions. The 3D numerical modeling was formulated in a time domain, to effectively simulate the nonlinear behavior of soil. As a modeling methodology, the soil medium was divided into near field and far field, the latter of which was not affected by soil-pile-structure interaction. The mesh was created only for the near field, to reduce the computing time. Far field response was applied as a boundary condition at the boundary of the near field. Soil nonlinearity was considered by adopting a hysteretic damping model and an interface model, which can simulate separation and slip between the soil and pile. The 3D modeling method was calibrated, by comparing the numerical modeling result with that of the dynamic centrifuge test. Finally, the 3D modeling method established in this research was evaluated, by comparing the numerical modeling results with those of other centrifuge tests.

RÉSUMÉ : Une analyse 3D dynamique, fondée sur la méthode des différences finies, a été réalisée afin de simuler le comportement dynamique du système sol-pieu-structure sous chargement sismique qui a été observé d'essais dynamiques en centrifugeuse. Pour les essais en centrifugeuse, le pieu modèle était placé dans une « boîte de cisaillement pour poutre ». Les accélérogrammes de 12 vagues sinusoïdales et de 10 tremblements de terre ont été utilisés comme données de calcul. La modélisation numérique 3D a été formulée dans un domaine temporel pour simuler efficacement le comportement non linéaire du sol. Comme méthode de modélisation, le sol a été divisé en champ proche et en champ lointain, ce dernier n'étant pas affecté par l'interaction sol-pieu-structure. Le maillage a été créé seulement pour le champ proche afin de réduire le temps de calcul. La réponse du champ lointain a été appliquée comme condition à la limite du champ proche. La non-linéarité du sol a été considérée en adoptant un modèle d'amortissement hystérétique et un modèle d'interface qui peut simuler la séparation et le glissement entre le sol et le pieu. La méthode de modélisation 3D a été calibrée en comparant les résultats de la modélisation numérique avec les essais dynamiques en centrifugeuse. Enfin, la méthode de modélisation 3D établie dans cette recherche a été évaluée en comparant les résultats de la modélisation numérique avec les résultats d'autres essais en centrifugeuse.

KEYWORDS: Centrifuge tests, Numerical analysis, Finite difference method, Dynamic soil-pile interaction

1 INTRODUCTION

Prediction of the behavior of pile foundations under strong earthquake loading is very important. Recently, the design procedure for evaluating pile behavior under strong earthquake loading has been modified, particularly after a series of mega-earthquakes, such as the Great East Japan (3/11) Earthquake. However, dynamic analysis of the soil-pile system is a very complicated procedure, different from the static case, and is affected by many factors, such as soil nonlinearity and dynamic soil-pile interaction.

Many researchers have investigated the soil-pile-structure interaction (SPSI) effect on pile foundations (e.g. Kaynia and Kausel 1982, Dobry and Gazetas 1988, Markis and Gazetas 1992, Klar and Frydman 2002, Martin and Chen 2005) in the frequency domain. However, analysis in the frequency domain is not straightforward, and requires the Fourier transformation for application. In addition, it is difficult to consider the nonlinearity of soil in this analysis. Therefore, seismic analysis in the time domain is an effective procedure to consider the nonlinearity that occurs during strong earthquake motion. 3D continuum modeling of the soil-pile system is one of the most accurate techniques among dynamic analysis methods, although this is difficult to apply, due to its complexity and lengthy analysis time.

In this study, three-dimensional continuum modeling of the soil-pile system using the FDM (Finite Difference Method) program, FLAC-3D, was performed. The seismic responses of soil-pile-structure observed in centrifuge tests were compared, to calibrate and validate the applied 3D nonlinear FDM analysis. The analysis mainly focused on internal responses of the pile, such as peak bending moment of the pile foundation.

2 CENTRIFUGE TESTS

Dynamic centrifuge model tests(Yoo et al. 2012) were performed at a condition of 40g centrifugal acceleration, using the KOCED centrifuge at KAIST (the Korea Advanced Institute of Science and Technology). The tests evaluate the dynamic behavior of piles embedded in a model soil, which consists of a dry-dense sand layer. A typical test layout is presented in Fig. 1. The model soil consists of Jumunjin sand, with a relative density of 80%. The model container used for the centrifuge tests reported in this paper is the ESB (equivalent shear beam) box. The internal dimension of the ESB box is 50cm x 50cm x 65cm. Model piles with a concentrated mass of 1.4kg were made with aluminum pipes, and fixed at the bottom of the ESB box, in order to simulate a rock-socketed pile. Strain gauges were attached on both sides of the pile according to depth, in order to measure the bending moment of the pile during vibration. The maximum bending moment of the pile was calculated, using the following equation.

$$M_{\max} = \frac{(\sigma_L - \sigma_R)}{y} I \tag{1}$$

where, σ_L and σ_R are the normal stresses at the left and right outermost pile mesh integration points, respectively; y is the distance between the integration points and the central axis; and, I is the the moment of inertia of the pile.

Table 1 shows the test programs. All values are given in prototype dimensions, which are converted according to scaling laws for centrifuge testing (Taylor 1995, Iai et al. 2005) As shown in Table 1, tests were performed under various input frequency and acceleration conditions, using 12 sine waves and 10 seismic waves. Three different pile diameters were used for the tests. Calibration was performed after the numerical modeling, which was carried out for the centrifuge test using the pile with the largest diameter of 2.5cm and thickness of 0.1cm. The same procedure was repeated for a pile with a diameter of 1.8cm and thickness of 0.1cm, and the suggested method of numerical modeling was validated by comparing the results.

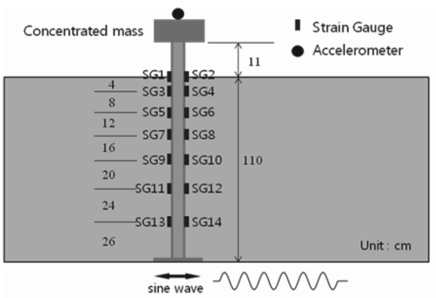


Fig. 1 Layout and Instrumentation

Table 1. Test program

Case	Input motion	Base input frequency(Hz)	Amplitude of base input(g)
(a) Sinusoidal wave			
a1 a2 a3	- - -	1 2 3	0.05, 0.13, 0.25, 0.45
(b) Real earthquake			
b1 b2	Ofunato Nisqually	-	0.06, 0.13, 0.25, 0.36, 0.51

3 3D FINITE DIFFERENCE ANALYSIS

3.1 *Soil model*

The soil model adopted in this study mainly consists of two parts, which are the constitutive model and the damping model. During a strong earthquake, soils show nonlinear plastic behavior, while large displacements take place. In order to simulate this type of behavior, the Mohr-Coulomb elastoplastic model was used as the constitutive model. The nonlinear behavior of soils was simulated by applying a hysteretic damping model. As shown in Eq. (2), in the hysteretic damping model, the tangential shear modulus is represented as a function of shear strain (Itasca Consulting Group 2006). L1 and L2 of Eq. (2) represent the decrease rate and decrease starting point of G/G_{max} of the G/G_{max}-γ curve, respectively. In this study, the G/G_{max}-γ curve of Jumoonjin sand is obtained by triaxial compression tests and resonant column tests. L1 and L2 were determined as 0.5 and 3.65, respectively, by parametric analysis, and used to calibrate the G/G_{max}-γ curve of the numerical model (Fig. 2)

$$M_t = s^2(3-2s) - \frac{6s(1-s)}{L_2 - L_1}\log_{10} e \tag{2}$$

Where M_t is the tangential shear modulus, $s = \frac{L_2 - L}{L_2 - L_1}$, $L=\log_{10}\gamma$, L_1&L_2 = Coefficient; and γ = is shear strain,

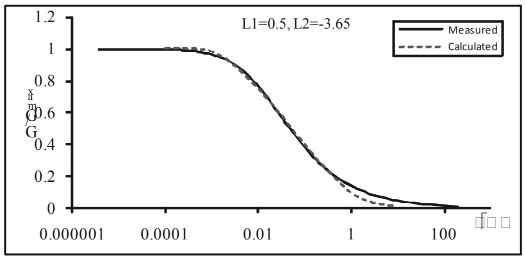


Fig. 2 Calibration of G/G_{max} (Jumoonjin Sand)

The maximum shear modulus of soils depends on the confining stresses according to depth (Hardin et al. 1978), and for this study, the calculation of the maximum shear modulus of soils was made using Eq. (3). The coefficients A and n were determined by prior test results (Yang 2009).

$$G_{\max} = AF(e)(OCR)^k P_a^{1-n}(\sigma_m^{'})^n \tag{3}$$

where, $F(e) = \frac{1}{0.3+0.7e^2}$, e is the void ratio, $\sigma_m^{'}$ is the average principal stress, P_a is the atmosphere pressure, and coefficients A and n are 247.73 and 0.567, respectively.

3.2 *Interface model*

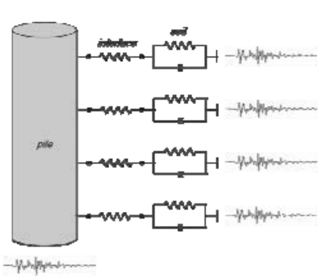
The interface between pile foundation and surrounding soil undergoes slippage and separation during strong earthquake motions. The interface model should consider this kind of phenomena accordingly. For this study, an interface model that can simulate the soil-pile separation, of slippage in the normal and shear directions, was adopted. The applied interface model uses normal and shear stiffness of the interface, in order to estimate the spring constant, and the constant is represented as Eq. (4) (Itasca Consulting Group 2006). The shear modulus used in Eq. (4) considers the nonlinear behavior of soil, as represented in 3.1. Therefore, the nonlinear behavior is also considered in the interface model. Parametric study on the stiffness showed that the numerical modeling results and test results were most similar when the shear stiffness and normal stiffness were identical in value; therefore identical values were applied ($k_n=k_s=1.51\times10^{10}$N/m):

$$k_n = \max\left[\frac{K+(4/3)G}{\Delta z_{\min}}\right] \tag{4}$$

where K and G are the bulk and shear modulus of the soil zone, respectively; and Δz_{min} is the smallest dimension of an adjoining zone in the normal direction.

3.3 *Boundary condition*

The most important aspect of boundary conditions for numerical modeling of pile foundations is the simulation of semi-infinite boundary conditions. When the proper boundary conditions are not applied, the input motion generates a reflected wave, resulting in an inaccurate simulation of actual motion. In addition, if modeling is done with an endless number of meshes, analysis time greatly increases, resulting in difficulty for numerical analysis for various conditions, and decrease of analysis efficiency. Therefore, in order to overcome the problems stated above, simplified continuum modeling was adopted (Kim et al. 2012). Fig. 3 shows a layout of the simplified continuum model. Fig. 4 shows the mesh that is used in this study.



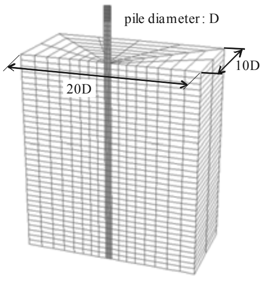


Fig. 3 Simplified continuum modeling

Fig. 4 Mesh of the simplified continuum model

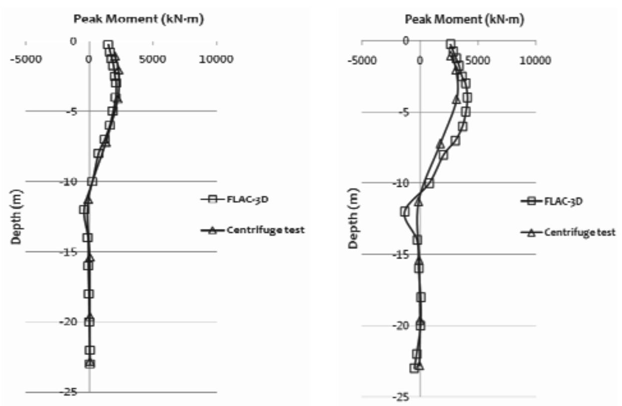
4 CALIBRATION AND VERIFICATION OF THE NUMERICAL MODEL

4.1 Calibration of the numerical model

Calibration of the proposed modeling method was performed by comparing results between dynamic centrifuge tests and 3D numerical simulation. The case of a pile with a diameter of 100cm and wall thickness of 4cm at the prototype scale was used in this procedure.

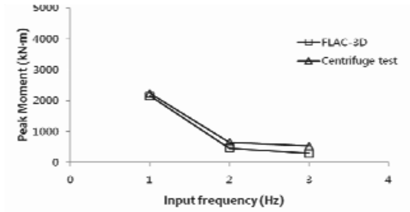
4.1.1 Response to sinusoidal waves

The peak bending moments along the depth, obtained by experiment and numerical analysis, are shown in Fig. 5. As the base input acceleration increases, the pile response increases, both in centrifuge test and FLAC-3D. It is observed that there is good agreement between the computed and measured values. The peak bending moment profile obtained from numerical simulation well predicts the location at which maximum bending moment occurs, and the infitie depth of the pile. As the input acceleration increases, the discrepancy between computed and measured values increases slightly. The average discrepancy through various input motions was 10%. In particular, when the input acceleration was 0.13g, the peak bending moment profiles obtained by experiment and numerical analysis were almost identical along the depth, as shown in Fig. 5 (a). The peak bending moment

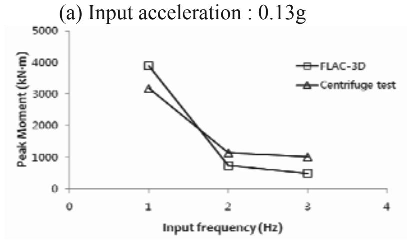


(a) Input acceleration : 0.13g (b) Input acceleration : 0.25g

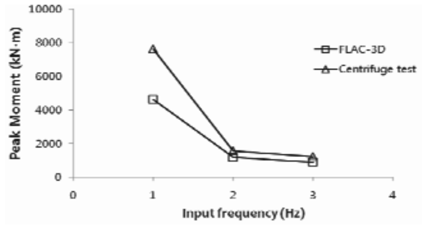
Fig. 5 Peak bending moment along the pile (1Hz)



(a) Input acceleration : 0.13g



(b) Input acceleration : 0.25g



(c) Input acceleration : 0.45g

Fig. 6 Peak bending moment according to frequency

profiles when the input accleration was 2Hz and 3Hz were similar to the profile at 1Hz.

The peak moment values varied considerably with the base input frequency, as shown in Fig. 6. The amplitude of response became highest for an input frequency of 1Hz. This resonance occurs both in the centrifuge test and FLAC-3D, which means that the proposed modeling method is capable of simulating the important dynamic characteristic. As input acceleration increases, the discrepancy between the measured value and computed value increases at 1Hz, due to resonance. In most of the input motions, the peak bending moment value obtained from numerical analysis agrees well with that from the centrifuge test.

Fig. 7 shows a comparison between the peak bending moments measured in the centrifuge test, and those calculated in FLAC-3D analysis. All the points are located near the 1:1 line, meaning that the modeling method proposed in this study has the ability to simulate pile behavior reasonably well.

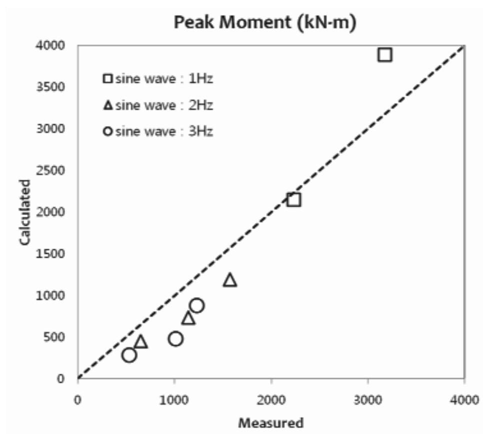
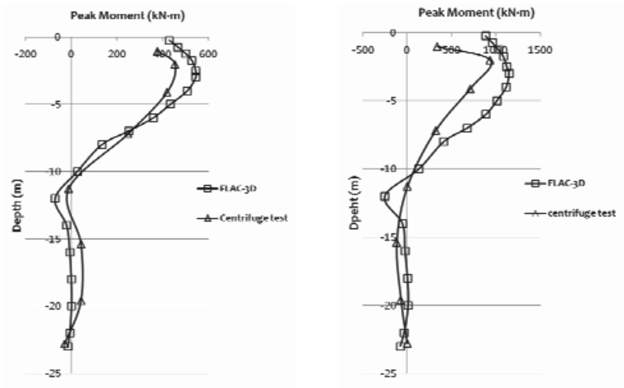


Fig. 7 Comparison between the test and FLAC results(sine wave)



(a) Input acceleration : 0.13g (b) Input acceleration : 0.25g

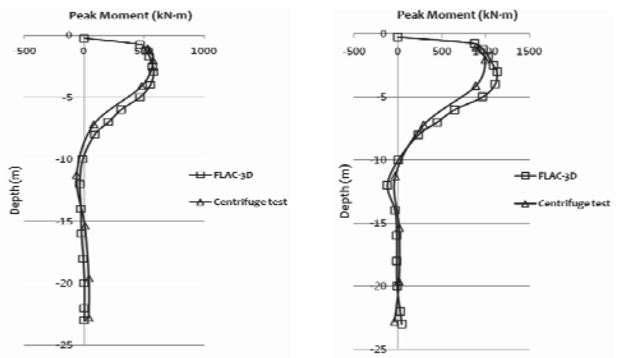
Fig. 8 Peak bending moment along the pile (Nisqually)

4.1.2 Response to a real earthquake event

The analysis for real earthquake events, which is meaningful in practice, was performed, to validate the modeling method proposed in this study. In this stage, the acceleration record of the Nisqually earthquake (2001) was used for the input motion. The peak moment profiles along the pile, as obtained by centrifuge test and numerical simulation, are shown in Fig. 8. As the base input acceleration increases, the response increases, both in the centrifuge test and FLAC-3D. These results are consistent with the results for the sinusoidal wave. Also, it can be seen that the results obtained from the numerical model agree reasonably well with the values recorded during the centrifuge test; and the average discrepancy was within 15%.

4.2 Verification of the numerical model

Another case of the centrifuge test with a pile diameter of 72cm



(a) Input acceleration : 0.13g (b) Input acceleration : 0.25g

Fig. 9 Peak bending moment along the pile (1Hz)

and pile wall thickness of 4cm at the prototype scale, is modeled by the proposed modeling method, and a comparison between the measured value and computed value was performed, to verify the applicability of the proposed modeling method. The peak moment profiles along the pile, obtained by centrifuge test and numerical simulation, are shown in Fig. 9. The peak bending moment profile obtained from numerical simulation well predicts the centrifuge test, and the discrepancy between the measured and computed values is within 10%. Therefore, it can be concluded that the numerical model proposed in this study has the ability to simulate the dynamic behavior of a soil-pile system under various input conditions. Also, it will be able to be applied to the practical seismic design of pile foundations for various conditions.

5. CONCLUSION

3D numerical analysis is proposed to simulate the dynamic behavior of a soil-pile system observed in dynamic centrifuge tests(Yoo et al. 2012). Calibration and verification were then carried out, to validate the applicability of the proposed modeling methodology.

(1) Calibration of the proposed model is performed for the case of a pile with a diameter of 100cm and wall thickness of 4cm. Both in the centrifuge test and numerical model, resonance occurs at the same frequency (1Hz), and peak bending moments obtained from the proposed model agree well with those from the centrifuge test.

(2) Applicability of the proposed model is verified for the case of a pile with a diameter of 72cm, and wall thickness of 4cm. The peak bending moment profile obtained from numerical simulation well predicts the centrifuge test, and the discrepancy between the measured and computed values is within 10%. Therefore, it can be concluded that the numerical model proposed in this study has the ability to simulate the dynamic behavior of a soil-pile system under various input conditions. Also, it will be able to be applied to the practical seismic design of pile foundations for various conditions.

6. ACKNOWLEDGEMENTS

This work was supported by the SNU SIR Group of the BK21 Research Program funded by the Ministry of Education, Science and Technology.

7. REFERENCES

Boulanger, R.W. and Curras, C.J. 1999. Seismic Soil-Pile-Structure Interaction Experiments and Analyses. Journal of Geotechnical and Geoenvironmental Engineering, Vol.125, No. 9, pp. 750-759

Dobry, R. and Gazetas, G. 1988. Simple method for dynamic stiffness and damping of floating pile groups. Geotechnique,38, 557-574.

Hardin, B. O. 1978. The nature of stress-strain behaviour of soils. Earthquake engineering and soil dynamics, 1, 3-90.

Iai, S., Tobita, T. and Nakahara, T. 2005. Generalised scaling relations for dynamic centrifuge test. Geotechnique,55(5), 355-362.

Itasca Consulting Group, Inc. 2002. FLAC3D(Fast Lagrangian Analysis of Continua in 3Dimensions) User's Guide, Minnesota, USA.

Kaynia, A. M. and Kausel, E. 1982. Dynamic behavior of pile groups. 2nd International Conference On Numerical Methods in Offshore Piling, Texas University, Austin, 509-532.

Kim S H, Kwon S Y, Kim M M, Han J T. 2012. 3D Numerical Simulation of a Soil-Pile System Under Dynamic Loading. Marine Geosources & Geotechnology. 30(4), 347-361.

Klar, A. and Frydman, S. 2002. Three-Dimensional Analysis of Lateral Pile Response using Two-Dimensional Explicit Numerical Scheme. Journal of Geotechnical and Geoenvironmental Engineering, 128 (9), 775-784

Makris, N. and Gazetas, G. 1992. Dynamic pile-soil-pile interaction. Part II: Lateral and seismic response. Earthquake Engineering and Structural Dynamics, 21, 145-162.

Martin, G.R. and Chen, C. Y. 2005. Response of piles due to lateral slope movement. Computers and Structures, Vol. 83, pp. 588-598

Taylor, R.N. 1995. Centrifuge in modelling: principles and scale effects. GeotechnicalCentrifugeTechnology, 19-33

Yang E K. (2009). Evaluation of dynamic p-y curves for a pile in sand from 1g shaking table tests. Ph.D. Dissertation, Seoul National University.

Yoo M T, Han J T, Choi J I, Kim M M. 2012. Comparison of Lateral Pile Behavior under Static and Dynamic Loading by Centrifuge Tests. GeoCongress 2012. Oakland, CA, USA. 2048-2057