Geomechanics and Engineering, *Vol. 12, No. 2 (2017) 239-255*
DOI: https://doi.org/10.12989/gae.2017.12.2.239

Nonlinear numerical analyses of a pile-soil system under sinusoidal bedrock loadings verifying centrifuge model test results

Yong-Seok Kim [1a] and Jung-In Choi [*2]

[1] *Department of Architectural Engineering, Mokpo National University, 58554, Republic of Korea*
[2] *Pangyo R&D Center, Samsung Heavy Industry, Pangyo, Bundang, Seongnam, Gyeonggi, 13486, Republic of Korea*

(*Received November 05, 2015, Revised October 25, 2016, Accepted October 28, 2016*)

Abstract. Various centrifuge model tests on the pile foundations were performed to investigate fundamental characteristics of a pile-soil-foundation system recently, but it is hard to find numerical analysis results of a pile foundation system considering the nonlinear behavior of soil layers due to the dynamic excitations. Numerical analyses for a pile-soil system were carried out to verify the experimental results of centrifuge model tests. Centrifuge model tests were performed at the laboratory applying 1.5 Hz sinusoidal base input motions, and nonlinear numerical analyses were performed utilizing a finite element program of P3DASS in the frequency domain and applying the same input motions with the intensities of 0.05 g~0.38 g. Nonlinear soil properties of soil elements were defined by Ramberg-Osgood soil model for the nonlinear dynamic analyses. Nonlinear numerical analyses with the P3DASS program were helpful to predict the trend of experimental responses of a centrifuge model efficiently, even though there were some difficulties in processing analytical results and to find out unintended deficits in measured experimental data. Also nonlinear soil properties of elements in the system can be estimated adequately using an analytical program to compare them with experimental results.

Keywords: complex centrifuge model test; pile foundation; P3DASS program; sinusoidal base input motion; nonlinear numerical analysis; Ramberg-Osgood soil model

1. Introduction

Experimental tests and analytical analyses for a single pile or a pile group including mockup tests, small and full scale field tests and centrifuge tests were performed during more than three decades. Recently various centrifuge model tests on the pile foundations were carried out to investigate the fundamental characteristics of a pile-soil-foundation system and it is easy to find experimental results, but it is hard to find the numerical analysis results of a pile foundation system considering the nonlinear behavior of soil layers due to the dynamic excitations.

Dynamic tests for the scaling relation to utilize the centrifuge model test kits, the responses analysis of a pile foundation etc. are carried out by many researchers for several decades.

∗Corresponding author, Ph.D., E-mail: jungin.choi@samsung.com
[a] Professor, Ph.D., P.E., E-mail: yskim@mokpo.ac.kr

http://www.techno-press.org/?journal=gae&subpage=7 ISSN: 2005-307X (Print), 2092-6219 (Online)

Minowa *et al*. (2001) executed centrifuge model tests to reproduce the dynamic behavior of a pile foundation structures in a NIED 1g large-scale test, and confirmed the composition similitude requirements and the effectiveness. Kim *et al*. (2006) performed experimental tests to verify the similitude laws through 1-g and centrifuge model tests.

Kim *et al*. (1987) compared the analytical solutions with the experimental test results of three small scale piles subject to vertical vibration excited with very small amplitude at very high frequencies. Giannnako *et al*. (2012) studied the vertical response of a pile numerically with three cyclic lateral loadings to investigate the response of the pile-soil system by using a finite element modeling, and compared it with the centrifuge test results. They found soil densification and system densification mechanisms, and also found the hardening mechanism of a system densification dominates upon soil densification in cyclic loading. Danno and Kimura (2009) evaluated long-term behavior of a pile foundation using coupled FEM, and conducted a centrifuge model test to verify the validity of the numerical analyses. And Yoo et al. (2012) studied pile group effects by comparing the centrifuge model test results of single pile and 3x3 pile group foundations.

Zhussupbekov *et al*. (2013) performed centrifuge model tests of piles to predict the in-situ performance, and observed that the pile behavior in the centrifuge is similar to that of the in-site piles. Yoo *et al*. (2014) also performed centrifuge model tests of a foundation-soil system with an irregular pile group to evaluate the seismic loadings at the foundation surface by comparing the seismic loadings at the free field surface. Choi and Brandenberg (2015) proposed a cyclic p-y plasticity model for a single pile in sand using the experimental response of a single pile achieved with a dynamic centrifuge model test.

However, no other nonlinear analytical study result of a single pile for the horizontal excitations could be found from the journal databases, and it was also hard to find any centrifuge model test result for a small scale pile excited with dynamic base motions.

In this study, nonlinear numerical analyses for a pile-soil system excited with sinusoidal dynamic bedrock loadings were carried out utilizing a pseudo 3-dimensional finite element program of P3DASS. Analytical results were also compared with experimental ones achieved from centrifuge model tests of a single pile-soil system which were performed at the KOCED (Korea Construction Engineering Development) Geo-centrifuge Testing Center in the Korea Advanced Institute of Science and Technology (KAIST).

2. Experimental test results

Centrifuge model tests were performed for a 1/40 scaled pile-soil system shown in Fig. 1 utilizing a centrifuge model testing system (Fig. 2) installed at the KOCED Geo-centrifuge Testing Center in KAIST (Kim *et al*. 2013, Lee *et al*. 2013).

Dynamic Centrifuge Model Testing system can test a maximum capacity of 700 kg (6.86 kN) by accelerating up to 20 g (g is gravity) with a frequency range of 40-300 Hz in the condition of applied centrifuge force of 100g. However, these tests were performed in the centrifuge acceleration of 40 g with 1.5 Hz sinusoidal base input motions. Model soil box of ESB (Equivalent Shear Beam) has a size of 49cm × 49cm × 63cm, and is made to deform depending on the deformation of a soil layer at each layer to reduce the boundary effect of reflecting waves by connecting box wall layers with rubber buckles.

The model of a pile-soil system has a concentrated mass of 627.5 kN on the steel pile head elevated a 5.2 m from the ground surface. The prototype pile embedded 22.8m from the ground

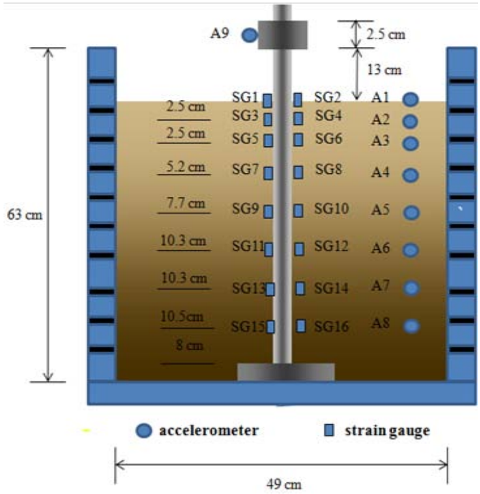
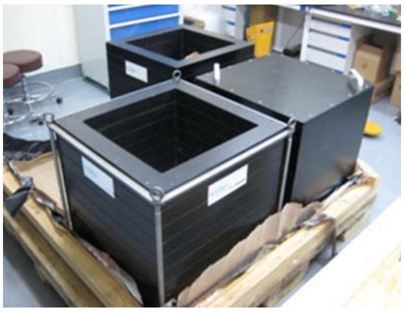


Fig. 1 1/40 Centrifuge test model of pile-soil system

(a) Centrifuge model testing system



(b) ESB box

Fig. 2 Centrifuge model testing system and ESB box

Table 1 Properties of a pile

	Prototype	40 gc model
Scale	1/1	1/40
Centrifugal acc. scale	-	1/40
Diameter of pile (m)	1	0.025
Thickness of pile (m)	-	0.001
Material	Steel	Aluminum
Flexural rigidity (EI) (kN×m^2)	9.44E+5	368.7
Concentrated mass on pile head (kN)	627.5	0.0098
Soil depth (m)	22.8	0.57

surface has a diameter of 1 m and a flexural rigidity (EI) of 944,000 kN-m^2. The properties of prototype and scaled aluminum pile are given in Table 1.

Also, engineering soil properties of typical Jumoonjin sand used for the centrifuge model test are shown in Table 2. Typical Jumoonjin sand has a unit weight of 13.3-16.6 kN/m^3 and a relative density of 80%. Resonance column test results for the Jumoomjin sand are plotted for two different confining mean effective stresses of 100 and 200 kPa in Fig. 3, and are reproduced using the Darendeli soil model (2001) defined by Eq. (1) as follows.

$$\gamma_y = 0.0475\, \sigma_m'^{\,0.239} \left[\frac{G_{\max}}{G} - 1\right]^{\frac{1}{0.895}} \tag{1}$$

In here, γ_y is in % and σ_m' is effective mean soil stress in kPa.

Eight pairs of strain gauges and nine accelerometers were installed to measure the test results of strains and accelerations at the locations including a pile head as shown in Fig. 1. Pile head accelerometer is attached to a concentrated mass which is located at about 5.7 m above the soil surface.

Centrifuge model tests for a pile-soil system were carried out by applying six sinusoidal base input motions at the bottom of a soil box. They have an exciting fundamental frequency of 1.5 Hz and maximum intensities of 0.05 g, 0.1 g, 0.15 g, 0.2 g, 0.3 g and 0.38 g respectively. Base-rock excitation frequency of 1.5 Hz was selected to produce the best responses for various input accelerations through the parametric tests.

Six input base motions were applied from weak motion to strong motion in order to minimize the disturbance of soil specimen during six sequential excitations, and the soil specimen was also prepared at a relative density of 80% as described in Table 2. Little settlement of soil specimen at the surface was measured after 6 experiments justifying the above assumption.

Response time histories of pile head for acceleration and displacement are plotted with six different base input motions of 0.05 g, 0.1 g, 0.15 g, 0.2 g, 0.3 g and 0.38 g in Fig. 4, and maximum

Table 2 Soil properties of Jumoonjin Sand

D_{10} (mm)	D_{50} (mm)	C_u	G_s	$\gamma_{d,\max}$ (kN/m^3)	$\gamma_{d,\min}$ (kN/m^3)	Relative density (%)
0.37	0.60	1.77	2.64	16.6	13.3	80

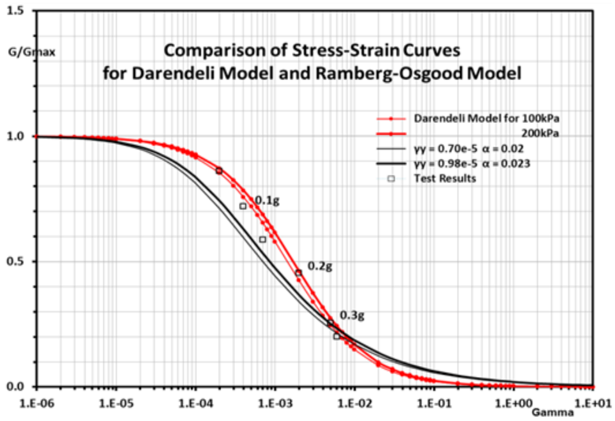
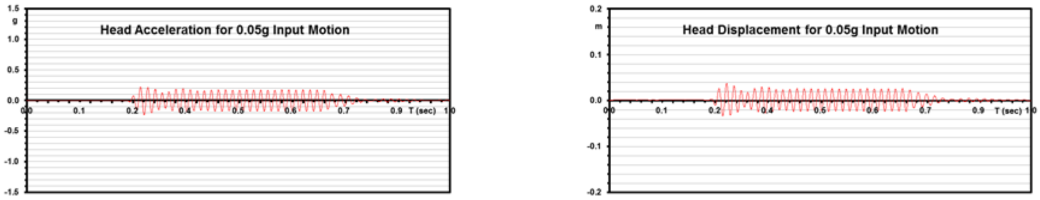
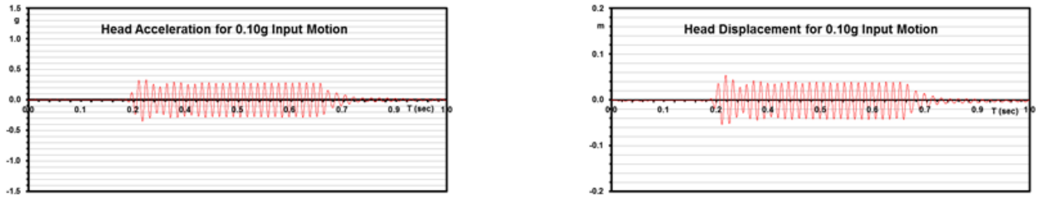


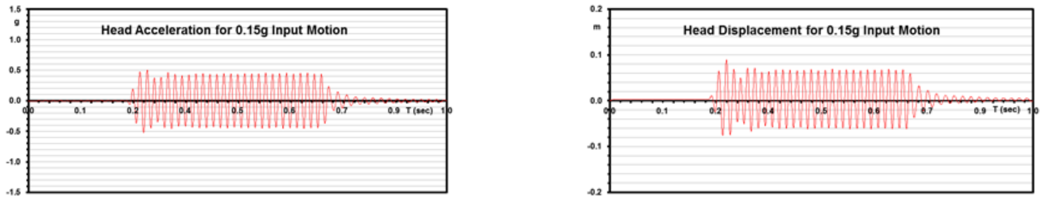
Fig. 3 Normalized shear modulus (G/G_{max}) with respect to shear strain



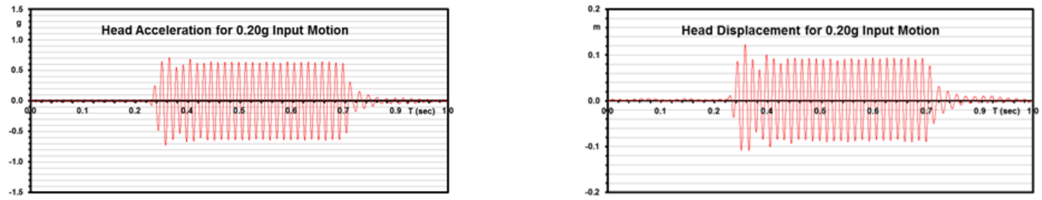
(a) For 0.05 g sinusoidal input motion



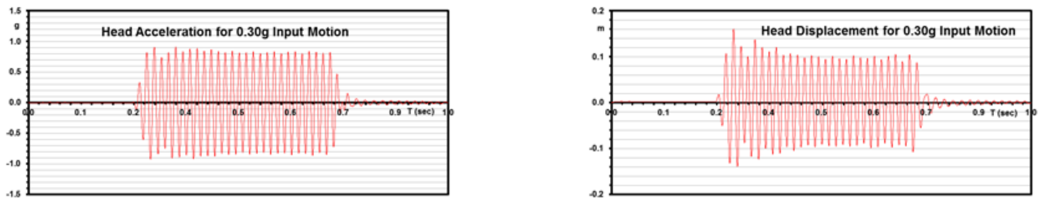
(b) For 0.10 g sinusoidal input motion



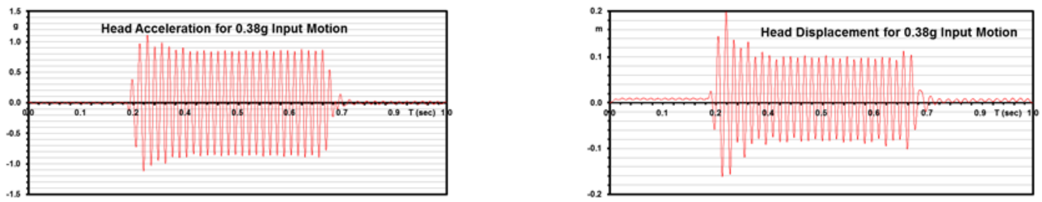
(c) For 0.15 g sinusoidal input motion



(d) For 0.20 g sinusoidal input motion



(e) For 0.30 g sinusoidal input motion

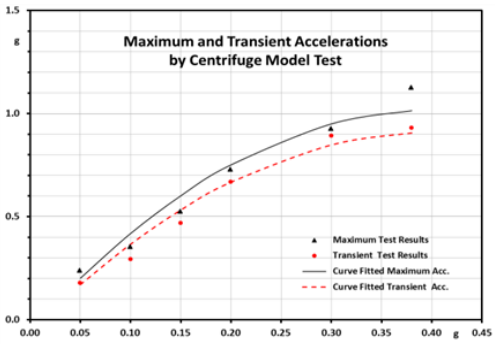


(f) For 0.38 g sinusoidal input motion

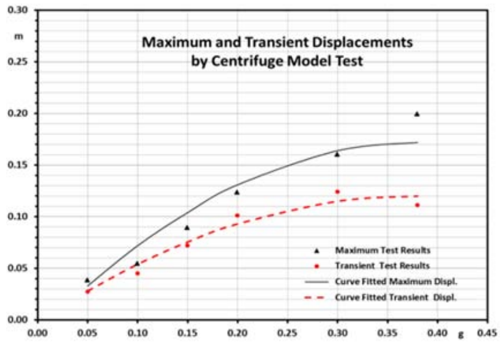
Fig. 4 Response acceleration and displacement of pile head for different input motions

Table 3 Results of centrifuge model test

Intensity of 1.5 Hz sinusoidal input motion	Acceleration (g)		Displacement (m)	
	Maximum	Transient	Maximum	Transient
0.05 g	0.236	0.178	0.038	0.027
0.10 g	0.351	0.294	0.054	0.045
0.15 g	0.523	0.469	0.089	0.072
0.20 g	0.726	0.669	0.123	0.101
0.30 g	0.925	0.917	0.160	0.138
0.38 g	1.124	1.000	0.199	0.111

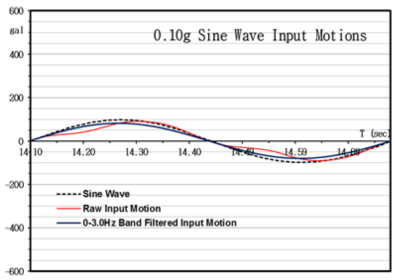


(a) Accelerations of centrifuge model tests

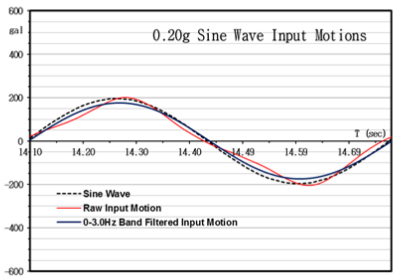


(b) Displacements of centrifuge model tests

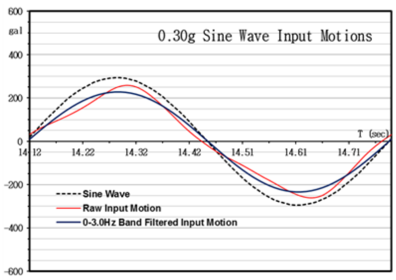
Fig. 5 Centrifuge model test results



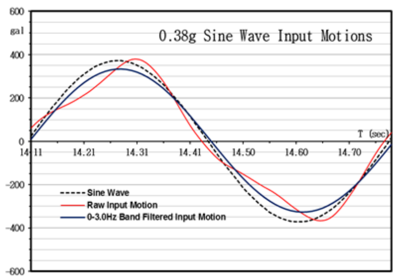
(a) 0.1 g



(b) 0.2 g

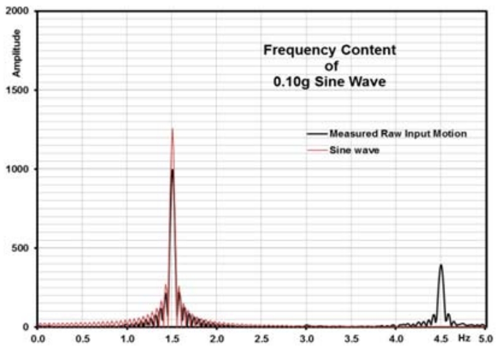


(c) 0.3 g

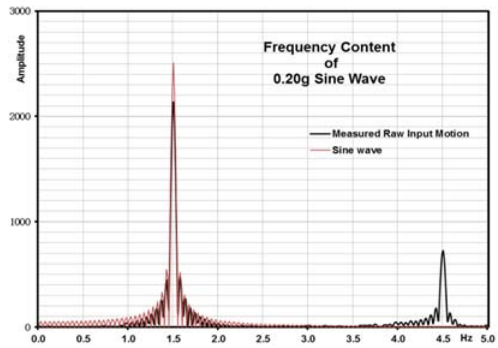


(d) 0.38 g

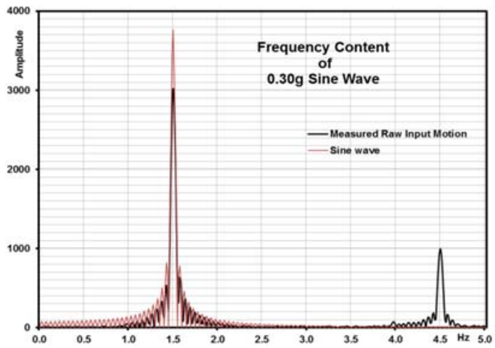
Fig. 6 Comparison of one-cycle sinusoidal input motions



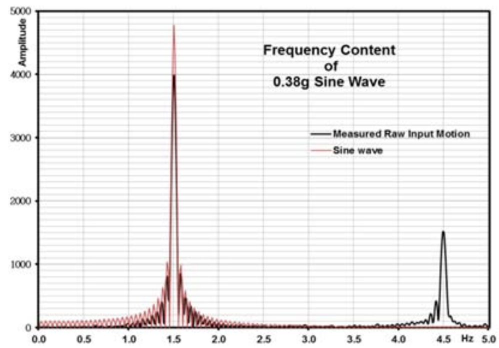
(a) 0.1 g



(b) 0.2 g



(c) 0.3 g



(d) 0.38 g

Fig. 7 Fourier amplitudes of sinusoidal input motions

and transient responses of acceleration and displacement are also given in Table 3. Transient responses in Fig. 4 are steady state motions, and displacement responses were calculated numerically by integrating acceleration responses in the frequency domain.

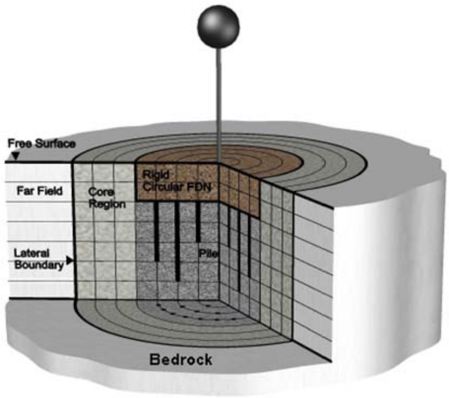
Maximum and transient responses of the pile head are compared for 6 different base input motions of 0.05 g, 0.1 g, 0.15 g, 0.2 g, 0.3 g and 0.38 g in Fig. 5, and they are also curve-fitted to find the general trend of the responses.

Measured raw base input motions used for the centrifuge model test are somewhat distorted as shown in Fig. 6, but shapes of 0-3Hz band filtered input motions are similar to those of normal sine waves. So, it can be concluded that distorted input motions are caused due to the noisy high frequency contents at around 4.5 Hz as shown in Fig. 7.

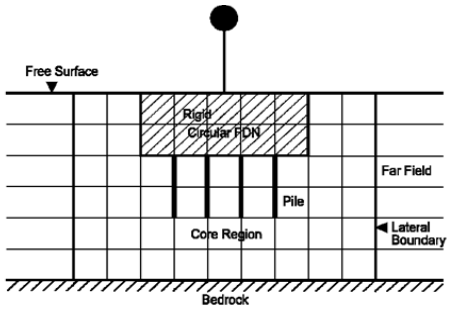
Transient maximum intensity of 0~3 Hz band filtered input motions are approximately 4%, 8%, 20% and 10% smaller than those of normal sinusoidal input motions for different excitations of 0.1 g, 0.2 g, 0.3 g and 0.38 g respectively, and maximum difference occurred with the 0.3 g excitation.

3. Introduction to modeling of P3DASS program

The centrifuge test results of a scaled single pile were verified by the numerical method using a program of Pseudo 3-Dimensional Dynamic Analysis of Structure-soil System (P3DASS). The finite element program of P3DASS was originally developed to perform a pseudo 3-dimensional



(a) 3D Model



(b) 2D Section

Fig. 8 Model of P3DASS and 2D section

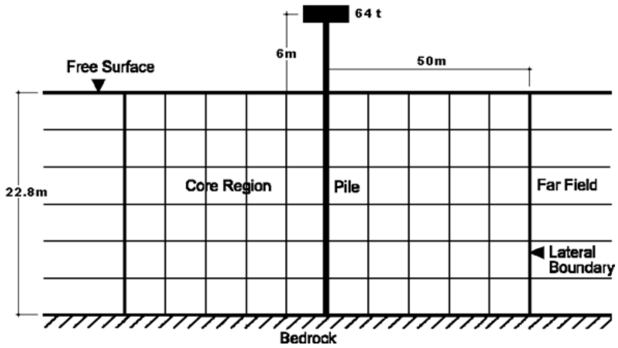


Fig. 9 Single pile-soil model for centrifuge model test

dynamic analysis of a structure-soil system considering vertically propagating waves from the bedrock. P3DASS was coded using a cylindrical coordinate system for the axis symmetric system in the frequency domain as shown Fig. 8. (Roesset and Kim 1987, Kim and Roesset 2004, Kim 2015). This P3DASS program also can be used to analyze the response of a structure built on a pile foundation with a single pile or a pile group taking into account the nonlinear soil properties.

A single pile-soil system used in the centrifugal model test is modeled as shown in Fig. 9 to perform numerical analyses by a program of P3DASS.

A soil layer in the core region is subdivided into toroidal finite-elements considering the horizontal and vertical displacements around the circumference of a cylinder. The far field is represented by the consistent transmitting boundary matrix developed by Kausel (1974) reproducing an extension of the finite-element mesh from the foundation boundary to infinity. The consistent transmitting boundary can be placed at the edge of a circular foundation for the linear elastic analysis according to Kim and Roesset (2004). In this study, the lateral boundary was placed at the distance of 50 m which is far enough to take into account the nonlinearity of soil layers in the far field.

A 22.8 m thick soil layer lying on stiff bedrock was assumed that it is horizontally layered, homogeneous, inelastic, viscous and isotropic. Shear wave velocities and yielding strains for the

Table 4 Soil Properties of Soil Layers

Thickness (m)	Shear wave velocity (m/s)	Yielding shear strain $\times 10^{-5}$	Curve-fitted α for Ramberg-Osgood model
2.0	185	0.50	0.020
2.0	230	0.60	0.020
3.2	250	0.70	0.020
6.0	310	0.83	0.023
6.0	310	0.92	0.023
3.6	320	0.98	0.023

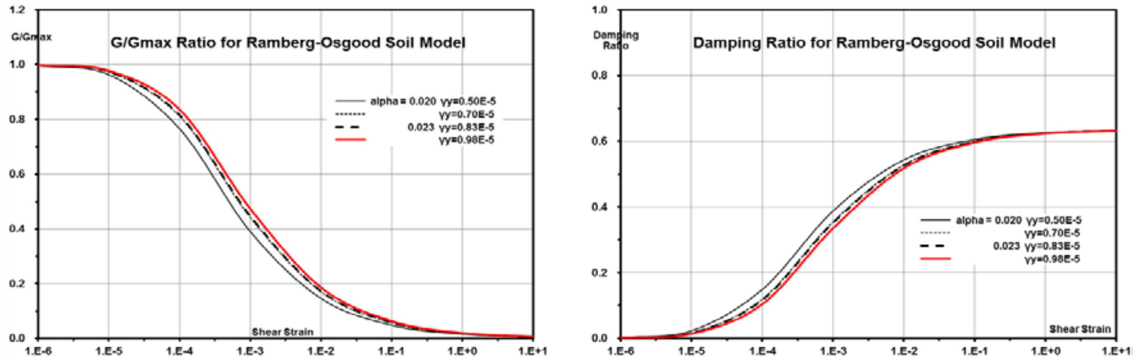
soil layers which were found by curve-fitting the experimental data shown in Fig. 3 to Ramberg-Osgood soil model are given in Table 4. Unit weight of the soil was assumed to be 15.78 kN/m^3, and initial material properties for Poisson's ratio and damping ratio were assumed to be 0.45 and 0.05 respectively for all soil layers.

On the other hand, a 6 m elevated foundation was assumed to be rigid and to have a weight of 627.5 kN (64 t). The property of a steel pile was assumed to be linear with the unit weight of 76.96 kN/m^3 and the Young's modulus of the pile was assumed to be 2.246×10^8 kN/m^2. Equivalent sectional area and moment of inertia was taken as 0.0351 m^2 and 0.00429 m^4. And Poisson's ratio and damping ratio of the pile were assumed to be 0.25 and 0.05 respectively.

And nonlinear soil properties of the soil layers were defined for the nonlinear dynamic analysis by the Ramberg-Osgood soil model specified in the following Eq. (2).

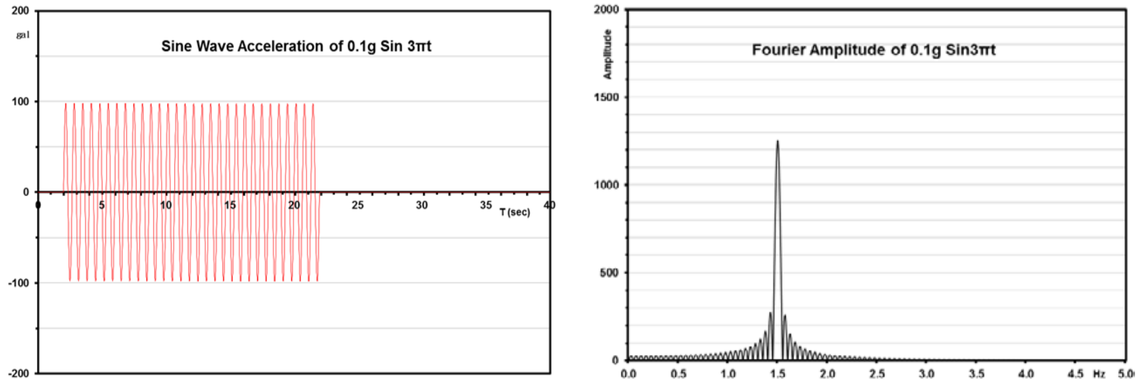
$$\frac{G}{G_{\max}} = \frac{2}{1+\sqrt{1+4\alpha\dfrac{\gamma}{\gamma_y}}} \qquad D = \frac{2}{3\pi}\,\frac{\sqrt{1+4\alpha\dfrac{\gamma}{\gamma_y}}-1}{\sqrt{1+4\alpha\dfrac{\gamma}{\gamma_y}}+1} \tag{2}$$

In here, G and G_{max} are shear moduli, γ is a shear strain, D is a damping ratio, γ_y is a yielding shear strain, and α is a material constant.



(a) G/G_{max} ratio (b) Damping ratio

Fig. 10 Ramberg-Osgood soil model



(a) Time history of 0.1 g sine wave acceleration (b) Fourier amplitude of 0.1 g sine wave

Fig. 11 Sample of 0.1 g sinusoidal wave

Ramberg-Osgood soil models for α of 0.02 and 0.023 and the yielding shear strain of 0.5~0.98×10^{-5} are given in Fig. 10, and two of them are also compared with Darendeli soil models in Fig. 3 to check the compatibility of soil models.

Pile head responses of a pile-soil system were investigated by applying sinusoidal accelerations having a period of 1.5 seconds shown in Fig. 11(a) to the bedrock under the soil layer. The amplitude of a sine wave was changed from 0.05 g to 0.38 g. Fourier amplitudes of the time history of 0.1 g sine wave acceleration are also shown in Fig. 11(b), and it can be seen that there is no frequency content at around 4.5 Hz.

Dynamic analyses of a pile-soil system were carried out in the frequency domain from 0 to 24.8 Hz with the frequency interval of 0.0244 Hz.

4. Numerical verification of experimental results

Numerical analyses using a P3DASS program were performed with measured raw base input motions and band filtered ones to check the reliability of test input motions. Transient acceleration and displacement responses of a pile head from tests and analyses are shown in Table 5 and Fig. 12 for 6 input motions. Acceleration and displacement responses of raw input motions are approximately 11% and 18% larger than those of 0-3 Hz band filtered motions in the case of strong excitations of 0.3 g and 0.38 g due to the effect of high frequency contents at around 4.5 Hz, but the effect of high frequency waves is minimal in the case of weak excitations smaller than 0.15 g.

Frequency amplitude contents shown in Fig. 7 indicate that raw input motions have almost negligible amplitudes in the lower frequency range. It is clear that measured raw input motions have poor wave contents in the frequency range below 3.0 Hz, and this might be due to the boundary conditions of the test apparatus and the technical problems in the data acquiring and processing procedures.

Fourier amplitudes in the low frequency range below 0.5Hz decreased responses about 20% and 50% with strong excitations of 0.3 g and 0.38 g respectively, but they affected just a little with weak excitations smaller than 0.15 g as shown in Fig. 12. Some part of those effects might be induced from the nonlinearity of soil layers depending on the intensity of input motions. Also

Table 5 Max. and transient responses of raw input motions

(a) With measured raw input motions

Input motion 1.5 Hz	Acceleration (g)			Displacement (m)		
	Test	P3DASS maximum	P3DASS transient	Test	P3DASS maximum	P3DASS transient
0.05 g	0.178	0.175	0.171	0.027	0.019	0.018
0.10 g	0.294	0.375	0.340	0.045	0.038	0.035
0.15 g	0.469	0.754	0.680	0.072	0.079	0.070
0.20 g	0.669	1.625	1.620	0.101	0.180	0.180
0.30 g	0.892	2.111	2.110	0.124	0.233	0.230
0.38 g	0.930	1.706	1.650	0.111	0.195	0.190

(b) With 0-3 Hz band-filtered raw input motions

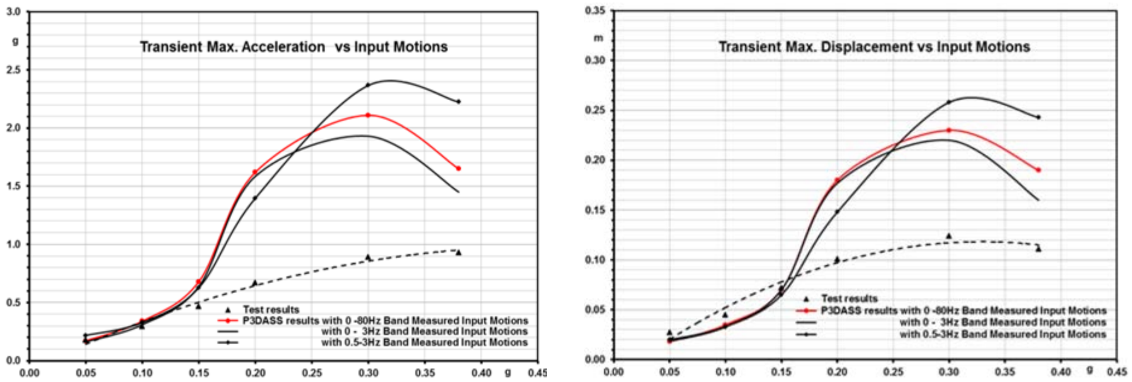
Input motion 1.5 Hz	Acceleration (g)			Displacement (m)		
	Test	P3DASS maximum	P3DASS transient	Test	P3DASS maximum	P3DASS transient
0.05 g	0.178	0.165	0.162	0.027	0.019	0.019
0.10 g	0.294	0.342	0.310	0.045	0.037	0.033
0.15 g	0.469	0.712	0.638	0.072	0.077	0.070
0.20 g	0.669	1.613	1.585	0.101	0.181	0.177
0.30 g	0.892	2.001	1.930	0.124	0.226	0.220
0.38 g	0.930	1.467	1.450	0.111	0.172	0.160

(c) With 0.5-3 Hz band-filtered raw input motions

Input motion 1.5 Hz	Acceleration (g)			Displacement (m)		
	Test	P3DASS maximum	P3DASS transient	Test	P3DASS maximum	P3DASS transient
0.05 g	0.178	0.239	0.218	0.027	0.025	0.020
0.10 g	0.294	0.356	0.328	0.045	0.040	0.033
0.15 g	0.469	0.655	0.628	0.072	0.072	0.065
0.20 g	0.669	1.516	1.397	0.101	0.158	0.148
0.30 g	0.892	2.402	2.366	0.124	0.265	0.258
0.38 g	0.930	2.285	2.224	0.111	0.259	0.243

some part of them might be due to amplitudes in the frequency range at around 1.5 Hz which are smaller than those of normal sine waves, and due to amplitude differences at the frequency of 1.5 Hz which are approximately 20%, 14%, 21% and 14% smaller for excitations of 0.1 g, 0.2 g, 0.3 g and 0.38 g respectively.

Also, P3DASS analyses were carried out to check the effect of amplitude at the frequency of 1.5 Hz by scale-downing Fourier amplitudes of sinusoidal input motions as much as amplitude differences between raw and sinusoidal input motions. Scale-down factor varies from 0.8 to 0.9 depending on the exciting level of input motion as shown in Table 6.



(a) Acceleration (b) Displacement

Fig. 12 Comparison of test and P3DASS analyses results

Table 6 Max. and transient responses for scale-downed sinusoidal input motions

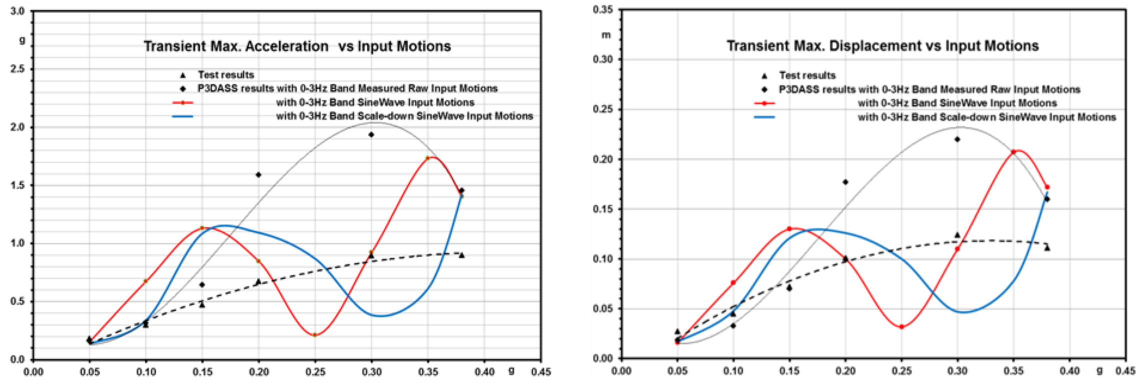
Input motion 1.5 Hz	Acceleration (g)			Displacement (m)			Scale-down factor
	Test	P3DASS maximum	P3DASS transient	Test	P3DASS maximum	P3DASS transient	
0.05 g	0.178	0.197	0.137	0.027	0.024	0.017	0.90
0.10 g	0.294	0.470	0.389	0.045	0.057	0.048	0.80
0.15 g	0.469	1.093	1.088	0.072	0.147	0.121	0.85
0.20 g	0.669	1.108	1.093	0.101	0.162	0.126	0.85
0.25 g	-	0.979	0.859	-	0.151	0.100	0.80
0.30 g	0.892	0.885	0.385	0.124	0.162	0.047	0.80
0.35 g	-	1.054	0.614	-	1.191	0.078	0.82
0.38 g	0.930	1.441	1.422	0.111	0.202	0.167	0.84

Also, P3DASS analyses were carried out to check the effect of amplitude at the frequency of 1.5 Hz by scale-downing Fourier amplitudes of sinusoidal input motions as much as amplitude differences between raw and sinusoidal input motions. Scale-down factor varies from 0.8 to 0.9 depending on the exciting level of input motion as shown in Table 6.

Responses of P3DASS analyses with sine wave input motions show big fluctuations for different amplitudes of the input motions, which might be due to the dynamic characteristics of the system. Analyses results with sinusoidal input motions shown in Fig. 13 indicate that Fourier amplitudes in the low frequency range reduces responses of acceleration and displacement in the cases of strong excitations, and scale-downed Fourier amplitudes of input motions make the trend of analytical responses more smooth and make analytical prediction of centrifuge test results better.

As responses of acceleration and displacement found with scale-downed sinusoidal input motions fluctuate along the trend curves of test results as shown in Fig. 14, they are curve-fitted using the least squares method to compare curve-fitted responses with test results of centrifuge model test.

Curve-fitted transient accelerations in Fig. 14(a) show a similar trend with the test results even



(a) Acceleration (b) Displacement

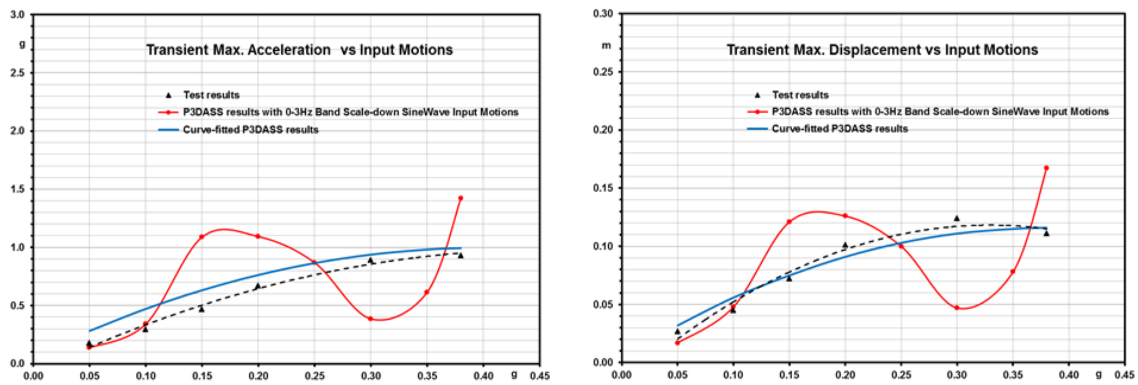
Fig. 13 P3DASS analyses results with raw and sinusoidal input motions

though there is some difference, but curve-fitted transient displacements show quite a good agreement to the test results with a little difference. Those differences seem to be caused due to some differences in the Fourier amplitudes of input motions shown in Fig. 7.

In the study, it can be noticed that measured input motions seem to be distorted somewhat in frequency contents and Fourier amplitudes due to the unclear reasons, and distorted input motions reduced responses of acceleration and displacement quite a bit. Those problems were verified by performing numerical analyses for a centrifuge test model with sinusoidal input motions and measured ones.

In comparing numerical results with test ones and predicting the trend of analytical results, it was a difficulty that test results for 0.25 g and 0.35 g excitations were missed due to the experimental convenience. However, it can be concluded that numerical finite element method using a P3DASS code is helpful to predict the experimental results of a centrifuge model test efficiently and properly.

Nonlinear soil properties were also investigated to verify the reliability of the program. In Table 7, experimental and analytical transient responses are compared for different intensities of input



(a) Acceleration (b) Displacement

Fig. 14 Curve-fitted P3DASS analyses results for scale-down sine input motions

Table 7 Summary of P3DASS analytical results for scale-down input motions

Input motion 1.5 Hz	Transient acc. (g)			Transient displ. (m)			G/G_{max}		
							Soil Depth 8.2 m		
	Test	P3DASS	Difference (%)	Test	P3DASS	Difference (%)	R = 9 m	R = 49 m	Far field
0.05 g	0.178	0.137	-23	0.027	0.017	-37	0.67	0.48	0.62
0.10 g	0.294	0.341	16	0.045	0.048	7	0.52	0.39	0.52
0.15 g	0.469	1.088	132	0.072	0.121	68	0.36	0.32	0.40
0.20 g	0.669	1.093	63	0.101	0.126	25	0.28	0.28	0.34
0.25 g	-	0.869	-	-	0.100	-	0.23	0.25	0.31
0.30 g	0.892	0.385	-57	0.124	0.047	-62	0.19	0.23	0.27
0.35 g	-	0.614	-	-	0.078	-	0.14	0.21	0.22
0.38 g	0.930	1.422	53	0.111	0.167	50	0.12	0.19	0.18

motions, and final normalized shear moduli (G/G_{max}) retrieved from numerical analyses at the depth of 8.2 m and the radial distance of 9 m, 49 m and far field are summarized.

It can be seen that the soil nonlinearity is increased and become clearer as the intensity of input motion becomes stronger, and it is also obvious even at the far field with weak input motions. So it seems to be unreasonable to consider the soil characteristics to be linear even in the far field.

Shear modulus ratios (G/G_{max}) of finite soil elements which were found with a scale-downed 0.3 g sinusoidal input motion are shown in Fig. 15. Shear modulus at the far field was assumed by averaging shear moduli of elements located in the region between the radius of 9 m and lateral boundary for each layer. Shear modulus ratio of soil element around a pile decreased gradually with the depth, and shear modulus ratios of soil layers varies quite gradually along the soil depth except at around the lateral boundary. Shear modulus ratio at the inside of lateral boundary reduced somewhat due to the effect of far field. However, the variation of shear modulus ratios

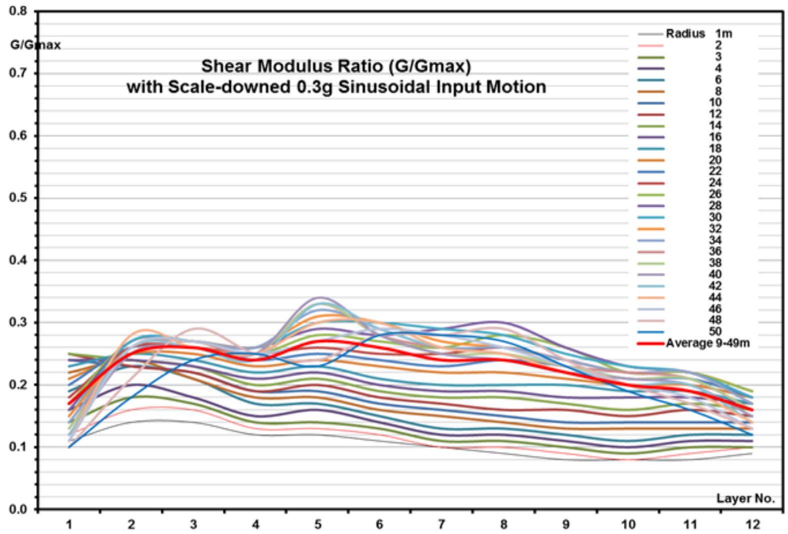


Fig. 15 Shear modulus ratio in finite elements with scale-downed 0.3 g input motion

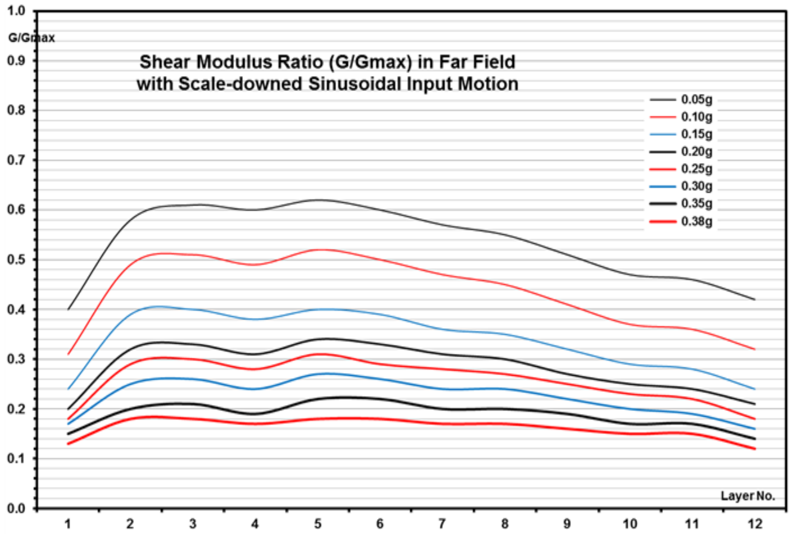


Fig. 16 Shear modulus ratio in the far field

seems to be reasonably well defined, even though it is not easy to estimate the nonlinear soil properties in the numerical process. The shear moduli of the overall soil system were estimated at the range of 10~30% of the initial shear moduli.

Shear modulus ratios of soil layers in the far field are also shown in Fig. 16 for 8 different intensities of input motion. Shear modulus ratio of each soil layer reduced asymptotically from maximum of 0.62 to minimum of 0.12 at all layers as the input motion becomes stronger, and shear modulus ratio decreased gradually along the soil depth except at the top surface layer. It might be due to the reflecting surface boundary effect that shear modulus ratio at the top layer is smaller than those of other layers.

Also it can be concluded that the soil nonlinearity at the far field is mostly induced from the input motion applied at the bottom boundary. However, soil properties at the far field are assumed to be linear in the most nonlinear seismic analyses.

5. Conclusions

In this study, numerical analyses for a pile-soil system were carried out to verify the experimental results of centrifuge model tests. Centrifuge model tests were performed at the KOCED Geo-centrifuge Testing Center in KAIST applying 1.5 Hz sinusoidal base input motions, and nonlinear numerical analyses were done utilizing a finite element program of P3DASS with the same sinusoidal bedrock input motions having the intensities of 0.05 g~0.38 g. The results of this analytical study can be concluded as follows.

Nonlinear numerical analyses utilizing a P3DASS program were helpful to predict the trend of experimental responses of a centrifuge model efficiently, even though there were some difficulties in processing analytical results and finding out unintended deficits in measured experimental data. Also nonlinear soil properties of elements in the system can be estimated adequately using this analytical program to compare them with experimental results.

However, there were some problems to revive the experimental responses properly by an analytical method. Measured input motions seem to be distorted in frequency contents and Fourier amplitudes due to the unclear reasons, and distorted input motions reduced responses of acceleration and displacement quite a bit. Measured input motions include unexpected high frequency contents at around 4.5 Hz, and Fourier amplitudes at all frequency range especially in low frequencies are smaller than those of normal sinusoidal motions.

Soil properties in the entire system including a far field showed high nonlinearity even with a weak input motion of 0.05 g affecting on the responses of a system, and soil nonlinearity changes depending on depth and radial distance of a soil element. So, it is reasonable and necessary to take into account the soil nonlinearity even with a weak excitation defining soil properties for each element separately. Soil characteristics of a test model also should be checked prior to each experimental test, because the soil in the test box could be consolidated due to the consecutive dynamic tests deteriorating the test results.

Acknowledgments

This research was supported in part by Research Funds of Mokpo National University.

References

Choi, J.I. and Brandenberg, S.J. (2015), “Cyclic *p-y* plasticity model applied to pile foundations in sand”, *J. Geotech. Geoenviron. Eng., ASCE*, **141**(5), 04015013.

Danno, K. and Kimura, M. (2009), “Evaluation of long-term displacements of pile foundation using coupled FEM and centrifuge model test”, *Soils Found., Japanese Soc. Soil Mech. Found.*, **49**(6), 941-958.

Darendeli, B.M. (2001), “Development of a new family of normalized modulus reduction and material damping curves”, Ph.D. Dissertation; University of Texas at Austin, TX, USA, August, 362 p.

Giannakos, S., Gerolymos, N. and Gazetas, G. (2012), “Cyclic lateral response of piles in dry sand: Finite element modeling and validation”, *Comput. Geotech.*, **44**, 116-131.

Kausel, E. (1974), “Forced vibrations of circular foundations on layered media”, Research Report R74-11; Department of Civil Engineering, MIT.

Kim, Y.S. (2015), “Pseudo 3D FEM analysis for wave passage effect on the response spectrum of a building built on soft soil layer”, *Earthq. Struct., Int. J.*, **8**(5), 1241-1254.

Kim, Y.S. and Roesset, J.M. (2004), “Effect of nonlinear soil behavior on the inelastic seismic response of a structure”, *Int. J. Geomech., ASCE*, **4**(2), 104-114.

Kim, Y.S., Roesset, J.M. and Stokoe II, K.H. (1987), “Interpretation of vertical vibration tests on small scale piles”, In: *Dynamic Response of Pile Foundations-Experiment, Analysis and Observation, Geotechnical Special Publication No.11*, (ISBN 0-87262-591-5), ASCE, pp. 110-126.

Kim, S.R., Hwang, J.I., Kim, M.M. and Ko, H.Y. (2006), “Comparison of 1-g and centrifuge model tests for similitude laws”, *J. Korean Geotech. Soc.*, **22**(5), 59-67.

Kim, D.S., Lee, S.H., Choo, Y.W. and Perdriat, J. (2013), “Selfbalanced earthquake simulator on centrifuge and dynamic performance verification”, *J. Civil Eng., Korean Soc. Civil Engrs.*, **17**(4), 651-661.

Lee, S.H., Choo, Y.W. and Kim, D.S. (2013), “Performance of an equivalent shear beam (ESB) model container for dynamic geotechnical centrifuge tests”, *Soil Dyn. Earthq. Eng.*, **44**, 102-114.

Minowa, C., Sato, M., Kagawa, T. and Abe, A. (2001), “Reproduction of large-scale 1G test on dry sand deposits and pile foundations using centrifuge modeling(Step-2)”, *NIST Special Publication SP, National Institute of Standards & Technology*, **963**, 263-270.

Roesset, J.M. and Kim, Y.S. (1987), “Specification of control motions for embedded foundations”,

Proceedings of 5th Canadian Conference on Earthquake Engineering, Ottawa, Canada, July, pp. 63-86.
Yoo, M.T., Cha, S.H., Choi, J.I., Han, J.T. and Kim, M.M. (2012), "Evaluation of dynamic group pile effect in dry sand by centrifuge model tests", *J. Korean Geotech. Soc.*, **28**(1), 67-77.
Yoo, M.T., Ha, J.G., Jo, S.B. and Kim, D.S. (2014), "Evaluation of seismic loading of pile foundation structure considering soil-foundation-structure interaction", *J. Earthq. Eng. Soc. Korea*, **18**(3), 125-132.
Zhussupbekov, A.Z., Utepov, Y.B., Shakhmov, Z.A. and Ling, H.I. (2013), "Model testing of piles in a centrifuge for prediction of their in-situ performance", *Soil Mech. Found. Eng.*, Springer Science + Business Media, **50**(3), 92-96.

CC