

LATERAL SOIL RESISTANCE OF EACH PILE IN PILE GROUP BY SHAKING TABLE TESTS AND ITS SIMULATION ANALYSIS

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Abstract

Pile foundation response during earthquakes is strongly affected by nonlinear soil-pile foundation interaction. The damages to pile foundations during the Hyogo-ken Nanbu earthquake of 1995 and the Tohoku-Chiho Taiheiyo-Oki Earthquake of 2011 were obviously attributed to nonlinear interaction of soil-pile foundation-superstructure. The dynamic nonlinear behavior of soil around each pile affects the lateral load distribution and displacement of each pile. Therefore, shaking table tests were conducted to clarify the dynamic nonlinear behavior of soil around piles, next earthquake response analyses were conducted to simulate the effect of the nonlinear soil-pile interaction system on the performance of the superstructure supported by pile group.

In the shaking table tests, 5x5-pile group foundation model was set up in Toyoura sand deposits. The sand deposits were prepared by air pluviation method in the laminar box. The model piles were acryl cylinder with 12mm in diameter and 400mm long. 25 piles were arranged in squares and pile spacing was 2.0 times pile diameter. The superstructure was modeled as a rigid body. The input waves were seismic waves in notification with random phase of Japan and the Hyogo-ken Nanbu earthquake. 3 input acceleration levels were used to investigate nonlinear effects of soil by input motion level. Tests with or without the mass of superstructure were conducted to investigate lateral load distribution of each pile. The validity of 3D FEM analyses was presented by comparing the calculated analysis results with the shaking table test results, and the nonlinear behavior of the pile group foundation-soil- superstructure was discussed.

The concluding remarks of these shaking tests and analytical studies are as follows.

(1) Lateral subgrade reaction around piles depends on the location of each pile in pile group. That of the corner pile is remarkably larger than that of the middle pile.

(2) The hysteresis curve of soil spring at the corner pile is asymmetric loop, while that at the middle pile is symmetric loop.

(3) The bending moment at pile head depends on the location of each pile in pile group. That of the corner pile head is remarkably larger than that of the middle pile head. As the input acceleration level increase, bending moment at each pile head becomes to be almost equalized.

(4) It was confirmed that nonlinear 3D FEM model represents well response of the superstructure supported by pile group.

Keywords: Pile group, Nonlinear soil resistance of pile, Shaking table test, Nonlinear 3D FEM



1. Introduction

Numerous pile foundations were damaged during the Hyogo-ken Nanbu earthquake of 1995. The observed near-fault ground motions were far stronger than those commonly used for Japanese seismic design. Similarly, the pile foundations in soft ground and liquefied ground were damaged during the Tohoku-Chiho Taiheiyo-Oki Earthquake of 2011 [1]. Pile foundation response during extreme earthquakes is strongly affected by nonlinear soil-pile foundation interaction. The nonlinear soil-pile foundation interaction in pile group is a complex phenomenon. Recently much attention has been paid to overcome this problem. Many tests of the lateral loading to the pile heads have been conducted.

As an instance of static tests, in-site lateral loading tests [2, 3] and lateral loading tests under normal gravity condition for model pile foundation [4-7] have been conducted. As an instance of dynamic tests, large shaking table tests, dynamic centrifugal tests [8, 9] and vibration tests using ground motion from large-scale blasting operations as excitation force [10] have been conducted. Many researchers have investigated the pile group effects on the pile head spring. However, only few studies have been reported on the nonlinear soil spring depended on the location of each pile in pile group, the depth and the loading direction [6, 10]. Therefore, shaking table tests were conducted to clarify the dynamic nonlinear behavior of soil around piles. Further, earthquake response analyses using 3D FEM were conducted to simulate the effect of the nonlinear soil-pile interaction system to evaluate the performance of the superstructure supported by pile group.

2. Shaking table tests

The shaking table tests were performed under a normal gravity condition. Figure 1 shows overview of the test model and measuring points. The shaking table has dimensions of length L=600mm and width W=1200mm at Osaka University. The laminar box has inside dimensions of length L=400mm, width W=600mm and height H=400mm. 5x5-pile group foundation model was set up in Toyoura sand deposits. The sand deposits were prepared by air pluviate in the laminar box. The model piles were acryl cylinders with 12mm in diameter and 400mm in length. As indicated in the Figure 2, 25 piles were arranged in squares and pile spacing was 2 times pile diameter. Pile group effect was large in this pile arrangement. The pile heads were connected to a foundation plate. The pile tips were connected to an iron plate. The superstructure was modeled as a rigid body. The mass of the superstructure is 54.2kg (a part of superstructure: 52.9kg, a part of foundation plate: 1.32kg).

Tests with or without the mass of superstructure were conducted to investigate the lateral load distribution of each pile. The input waves were seismic waves in notification with random phase of Japan (ART) and the Hyogo-ken Nanbu earthquake (Hyogo). The two input waves were modified based on the maximum velocity to obtain 50kine and the time axes were made one-fifth. Figure 3 shows the two modified input waves. In tests, the amplitude of the acceleration of 0.4, 1.0, 2.0 times were used. Table 1 summarizes the order of the shaking table tests.



P5

24

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11

Unit:mm







Figure 3. Acceleration time histories of input motion

Table	1.	Case	of	shaking	table	test
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case	input	amplitude (%)	weight(kg)	
Hyogo04	Hyogo	40		
ART04	ART	40		
Hyogo10	Hyogo	100	54.2	
ART10	ART	100	34.2	
Hyogo20	Hyogo	200		
ART20	ART	200		



3. Result of shaking table tests

3.1 Fourier spectrum ratio

Figure 4 shows Fourier spectral ratios of acceleration time histories between that at the top of superstructure and the input acceleration obtained by shaking table test for ART04, ART10 and ART20. The natural frequency of the soil-structure supported by pile group system was about 8Hz for ART04. Figure 5 shows Fourier spectral ratios of acceleration tiome histories between that at the surface of soil and the input acceleration by shaking table test in ART04, ART10 and ART20. The natural frequency of the soil model was about 23Hz in ART04. As the input acceleration level was larger, these peak amplitudes and peak frequencies were lower respectively. This showed that hysteresis damping was more significant and stiffness was smaller due to the soil nonlinearity around pile.



top of superstructure/input



3.2 The distributions of the maximum bending moments of the pile

Figure 6 shows the distributions of the maximum pile bending moment obtained by shaking table test in ART04, ART10 and ART20. The distributions of the maximum bending moment of each pile were different. The bending moment at pile head of P5 was larger than that of P3 and P13 in all cases. The inflection depths of the maximum bending moment for P5 were found to be shallower than those for P3 and P13 in all cases. As the input acceleration level increased, the inflection point for each pile was deeper, and the distribution of the maximum bending moment of each pile was closer.



Figure 6. The distributions of the maximum pile bending moment



3.3 Soil springs along each pile

Figure 7 (a) and (b) show the relationship of subgrade reaction around pile and relative displacement of P3, P5 and P13 at GL-35mm and GL-95mm for ART04, ART10 and ART20. Relative displacement was evaluated at the various locations of pile relative to the pile tip. Equivalent stiffness at the maximum displacement for ART04, ART10 and ART20 were shown in the graphs. The subgrade reaction and the pile displacement were calculated as follows. The bending moment distribution of pile was approximated to fourth-order function (Eq.1) by the least square method and subgrade reaction was calculated by the second differentiation of the approximated function.

$$M = \sum_{k=0}^{4} a_k z^k \tag{1}$$

Where z is depth starting from the pile head and a_k is coefficient of z^k . The pile displacement was calculated by the second integration of the approximated function of curvature using boundary condition. The boundary conditions were assumed that the displacement of pile head was equal to that of the foundation plate at pile head and the displacement of pile tip was equal to zero.

In Figure 7, equivalent stiffness of P5 was larger than those of P3 and P13. As P5 was displaced in the positive direction, subgrade reaction became large because P5 turned to be located in the front row. On the other hand as P5 was displaced in the negative direction, P5 turned to be located in the back row. For that reason the hysteresis loop of P5 was asymmetric. Regardless of displacing direction, P3 and P13 were located between piles. Thus the hysteresis loop of P3 and P13 was found to be symmetric. Soil resistance around pile group depended on the location of each pile in pile group and the shaking direction. The difference of soil resistance around piles caused the difference of pile bending moment. The inertia force of the superstructure was distributed to the each pile according to soil resistance. For that reason the bending moment at pile head of P5 was larger than that of P3 and P13. Subgrade reaction along the pile also affected the distribution shape of bending moment. As subgrade reaction depth of the pile maximum bending moment for P5 was found to be shallower than that for P3 and P13. Focusing on input acceleration level, the hysteresis loop for ART20 included that for ART10 and ART04. In addition to this, as the input acceleration level increased, equivalent stiffness was lower. Thus pile maximum bending moment distribution of each pile was closer.

4. Overview of 3D FEM analysis

Figure 8 shows finite element model for the shaking table test. The size of the finite element model was same as that of the test model. The material properties used in the analyses are summarized in Table2. The superstructure and foundation plate were modelled as a rigid body. The mass of the superstructure and foundation plate was equivalent to the test model. For simulating a laminar container, the bottom of the soil model was fixed. The boundary of the soil model was applied the condition that the displacements of all three coordinate directions at the same depth equaled. In order to consider the friction and separation between piles and soil, the contact condition based on penalty method was used between the contact surface of piles and soil. The piles were modelled as elastic shell elements. The pile diameter was equivalent to the test model. The thickness of the shell element assumed to be 6mm. The Young's modulus of the shell element was equivalent to the bending rigidity of the test model. Considering the contact condition of pile heads, rotational springs (k_{θ}) were applied between pile head and foundation plate. The value of damping ratio was 5% for the initial frequency of the system.

Shear velocity of soil model was given by Eq.2. Where H is depth in the soil and α is coefficient adopted for evaluation.

$$Vs = \alpha H^{0.25} \tag{2}$$

The value of α was determined from adjusting according to the peak frequency of Fourier spectral ratios between the ground surface and the input acceleration of shaking table test. Those were done using the response of free



ground for ART04 without the mass of superstructure. The response of free ground was calculated by one dimensional time history analysis. From the result of simulation analysis, the value of α was found to be 59. The soil model was modelled as solid. The relationships of the shear stress and shear strain in the soil model was defined by the polylinear skeleton curve [11]. Initial shear stiffness was calculated from the initial shear velocity. The yield shear stress was calculated from the Mohr-Coulomb constitutive model.



(b) GL-95mm

Figure 7. Relationship of the subgrade reaction around pile- relative displacement



Table2. Material properties

Structure	Rigid	Density(g/cm ³)	7.8
Structure	8	Young' modulus(N/mm ²)	205000
Pile	Elastic	Density(g/cm ³)	1.2
		Young' modulus(N/mm ²)	3500
Soil	Polylinear	Density(g/cm ³)	1.5
		Poisson's ratio	0.4
		Internal friction angle(deg)	40
		Shear velocity(m/s)	$59H^{0.25}$ (Depth)

Figure 8. 3D FEM analytical model (1/2model)



5. Simulation analyses by the 3D FEM

In this section, the calculated results were compared with the test results for Hyogo04, Hyogo10 and Hyogo20.

5.1 Acceleration response spectra at the top of the superstructure

Figure 9 compares the calculated acceleration response spectra for 5% damping at the top of the superstructure with the test results for Hyogo04, Hyogo10 and Hyogo20. The calculated results were good agreement with the test results. However the peak amplitudes of the calculated acceleration response spectra were a little smaller than those of the test results.



Figure 9. Comparisons of the acceleration response spectra for 5% damping at the top of the superstructure

5.2 Distributions of the maximum bending moments of the pile

Figure 10 compares the calculated distributions of maximum bending moments of the piles with the test results for Hyogo04, Hyogo10 and Hyogo20. The calculated results were good agreement with the test results. The calculated distribution of the maximum bending moments of each pile was different for all cases. The bending moment at pile head of P5 was larger than those of P3 and P13 in all cases. The inflection depth of the distribution of the maximum bending moments for P5 was found to be shallower than those for P3 and P13 in all cases. As the input acceleration level increased, the inflection depth for each pile was deeper, and the maximum bending moment distribution of each pile was closer. 3D FEM analyses simulated well these pile responses depending on the location in pile group and the input acceleration level.

5.3 Strain distribution around pile calculated by 3D FEM

Figure 11 shows logarithmic x strain distribution in X-Y section at ground surface when the bending moment at pile head of P5 reached the maximum value for ART10. The high strain region occurred in the soil around piles. In particular, the high strain region of soil was concentrated around the leading piles. Thus the inertia force of the superstructure was mainly distributed to the leading piles.

Figure 12a and 12b show logarithmic x strain distribution in X-Z section when the bending moment at pile head of P5 reached the maximum value for ART10. Figure 12a was the section which was the vertical section through P1-P5. Figure 12b was also the vertical section through P11-P15. The high strain region occurred in the soil around piles. The high strain region around the piles extended to the deeper depth in the front row and in the back row of pile group. In addition, the high strain region of soil occurred in the front side of the piles. However the strain around the middle piles was not so large. The more depth of the soil was shallow, the high strain region more extended.



Figure 10. Comparisons of the distribution maximum pile bending moment



Figure 11. Logarithmic x strain distribution at ground surface



 5×10^{-5}

Figure 12. Logarithmic x strain distribution around pile



6. Conclusions

To investigate the mechanism of nonlinear soil-pile interaction system on performance of the superstructure supported by pile group foundation, the shaking table tests were conducted. Furtherrmore, 3D finite element analyses were performed to get understandings of the tests results. The following conclusions were obtained.

- 1) From the result of the shaking table tests, each pile in pile group showed the different response. The inertia force of superstructre was distributed to each pile according to soil resistance around pile. Thus the bending moment of the corner pile was found to be larger for the other piles. The bending moment distribution of each pile in pile group was different according to the location in pile group. The inflection depth of the bending moment distributions for the corner pile were shallower than the other piles. As the input acceleration level increased, the inflection depth of distribution of the maximum bending moments for each pile was deeper, and that of each pile was closer.
- 2) From the result of the shaking table tests, the hysteresis loops of the soil spring around pile in pile group were found to be different. The lateral subgrade reaction around the corner piles was found to be different according to the displacing direction. For that reason, the hysteresis loop of the corner piles was asymmetric. However the hysteresis loops of the middle piles were symmetric.
- 3) The behavior of nonlinear response of the superstructure supported by pile group foundation obtained by the shaking table tests can be simulated well by 3D FEM analysis.

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