Shake Table Test of Nonlinear Structure-Pile Foundation-Soil System

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SUMMARY:

This paper presents shake table tests for evaluating dynamic interaction of RC column with a pile foundation and soil system. In the case of small input motion, both the structure and the soil were linear and the inertial interaction was dominant in the response of the pile foundation. Increasing the shaking level, the effect of kinematic interaction on the response became major and when the RC column became nonlinear, the piles' behaviour was strongly affected by the ground displacement and the interaction between the structure and the soil became smaller. Compared with the dynamic response of RC column between the column stand-alone tests and SFSI tests, the effect of the dynamic interaction can be observed.

Keywords: dynamic interaction, nonlinear response, RC column, shake table test, pile

1. INTRODUCTION

As one of the philosophies of seismic design of bridges, a principal plastic hinge is formed at the bottom of piers and no damage or secondary plastic hinge is formed at foundations because the damage of foundation is not easy to be repaired. In order to evaluate the seismic performance of a bridge, it is necessary to model the whole system, including foundations and soil. For a bridge system subjected to seismic loadings on the aboveground portion of the structural system, the soil-foundation-structure interaction (SFSI) problem involves evaluation of the performance.

A considerable number of studies have been made on SFSI problem theoretically, numerically and experimentally. However in almost all studies the aboveground portions of the structural system were modelled very simply and researchers were interested in the nonlinear behaviour of foundations. On the other hand, experimental tests of nonlinear structures without foundation are common in the structural engineering field. As the results, no SFSI studies have ever tried to conduct experimental tests with nonlinear structures in which structural details are satisfied with a design code. In this study, shake table tests of nonlinear RC columns with a foundation and soil were conducted at the shake table of Obayashi Corporation to evaluate the nonlinear dynamic characteristics of the column and the whole system.

2. SHAKE TABLE TESTS OF RC COLUMNS

Takahashi and Kobayashi (2011) conducted shake table tests of RC columns at the world's largest shake table, E-Defense. 16 RC column models were placed on the 20 x 15 m table (Fig. 1) and excited simultaneously. This is a test of RC columns without the foundation and a typical test setup in structural engineering field, but a very unique because the multiple same test units were excited simultaneously. As the results, the data of the dynamic nonlinear response in which the same dynamic



input is guaranteed were obtained in the process of collapsing.

The model consists of a RC column, a weight and a footing part. The column is a 1.6 m tall, 0.32 m square reinforced concrete column. D10 was used for longitudinal reinforcement and D4 was used for transverse reinforcement and intermediate reinforcement. Transverse reinforcement was arranged every 40mm. As a result, the longitudinal reinforcement ratio was 1.11% and transverse reinforcement ratio was 0.86%. The weight is a reinforced concrete cube of 1.6 m on a side. The weight is 10 ton. The column is a 1/7.5 scaled RC pier of the objective bridge pier.

The input earthquake motions were 3 components of JR Takatori record, 1995 Kobe Earthquake, and the time scale was 1/7.51/2 of the original scale considering the similitude. After series of linear response tests, 100% JR Takatori record was inputted and the models showed the dynamic nonlinear behavior. Finally 150% JR Takatori record was inputted.

The typical results of the displacement responses of columns in the 100% input test are shown in Fig. 2. The histograms of the maximum and the residual deformation are shown in Fig. 3. In spite of the dynamic nonlinear behavior, the COV of the maximum displacement is only 3.8 %.



Figure 1. 16 Models on E-Defense tests and the bar arrangement of the model



Figure 2. Displacement Responses of 16 Models



Figure 3. Histograms of maximum and residual displacement and their average, standard deviation and covariance



Figure 4. Model of shake table tests



Figure 5. Test setup on shake table

3. SFSI MODEL OF SHAKE TABLE TESTS

Fig. 4 and 5 show the model of the shake table tests. The model was constructed of a structure (RC column and a weight), a pile group foundation and soil in a laminated shear box.

3.1. Structure

The structure part was a RC column with a weight. The RC column was the same in Fig. 1, but the weight (10.1 ton) can be changed to the light weight (5.9 ton) to evaluate the influence of the inertial force. In this paper, the case in which mass of the weight was 10.1 ton was called as "Case 1", and the case in which mass of the weight was 5.9 ton was called as "Case 2". The natural period of the linear structure in Case 1 was 0.23 sec, the one in Case 2 was 0.18 sec.

3.2. Pile Foundation

In these tests, the pile group foundation was designed not to fail before the RC column failed. Within this restriction, the number, the diameter and the length of the piles were decided. And, βL was set as 2.5 and over, because piles with $\beta L < 2.5$ are considered as short piles and generally, piles of bridges are βL is about 3. β is the characteristic value and L is the length. β is calculated as

$$\beta = \sqrt[4]{\frac{k_h D}{4EI}} \tag{1}$$

 k_h is lateral subgrade reaction modulus (kN/m³), *D* is diameter (mm), *EI* is flexural rigidity (kNm²). As a result, the number of steel piles is 6 (3 for *x*-direction (NS-direction), 2 for *y*-direction (EW-direction)), the diameter is 190.7 mm, the thickness is 5.3 mm and the length is 3300 mm. The pile heads were fixed to a footing and their tips were connected to the base of the shear box with pins.

In this situation, nonlinear static analysis was performed to the piles. In this analysis, both inertial force and ground displacement acted on the pile foundation. Considering Takahashi, Y. and Kobayashi, N. (2011), the yield acceleration of the structure was assumed to be 700 gal. And, the ground displacement was calculated by 1-dimensional dynamic analysis, assuming relative density of the soil to be 60%. When these two effects acted on the piles, it was confirmed that safety factor was twice and over in terms of yield stress.

3.3. Soil Profile

In these tests, the dimensions of the laminated shear box were 4.0 m in width, 1.5 m in length, 3.0 m in height. All input motions were 1-dimensional shakings, so the shear box was designed to deform only in *x*-direction (NS-direction). Soil was dry silica #6. The soil profile consisted of 1-layer, 3.0 m thick. The relative density was about 80% and the natural period of the soil profile was 0.11 sec.

4. INPUT MOTIONS OF SHAKE TABLE TESTS

In order to compare the results of this SFSI tests with the E-Defense test, the input motion to the shake table was calibrated to be the same as the E-Defense tests at the ground surface level. Since the laminated soil box can move only in one direction, N45W direction of JR Takatori record was selected as the input motion considering the strong axis of the response of the E-Defense tests. Fig. 3 shows time history of one of the input motions (amplitude is the same as JR Takatori) and Fig. 4 shows acceleration response spectra of the input motion.

The amplitude of the input motion was gradually increased in the series of tests. In Case 1, the input motion was adjusted to 100 gal, 200 gal, 616 gal (amplitude is the same as JR Takatori, Fig. 6 and 7) and 924 gal (1.5 times of JR Takatori) for peak acceleration. And, in Case 2, in addition to the four motions in Case 1, one input motion whose peak acceleration was 1233 gal (2 times of JR Takatori) was also made. In this paper, each case is expressed as "Case-peak acceleration", e.g. Case 1-616.



Figure 6. Time history of the input motion



Figure 7. Acceleration response spectrum of the input motion

5. NONLINEAR LEVEL OF EACH CASE

Fig. 8 shows load-displacement relationship of structure (left row) and shear stress-shear strain relationship of soil at the 0.6 m (middle row) and 2.1 m depth (right row) in Case 1. From Fig. 5, in Case 1-100, both structure and soil are considered to be linear. In Case 1-200, structure is considered to be linear, but soil is considered to become nonlinear. In Case 1-616 and Case 1-924, both structure and soil are considered to become nonlinear.



Figure 8. Load-displacement relationship of the structure and shear stress-shear strain relationship of the soil at 0.6m and 2.1m depth in Case 1

Similarly, Fig. 9 shows load-displacement relationship of structure and shear stress-shear strain relationship of soil at the 0.6m and 2.1m depth in Case 2. In Case 2-1233, the wire sensor exceeded their range, so displacement of structure was not able to be correctly measured from the middle of the shaking. From Fig. 6, in Case 2-100, both structure and soil are considered to be linear. In Case 2-200, structure is considered to be linear, but soil is considered to become slightly nonlinear. In Case 2-616, Case 2-924 and Case 2-1233, both structure and soil are considered to become nonlinear. Table 1 shows natural period of structure after shaking in each case (Case 2-1233 was no data).



Figure 9. Load-displacement relationship of the structure and shear stress-shear strain relationship of the soil at 0.6m and 2.1m depth in Case 2

Table 1. Natural period of structure after shaking in each case									
Case	1-100	1-200	1-616	1-924	2-100	2-200	2-616	2-924	2-1233
Natural(sec)	0.23	0.23	0.48	0.56	0.18	0.19	0.40	0.46	-

Fable 1 Natural period of structure after shaking in each case

6. EVALUATION OF SOIL-STRUCTURE INTERACTION BY DISTRIBUTIONS OF BENDING MOMENT ALONG PILE

In this chapter, the effects of dynamic interaction are considered in terms of distributions of bending moment along a pile. In section 5.1, distributions of bending moment along the pile are compared among each case. In section 5.2, distributions in the cases of nonlinear structure are evaluated more in detail.

6.1. Comparison among Each Case

Fig. 10 shows the distributions of bending moment along the pile when bending moment at the pile head took peak value in each case. First, in the cases when both structure and soil are linear, Case 1-100 and Case 2-100, bending moment takes plus value from the tip slightly and starts to increase in minus area from about 1m in depth. These distributions express that inertial force affected the response of the piles, so inertial interaction was dominant. In fact, ground displacement was very small, so it can be considered that the influence of inertial force of structure is relatively greater than that of ground displacement.

Next, in the cases when structure is linear, but soil is slightly nonlinear, Case 1-200 and Case 2-200, the point where bending moment starts to increase in minus area is $1.5 \sim 2.0$ m in depth. These express that the influence of ground displacement in the case of 200 gal got greater than in the case of 100 gal, because soil slightly became nonlinear.

Finally, in the cases when both structure and soil are nonlinear, Case 1-616, Case 1-924, Case 2-616, Case 2-924 and Case 2-1233, bending moment starts to increase in minus area from the tips. This distribution expresses that ground deformed so large and kinematic interaction got dominant. In these cases, structure became nonlinear, and they are most notable cases. So, they will be evaluated more in detail in the next section.



Figure 10. Distributions of bending moment along pile when bending moment at pile head takes peak value in each case

6.2. Evaluation in the Cases of Nonlinear Structure

In this section, the cases when structure became nonlinear are evaluated more in detail. Now, from the load-displacement relationships in Case 1-616 and Case 1-924 (Fig. 8), it can be noticed that the peaks of inertial force ("Load" in Fig. 8) were most of the same, though the peak acceleration of the input motion in Case 1-924 was 1.5 times as large as in Case 1-616. Similarly, the peaks of inertial forces in Case 2-616, Case 2-924 and Case 2-1233 were most of the same. From this, it can be considered that when structure became nonlinear, inertial force hit a peak, so the influence of inertial force to the piles became relatively small. And, the influence of ground displacement became more major. Fig. 11 shows the distributions of bending moment along the pile when bending moment at the pile head, ground displacement and inertial force took peak values respectively in the cases when structure became nonlinear.



when bending moment at pile head took peak value (left), ground displacement took peak value (middle), inertial force took peak value (right).



(e) Case 2-1233

First of all, I explain Case 1-616 (Fig. 11(a)). From Fig. 11(a), it can be recognized that the peak ground displacement and the peak bending moment occurred at the same time (both 4.465 sec). On the other hand, the peak inertial force and the peak bending moment occurred at the subtle different time (4.465 sec and 4.530 sec) and the bending moment at the time of the peak inertial force also took relatively large value. Thus, the piles were likely to be affected by the ground displacement as well as the inertial force.

Similarly, also in Case 1-924 (Fig. 11(b)), the peak ground displacement and the peak bending moment occurred at the same time (both 6.820 sec). On the other hand, the peak inertial force and the bending moment occurred at the completely different time (6.820 sec and 7.000 sec), and the bending moment at the time of the peak inertial force took so small value, though the inertial force took the peak value. Thus, in Case 1-924, the piles were likely to be strongly affected by the ground displacement. This is considered that by structure becoming nonlinear, the inertial force got out of phase with the ground displacement, and additionally, the inertial force hit a peak. As a result, behavior of the piles was dominated by the ground displacement.

In Case 2-616 (Fig. 11(c)), the peak bending moment, the peak ground displacement and the peak inertial force occurred at the almost same time (4.170 sec, 4.170 sec and 4.175 sec). Thus, the piles were likely to be affected by both the ground displacement and the inertial force. Additionally, it can be also noticed that the response of the piles in Case 2-616 was larger than in Case 1-616. It is considered that because the nonlinear level of structure in Case 2-616 was smaller than in Case 1-616 (Fig. 8 and Fig. 9), the piles were affected by both the peak inertial force and the peak ground displacement, and the response of the piles in Case 2-616 was larger than in Case 1-616.

In Case 2-924 (Fig. 11(d)), the peak ground displacement and the peak bending moment occurred at the almost same time (5.325 sec and 5.320 sec), but the peak inertial force and the peak bending moment occurred at the completely different time (5.425 sec and 5.320 sec). And, the bending moment at the time of peak inertial force took only about half value of the peak bending moment, though the inertial force took the peak value. Nonlinear level of the structure in Case 2-924 became larger than in Case 2-616, so the inertial force got out of phase with the ground displacement, and the influence of the inertial force to piles was likely to be small.

Case 2-1233 (Fig. 11(e)) is the case in which nonlinear level was the largest of all. Like Case 1-924, the bending moment at the time of the peak inertial force took small value relative to the others, though the inertial force took the peak value, and the behavior of the piles was dominated by the ground displacement.

As above, in the cases in which the structure became nonlinear, as nonlinear level of the structure became larger, the effect of the inertial force to the piles got smaller. Thus, the responses of the piles were strongly affected by the ground displacement in these cases. Additionally, the inertial force got out of phase with the ground displacement, and the interaction between the structure and the soil became smaller.

7. COMPARISON OF DYNAMIC NONLINEAR BEHAVIOUR OF RC COLUMN BETWEEN E-DEFENSE TESTS AND SFSI TESTS

The E-Defense tests show the responses of the RC columns on the fixed table, and the SFSI tests show the response of the columns with the foundation and the soil. Fig. 12 shows the comparison of the dynamic nonlinear behaviour of the RC columns between these tests. Since the models are the same specifications and the acceleration at the foundation level are the almost the same, the response of the columns can be compared. The response of the SFSI tests is similar to that of the E-Defense, especially in the dominant period and the amplitude. However, after the major response at 8 seconds, the response of the SFSI test immediately decreases and finally the column stopped the vibration around 14 seconds. This is because of the radiation damping which is the dynamic interaction. Although the additional evaluations are needed, this is one example of the fruitful results collaborating

the different-scale experimental projects.



Figure 12. Comparison of response of RC column between E-Defense tests and SFSI tests

8. CONCLUSION

Based on the shake table tests of structure-pile foundation-soil system in which mass of the weight and the level of input motion were changed, dynamic interaction was evaluated in terms of distributions of bending moment along a pile. The results have shown the following:

- 1. In the case of small input motion, the inertial interaction was dominant in the response of the pile foundation. Increasing the shaking level, the effect of kinematic interaction on the response became major.
- 2. When the structure became nonlinear, the piles' behavior was strongly affected by the ground displacement and the interaction between the structure and the soil became smaller.
- 3. Compared with the dynamic response of RC column between E-Defense tests and SFSI tests, the dynamic response of SFSI tests shows small and high damping because of the dynamic interaction.

ACKNOWLEDGEMENT

This research was supported by Ministry of Education, Culture, Sports, Science and Technology (MEXT), Japan. (Grant-in-Aid for Young Scientists (S)19676004, representative : Yoshikazu TAKAHASHI)

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