Hybrid Vibration Experiment on Seismic Behavior of Bridge-Soil System

by

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ABSTRACT

In order to improve seismic design technology of highway bridges, it is most essential to investigate seismic behavior of a whole bridge system. For this purpose, we have developed a hybrid experiment technique, which integrates numerical response analysis with vibration experiment, and applied it for studying the seismic behavior of highway bridge system including surrounding soils. In our hybrid vibration experiment, we made a pile foundation and surrounding soils as the actual model, which were constructed in a laminar box placed on a shake table, and we made a footing, pier and superstructure as the numerical model. Both non-liquefiable and liquefiable soils were employed to construct the ground model. For the former, we assumed two kinds of highway bridges that have different horizontal capacities of pier, conducted hybrid vibration experiments with those models, and systematically examined the interactive seismic response of bridge pier and pile foundation. With the latter ground model, we studied the influence of liquefaction on seismic response of the highway bridge system.

KEYWORDS: Highway Bridge, Hybrid Vibration Experiment, Liquefaction, Seismic Design

1. INTRODUCTION

In the seismic design of highway bridges, we generally divide a bridge into two parts, i.e., superstructure-pier and foundation. This is mostly for simplicity, however, there exists interaction between them, and studying this interaction is most essential to solve the seismic behavior of whole bridge system, which would contribute to the further development of seismic design technology. Vigorous efforts have been devoted to study this interaction analytically. On the other hand, experimental studies have been rather limited, because they generally require large-scale experiments.

We examined in this study the seismic behavior of highway bridge system that consists of superstructure, pier, foundation and surrounding soils by using hybrid vibration experiment technique, which integrates numerical response analysis with vibration experiment. We conducted experiments with non-liquefiable soils and liquefiable soils, respectively. Based on the experimental results, we investigated the interaction between seismic response of bridge pier and foundation, and the influence of liquefaction on dynamic response of the highway bridge system.

2. OVERVIEW OF HYBRID VIBRATION EXPERIMENT

As illustrated in Fig.1, an original structure is divided into two parts in the hybrid vibration experiment. One is an actual model specimen of original structure. This specimen is usually taken as a part of structure whose seismic behavior is unknown or complicated. The other is a numerical model for vibration response analysis. This model represents a part of structure whose seismic behavior can be evaluated by numerical analysis.

In our hybrid vibration experiment [1], we made a pile foundation and surrounding soils as the actual model, and footing, pier and superstructure as the numerical model. An outline of experimental process is as follows:

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(1) Place a spacer and balance weight on the pile foundation model, and connect an actuator with the balance weight. For the vertical direction, adjust the weight of balance weight so that the total weight of spacer and balance weight corresponds to the dead weight of superstructure, pier and footing that acts on the pile foundation.
(2) Shake the table horizontally with an input motion, and measure the reaction force of model specimen at the boundary of specimen and numerical model. Compute response displacement of numerical model to this reaction force and external force such as inertia force.
(3) Apply the calculated displacement of numerical model to the specimen by the actuator and thus reproduce seismic response of highway bridge system. Note that we ignore the rotational motion in this experiment.

3. PROTOTYPE AND EXPERIMENTAL MODEL

3.1 Bridge on Non-liquefiable Ground [2]

We assumed two kinds of bridge models in this study; Model-1 was designed after the 1971 Seismic Design Guidelines for Highway Bridges in Japan [3] (hereinafter mentioned as "1971 Guidelines"), and Model-2 was designed after the 1996 Design Specifications for Highway Bridges [4] ("1996 Specifications"). The prototype of experimental models is a 30m-span simple girder bridge on the medium soil ground, which is schematically illustrated in Fig.2. The difference between those two experimental models is the horizontal capacity of bridge pier. To realize this, we changed the number and diameter of reinforcing bars of pier between the two models, and the rests were set as the same.

The prototype bridge was reduced to 25% in size to produce an experimental model. Two piles in the longitudinal direction were extracted for the test specimen as shown in Fig.3. According to the number of piles of experimental model, we reduced the external force generated at the boundary of real specimen and numerical model to 25% of the prototype. Since the preliminary objective of this study is to examine nonlinear seismic response of both bridge pier and foundation, we used RC piles for experiments.

The number and diameter of reinforcing bars of a model pile were determined to be consistent with the reinforcement ratio of the prototype pile. The diameter and length of model pile are 300mm and 3.0m, respectively. The pile heads were rigidly connected to the spacer, while their tips were connected to the bottom of laminar shear box by hinges to allow rotation.

We made model ground in a laminar shear box, which was mounted on the shake table. The inner size of laminar box is 3.5m high, 4m wide and 4m long. The ground model used consists of two layers, i.e., 2.5m-thick surface layer and 0.5m-thick lower layer. Both layers were of dry silica sand, and the major physical properties of the sand are as follows: maximum void ratio e_{max}=1.044, minimum void ratio e_{min}=0.616, mean grain size D_{50}=0.172mm and fines content FC=2%. The target N-values, i.e., blow count per foot by standard penetration test, were 7 and 12 for the surface and lower layers, respectively. We adopted compaction control by density when we constructed the ground model. N-value and shear-wave velocity were measured at each stage of experiments by Swedish-sounding test and bender element test, respectively, and the test results are plotted in Fig.4. Although N-value has changed before and after a series of experiments, change of shear-wave velocity is insignificant.

In order to examine the vibration characteristics of experimental model, eigenvalue analysis was carried out, in which the ground and pile foundation were modeled by plane elements, and the footing and pier were modeled by beam elements. Fig.5 shows the first and second natural vibration modes, and the first and second natural frequencies were computed as 8.02Hz and 27.82Hz, respectively. We see from this figure that the footing and superstructure vibrate in phase for the first mode and they vibrate out of phase for the second mode.

3.2 Bridge on Liquefiable Ground

The prototype highway bridge on the liquefiable ground was designed after the 1996 Specifications. The experimental model was essentially same with Model 2 for the non-liquefiable ground. Fig.6 illustrates an overview of the experimental model. The ground model was 3m thick one-layer saturated silica sand. We constructed liquefiable and non-liquefiable grounds by adjusting the relative density of soils as 40% and 88%, respectively. The liquefiable loose ground was produced by boiling the sands. The non-liquefiable ground was built so that the relative density of soils was the target value by shaking the loose ground model on the shake table.

4. VIBRATION RESPONSE ANALYSIS

The numerical model consists of structural elements (mass, damping and stiffness matrices), external force that is calculated from the acceleration of shake table, and reaction force generated at the boundary of the actual and numerical models. In the numerical analysis, the external and reaction forces are inputted, and the displacement of actual model for the next time step is calculated. This displacement is realized by an actuator. Then, the external and reaction forces are measured and taken into numerical analysis. Iterating these procedures, the seismic behavior of original structure can be accurately simulated. The equation of motion for numerical analysis may be described as

$$M\ddot{x} + C\dot{x} + Kx = p + q \tag{1}$$

where

M: Mass matrix
C: Damping matrix
K: Stiffness matrix
x: Relative displacement vector
p: External force (seismic response) vector
q: Reaction force vector.

Using Eq. (1), the vibration response (displacement vector x) after a short interval Δt can be calculated from the measured reaction force vector q and the external force vector p. The central difference method is employed in vibration response analysis, because it requires short time to generate actuator signal for the next time step after measuring reaction force. Time required for one cycle process is 2.08ms [5].

As the numerical model, we assume a 2-degree-of-freedom system consisting of mass of footing, and that of pier and superstructure. Fig.7 shows the force and displacement relationship of pier, which is idealized as a bi-linear system.

5. EXPERIMENTAL METHOD

5.1 Bridge on Non-liquefiable Ground

Although actuator response delay has unfavorable influence on the hybrid vibration experiment, it is inevitable with a hydraulic actuator. Therefore, a compensation technique was adopted for the experiments with non-liquefiable and liquefiable grounds in common. This technique predicts the displacement of an actuator at the time after actuator delay time [6].

As the input motions for experiments, we used sinusoidal waves with frequencies corresponding to the first natural frequency of experimental model and the mean of first and second natural frequencies. Also employed was the strong motion record obtained at the Kobe Maritime Observatory, Japan Meteorological Agency during the 1995 Hyogo-ken Nanbu (Kobe) earthquake. This record was converted to the surface of base layer of the site, which will be referred as "JMA record" in this paper. The time axis of JMA record was compressed to 23.5% of the original record based on the ratio of first natural frequencies of prototype bridge (1.89Hz) and experimental model (8.02Hz), and the peak accelerations were adjusted to 0.49G and 0.07G, which correspond to 70% and 10% of the peak accelerations on the surface of base layer. To secure stability of the hybrid vibration experiment, we expanded the time axis of input motions three times as long as the original time axis. Table 1 summarizes the experimental cases.

5.2 Bridge on Liquefiable Ground

We employed sinusoidal waves for the experiments with liquefiable soils. By utilizing the advantage of hybrid vibration experiment, we variously changed the property of bridge pier. We assumed both linear and bi-linear systems for the bridge pier. In addition to this, we also assumed the retrofitted bridge pier, for which both the initial rigidity and yield displacement were set as 1.2 times of the original bridge pier. The experimental cases are shown in Table 2.

6. EXPERIMENTAL RESULTS

6.1 Bridge on Non-liquefiable Ground

Relationships between horizontal force and displacement of pier are plotted in Fig.8 for Cases 2-1 and 2-2. As seen from this figure, the pier of Model-1, which was designed after the 1971 Guidelines, shows plastic behavior, while the pier of Model-2, which was designed after the 1996 Specifications, remains elastic. Figs.9 and 10 compare the maximum acceleration and pile curvature for Cases 2-1 and 2-2, respectively. Acceleration response of Model-1 is a little smaller than that of Model-2, whereas the distributions of pile curvature are almost similar between these two models except the intermediate part of pile. It seems within the scope of present study that the acceleration response is affected by the horizontal capacity of pier, while bending moment is rather insensitive to it.

Relationships between horizontal force and displacement of pier are shown in Fig.11 for Cases 3-1 and 3-2. The bridge pier behaviors plastically in Case 3-2, where 70 % amplitude of JMA record was inputted. Fig.12 compares the distributions of pile curvature. Also plotted is the seven times of the curvature obtained in Case 3-1. The maximum curvature for Case 3-2 at the intermediate part of pile is larger than the seven times of Case 3-1, which suggests that the pile plasticizes around this depth. Fig.13 presents the distributions of maximum acceleration for Cases 3-1, 2-1 and 3-2. These three cases correspond to the followings; both pier and pile remain elastic (Case 3-1), pier becomes plastic, while pile remains elastic (Case 2-1), and both pier and pile become plastic (Case 3-2). Regarding pile response, Case 2-1 in which the pier has become plastic yields the largest value. Case 3-2, in which the pile behaviors plastically, develops small pile response. As for pier response, Case 3-1 produces the largest, and Case 2-1 yields the smallest acceleration. Note that Case 2-1 yields the largest pile acceleration. Case 3-2 locates somewhere between Cases 2-1 and 3-1. Comparison of Cases 2-1 and 3-2 indicates that nonlinearity of pile may affect the seismic response of pier.

6.2 Bridge on Liquefiable Ground

Fig.14 shows the time histories of excess pore water pressure ratio, which were observed 0.95m beneath the spacer. The excess pore water pressure ratio is generally less than 0.5 for Case 3, in which the relative density of the ground was adjusted as 88%. This fact signifies that the ground did not liquefy in this case. While it reached 1.0 after a few cycles from the beginning and maintained this level during the excitation for Case 4, in which the relative density was 40%.

Fig.15 plots the acceleration time histories recorded 0.95m beneath the spacer for Cases 3 and 4. Accelerations on the shake table, which are the input motions of experiments, are also plotted in this figure. In Case 3 the ground acceleration 0.95m beneath the surface is almost similar to the input motion. By contrast, the ground acceleration in Case 4 decreases quickly after a few cycles from the beginning. This is harmonic with the change of excess pore water pressure ratio presented in Fig.14.

Fig.16 compares shear force at pile head, accelerations at footing and superstructure for Cases 3 and 4. Note that the bridge pier is assumed to be elastic and the input motion is sinusoidal wave with frequency corresponding to the second natural frequency of the bridge system in both cases. The shear force at pile head, accelerations at footing and superstructure are almost identical between those two cases at the beginning of experiment when the ground did not liquefy. While after the ground liquefied, the shear force and accelerations in Case 4 become smaller than those in Case 3. This may be attributed to the fact that the ground acceleration decreases in Case 4 as indicated in Fig.15 and the seismic response of bridge pier and superstructure becomes small. The frequency of input motion coincides with the second natural frequency of the bridge system, and the vibration of bridge pier and superstructure is predominant in this second vibration mode.

7. CONCLUSIONS

We have developed a hybrid vibration experiment technique, and applied it for studying seismic behavior of highway bridge system. Experiments were performed for highway bridge on the non-liquefiable ground and liquefiable ground, respectively. Based on the experimental results, we examined the interaction between seismic response of bridge pier and foundation, and the influence of liquefaction on dynamic response of bridge system. Main conclusions of the present study may be summarized as follows:

(1) According to the experimental results with two different highway bridge models, it seems that the acceleration response is affected by the horizontal capacity of pier, while bending moment of pile is rather insensitive to it.

(2) The generation of plasticity in bridge pier or pile foundation may affect the mutual seismic response. For example, large pile acceleration was observed when the pier had become plastic. Pier acceleration decreases when the pier plasticizes, in which the decrease rate is small when the pile also becomes plastic.

(3) The occurrence of liquefaction decreased the seismic response of both foundation and superstructure within the scope of the present study. This may be attributed to the fact that the input motion employed develops the second natural vibration mode, in which the vibration of bridge pier and superstructure is predominant, the seismic response of bridge pier and superstructure decreases after the ground liquefies, and this contributes to reduce dynamic response of the whole bridge system.

8. REFERENCES

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Table 1 Experimental cases for non-liquefiable ground

Input motion	Frequency	Bridge model*	Peak acceleration	Case No.
Sinusoidal wave	8.1 Hz	Model 1	0.05G	1
	18 Hz	Model 1	0.4G	2-1
		Model 2	0.4G	2-2
Seismic wave	JMA record	Model 1	0.07G	3-1
			0.49G	3-2

*Model 1 and Model 2 were designed after 1971 Guidelines and 1995 Specifications, respectively.

Table 2 Experimental cases for liquefiable ground

Case No.	Input motion		Bridge pier	Relative density	Natural frequency		Ground model
	Peak accel.	Frequency			1st mode	2nd mode	
C3	0.6G	36 Hz	Elastic	88%	9.99 Hz	36.9 Hz	Non-liquefiable
C4	0.6G	36 Hz	Elastic	40%	7.53 Hz	35.6 Hz	Initial
					3.57 Hz	34.6 Hz	Liquefied
C5	0.6G	36 Hz	Bi-linear	88%	9.99 Hz	36.9 Hz	Non-liquefiable
C6	0.6G	36 Hz	Bi-linear	40%	7.53 Hz	35.6 Hz	Initial
					3.57 Hz	34.6 Hz	Liquefied
C8	0.6G	39 Hz	Retrofitted Bi-linear	40%	7.59 Hz	38.8 Hz	Initial
					3.57 Hz	37.8 Hz	Liquefied

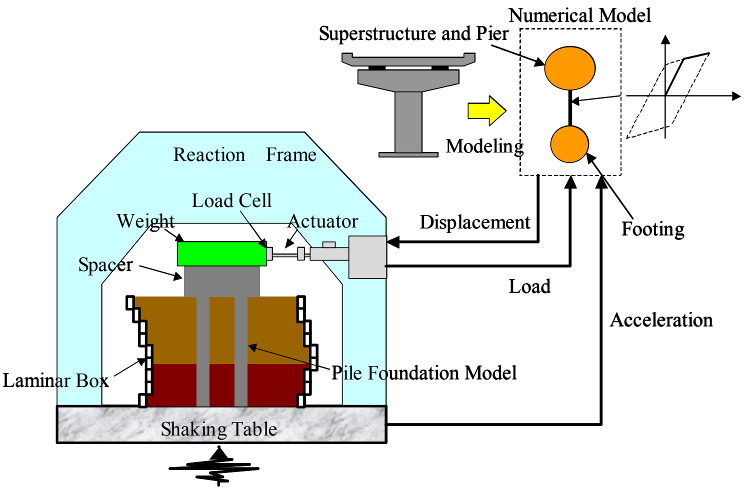

Fig.1 Conceptual view of hybrid vibration experiment

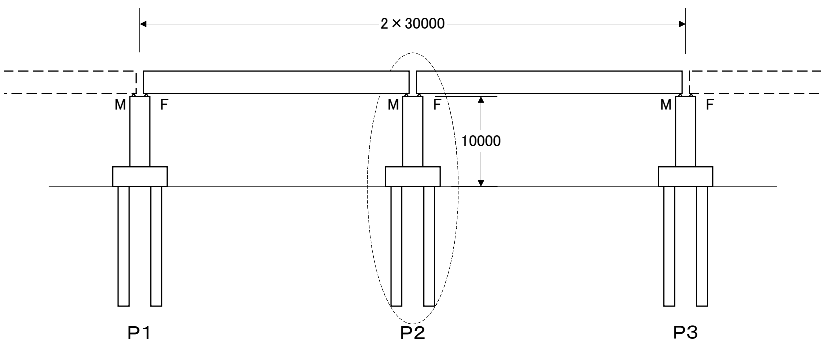

Fig.2 Schematic view of prototype bridge

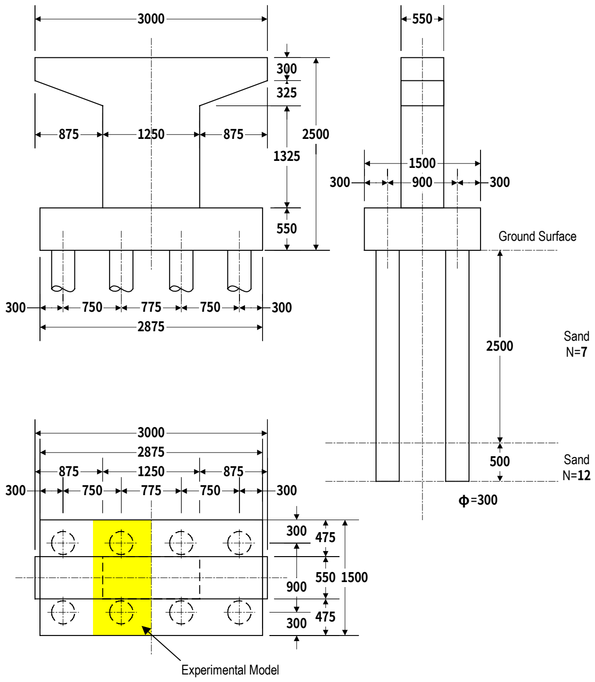

Fig.3 Overview of experimental model on the non-liquefiable ground

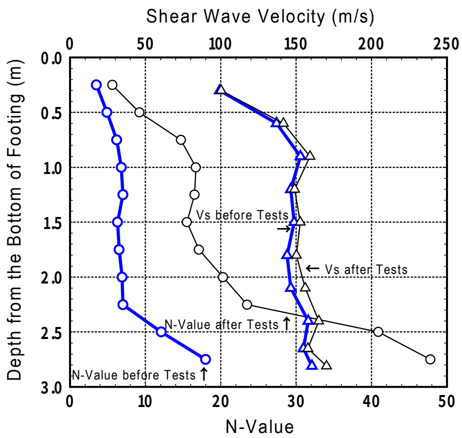

Fig.4 N-value and shear-wave velocity of ground model

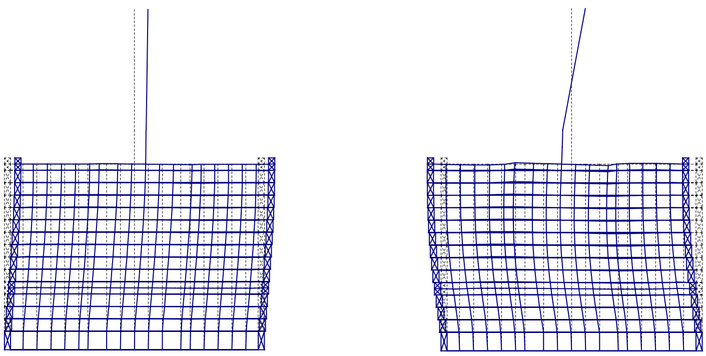
(a) First vibration mode (b) Second vibration mode

Fig.5 Natural vibration mode

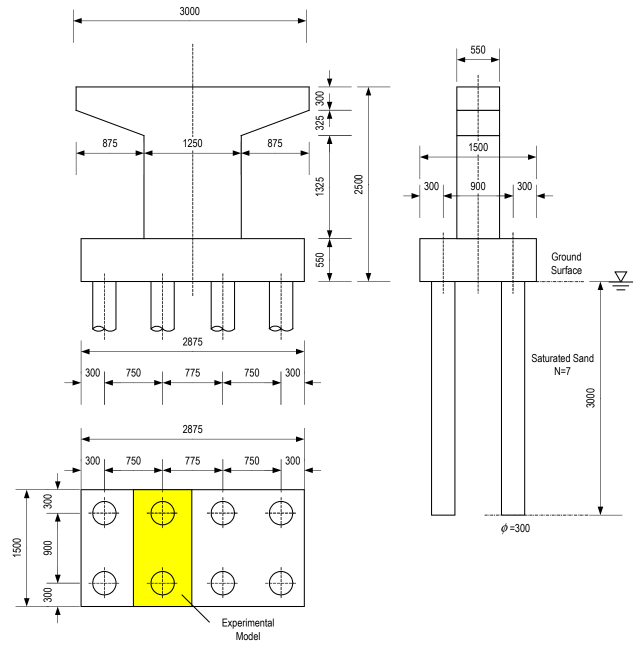

Fig.6 Overview of experimental model on the liquefiable ground

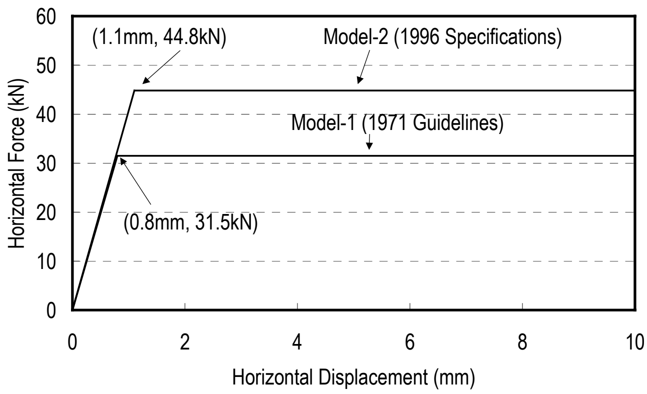

Fig.7 Horizontal force-displacement relationship of pier

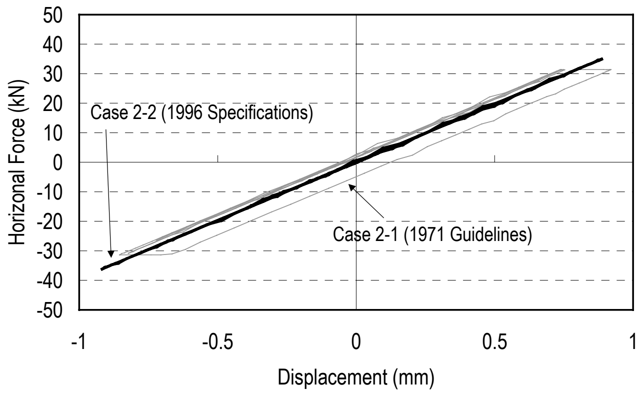

Fig.8 Horizontal force-displacement relationship of pier (Cases 2-1 and 2-2)

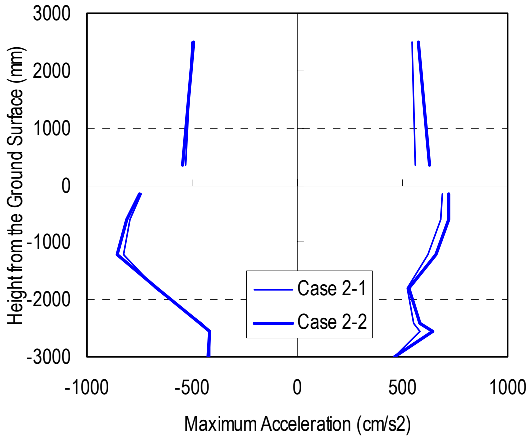

Fig.9 Maximum acceleration distribution (Cases 2-1 and 2-2)

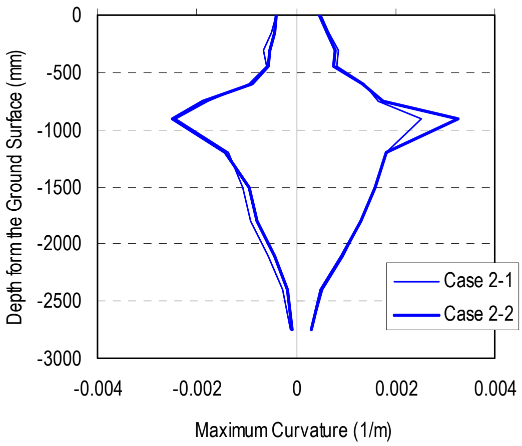

Fig.10 Maximum curvature distribution (Cases 2-1 and 2-2)

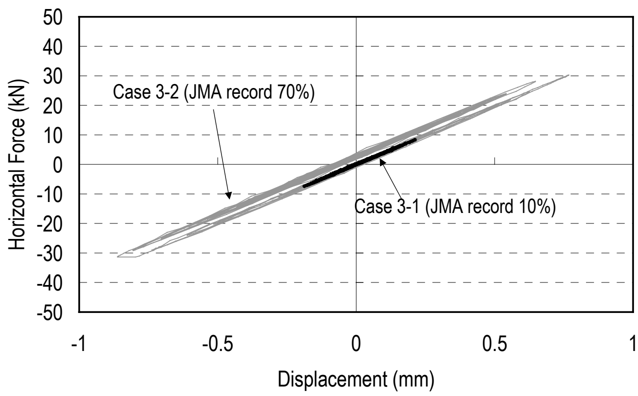

Fig.11 Horizontal force-displacement relationship of pier (Cases 3-1 and 3-2)

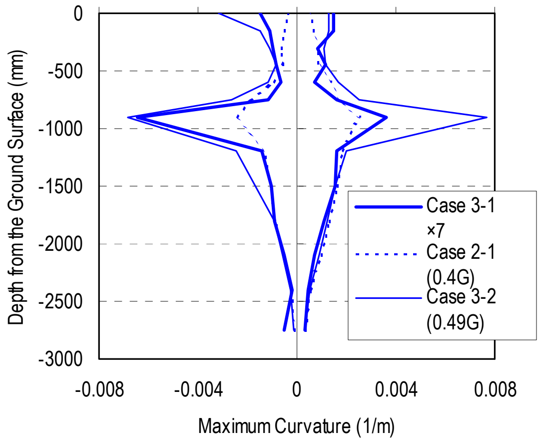

Fig.12 Maximum curvature distribution (Cases 3-1, 2-1 and 3-2)

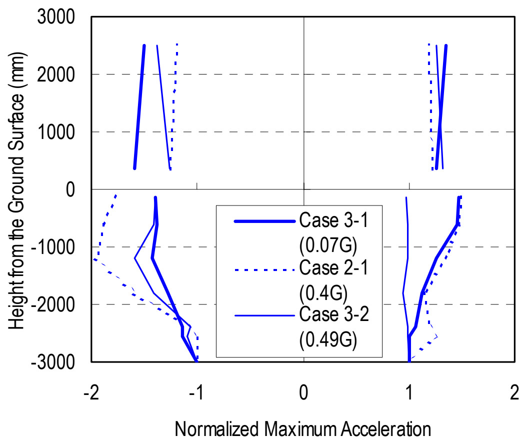

Fig.13 Maximum acceleration distribution (Cases 3-1, 2-1 and 3-2)

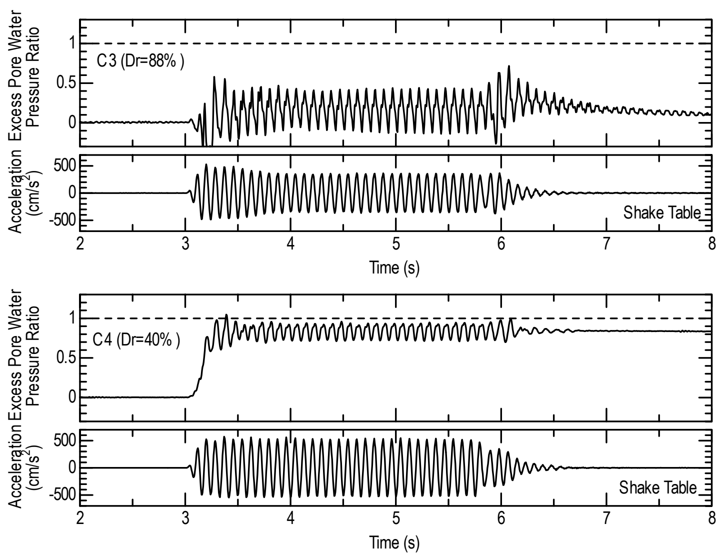

Fig.14 Excess pore water pressure ratio at 0.95m beneath the surface (Cases C3 and C4)

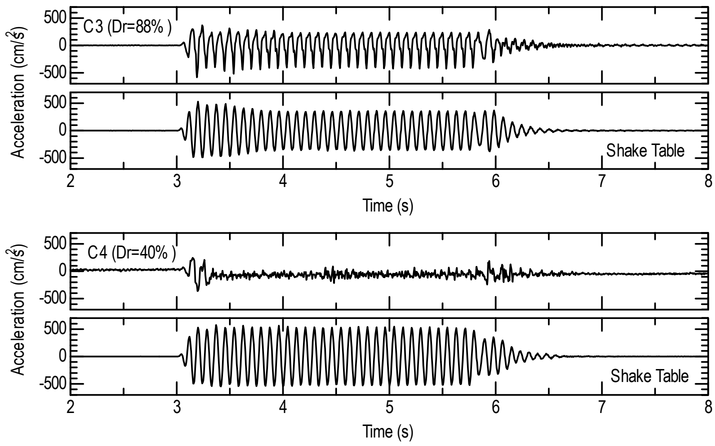

Fig.15 Acceleration at 0.95m beneath the surface (Cases C3 and C4)

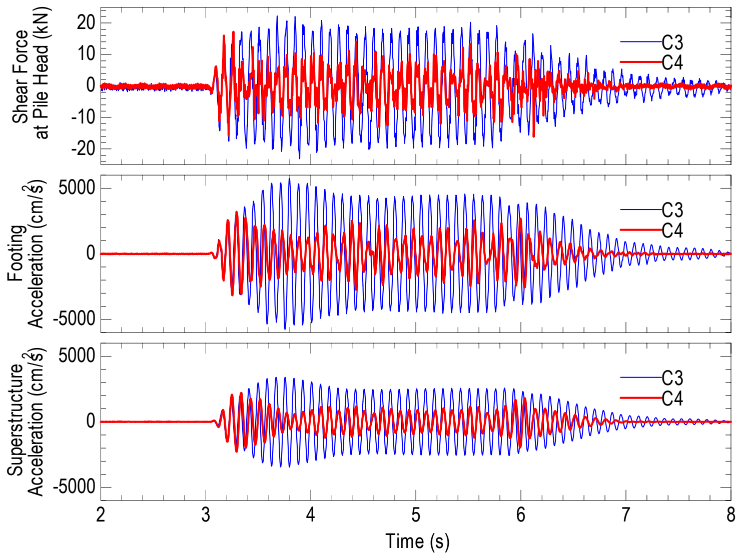

Fig.16 Shear force at pile head, accelerations at footing and superstructure (Cases C3 and C4)