- Technical Paper -

RESPONSE BEHAVIOR OF RC BRIDGE PIERS USING PSEUDO-DYNAMIC TEST INCLUDING GROUND

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ABSTRACT

This study presents the development of a pseudo-dynamic testing system of RC bridge piers under a strong earthquake including the effects of ground. The model used for the pseudo-dynamic test was 3-DOF system consisting of two lateral displacements, one at pier top, and the other at footing, together with 1 rotation of footing. In order to incorporate the interaction with ground, a sway spring and a rocking spring were attached to the mass of footing. A pseudo-dynamic test was conducted considering the effect of ground on two bridge piers having different stiffness and yield displacement.
Keywords: pseudo-dynamic test, soil-structure interaction, earthquake, RC bridge piers

1. INTRODUCTION

The significance of interaction between structures and ground has been clearly evident in the Kobe earthquake [1]. Several studies have indicated the need of considering the interaction between ground and the bridge pier. The main topic of concern is the increase in the ductility demand of piers due to the effect of soil-structure interaction (SSI) [2].

After an earthquake, many RC bridge piers have been reconstructed or strengthened to a greater capacity in order to achieve both higher capacity and ductility. However, the behavior of these strengthened bridges under the next coming earthquake is still poorly understood. With an enhancement in the capacity of pier, the position of failure may shift from the pier down to the foundation. Therefore, a study on the behavior of strong RC bridge piers including effect of SSI is indispensable.

The study on the seismic behavior of bridge pier considering soil-structure interaction can be conducted by either analysis or experiment. In analytical study, it is possible to model the full size bridge pier as there is no size limitation. However, the detail modeling of several parts of the bridge such as joints and connecting elements is difficult as their behavior is still poorly understood. In contrary, experiment allows one to test a structure without fully understanding its behavior. However, in the experimental study the size of testing specimen is always controlled by the capacity of testing system.

In this context, pseudo-dynamic test is appealing as it combines the merits of both the experimental and analytical study. Generally speaking, pseudo-dynamic (PSD) test is a numerical time integration of the equation of motion. The behavior of structure is assumed to be described by a simple structural model. The mass and damping matrices as well as force vector are formulated according to a selected simple model, and the restoring force is evaluated from specimen during a test.

In this study, the pseudo-dynamic test on bridge pier system including ground and piles was conducted using a 3-DOF model. The restoring force of pier was obtained by the loading test on a scaled down pier specimen. On the other hand, the restoring effect from ground was modeled by sway and rocking springs. Hardin-Drnevich model [3] was used for the sway spring whereas the linear model was used for the rocking stiffness.

2. PSD TEST ON RC BRIDGE PIER SYSTEM

2.1 Structural Model and Equation of Motion

3-DOF model was used to represent the RC bridge pier with the effect of soil-structure interaction in this study as shown in Fig. 1. The model consists of two masses and one moment of inertia, together with three stiffness springs. In this study, the viscous damping of structure was neglected.

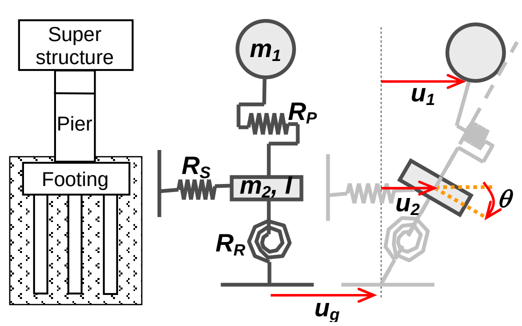

Fig. 1 The 3-DOF model used in PSD test

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The equation of motion of this 3-DOF model can be formulated in the following matrix form

$$M\ddot{u} + R = P \tag{1}$$

when,

$$M: \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & I \end{bmatrix}, \quad R: \begin{bmatrix} R_P \\ R_S - R_P \\ R_R - R_P H \end{bmatrix}$$

$$u: \begin{Bmatrix} u_1 \\ u_2 \\ \theta \end{Bmatrix} = \begin{Bmatrix} u_P + u_S + u_R H \\ u_S \\ u_R \end{Bmatrix}$$

$$P: \begin{Bmatrix} m_1 \\ m_2 \\ 0 \end{Bmatrix} \ddot{u}_g \quad , \ \ddot{u}_g : \text{Ground Acceleration}$$

m_1 : Mass of superstructure including 30% mass of pier
m_2 : Mass of footing
I : Moment of inertia about the weak rotational axis of the footing which belongs to the mass of pier and footing
H : Pier height
R_P : Pier restoring force,
R_S : Lateral (sway) restoring force from foundation
R_R : Rotational (rocking) restoring force from foundation
u_1 : Lateral displacement at pier top (Global)
u_2 : Lateral displacement of footing (Global)
θ : Rotation of footing (Global)
u_P : Pier spring deformation (Local)
u_S : Sway spring deformation (Local)
u_R : Rocking spring deformation (Local)

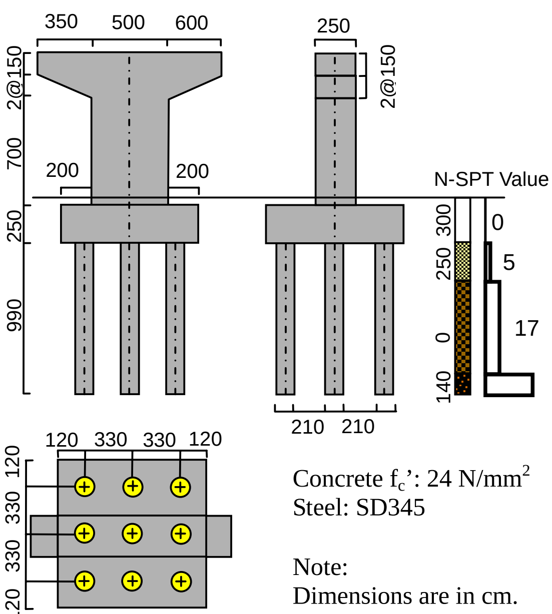

Fig. 2 Bridge system configuration

2.2 PSD Test of Bridge Pier System

Fig. 2 shows the bridge pier system used in this study. The vibration in only weak direction of pier was considered. The bridge pier system was modeled by the 3-DOF system as already shown in Fig. 1.

In PSD test, the average acceleration Newmark's numerical time integration scheme was performed using the Operator Splitting (OS) Technique [3] to solve the equation of motion. The value of each mass was estimated. And for restoring force, only the pier restoring force was obtained from the experiment on fixed base pier with footing specimen, whereas, both sway and rocking restoring forces were calculated from the load-displacement relationship models, which will be further discussed in the next section.

2.3 The Load-Displacement Model of Ground DOF

The detail of foundation is also shown in Fig. 2. The foundation was modeled by beam-spring model as shown in Fig.3. The model contains a rigid element representing the footing, three beams elements as the three rows of piles and also several springs along the pile length representing the restoring force from soil. This beam-spring model was analyzed statically up to failure using 2D analysis software in order to obtain monotonic load-displacement relationship of foundation. The ultimate condition was set as the strain at the extreme fiber of beam element (pile) reaches 0.0035

Tri-linear model was used as the moment-curvature relationship of beam elements in the modeling each row of piles. The yielding moment of beam element was assumed to be triple times of a single pile; however, corresponding to the same yield curvature. The yield point of the beam elements was evaluated using fiber model technique.

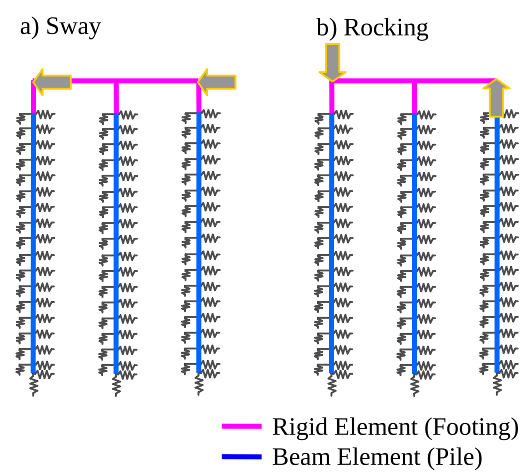

Fig. 3 Foundation model used the calculation of load-displacement relationship in soil springs

For the restoring force provided by soil, each soil spring was modeled with bi-linear load-displacement relationship, carrying constant force after yield. The stiffness value of each spring, which represents the soil resisting force along the surface of pile, is calculated from N-SPT (Standard Penetration Test) values of each stratum. The lateral springs along the pile length were modeled to provide restoring force when soil is in compression only. The friction force (vertical spring) gives resisting force in both tension and compression. And at pile tip, bearing resisting force is provided when soil is in compression; whereas, lateral resistant is given in both directions. Further detail on the calculation of spring stiffness could be found in [5].

Monotonic load-displacement relationships of the foundation models, for both lateral direction and rotation, were obtained by applying the load configuration as also shown in Fig. 3, and then extended to become reversed by fitted with models. For sway spring, Hardin-Drnevich model (HD-model) was used as reversed load-displacement relationship, with the monotonic load-displacement relationship as envelop curve. The load-displacement relationship of the sway spring (HD-model) is shown in Fig. 4. On the other hand, linear-elastic model was used in the rotation (rocking) spring.

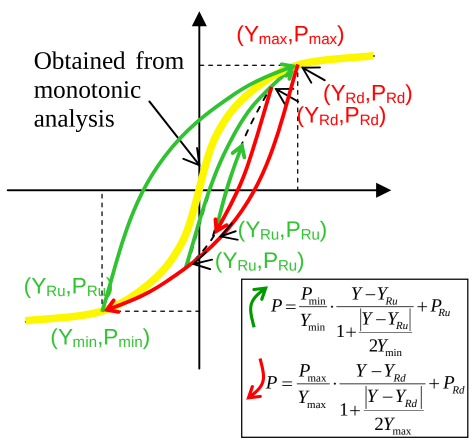

Fig. 4 HD-model used in sway spring

2.4 Experimental Study

The PSD test of RC bridge pier system in Fig. 2 was conducted on 2 pier cases, A and B, having different stiffness and yield point. The weaker pier (Case A) represents a normally designed pier, while, the stronger pier (Case B) was selected based on the pier of Case A with 10 mm. steel jacketing. The steel jacket applied in Case B was assumed to fully enhance the flexural capacity of pier. The input ground acceleration was the recorded Kobe, 1995 earthquake.

Due to the limitation of the capacity of the available actuator, the testing specimens were scaled down by force and displacement scaling factors with the following conditions,

$$P_{yR} \times SF_p = P_{yS}$$
$$d_{yR} \times SF_d = d_{yS} \qquad (2)$$

where,

P_{yR} : yield load at pier top (full size)
P_{yS} : yield load at pier top (scaled size)
d_{yR} : yield displacement (full size)
d_{yS} : yield displacement (scaled size)
SF_p: load scaling factor
SF_d: displacement scaling factor

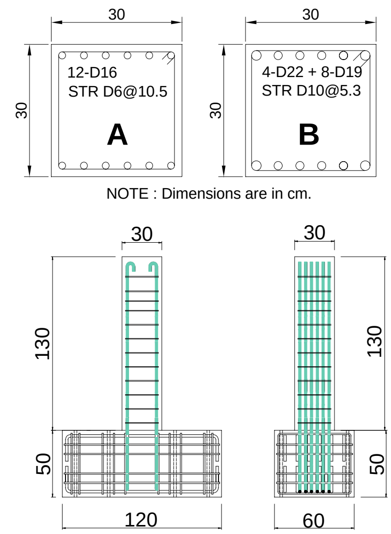

Fig. 5 Scaled down specimens

It is noted that the stiffness of each specimen was also automatically scaled down due to Eq. 2. In order to determine the scaling factors, yielding load and displacement of the full size piers were calculated according to fiber model, while ones of the scaled down specimens were obtained from reversed cyclic test. The details of full size piers are shown in Table 1 and 2. The scaled down specimens are shown in Fig. 5.

Fig. 6 shows the experimental set up. All the masses, forces, as well as the stiffness, used in the numerical time integration scheme, belong to the full size system.

In order to verify the stability of PSD test, analysis for both cases using bi-linear model as the load-displacement relationship of piers was compared with the obtained PSD test results. The bi-linear model used in evaluating the pier restoring force is shown in Fig. 7.

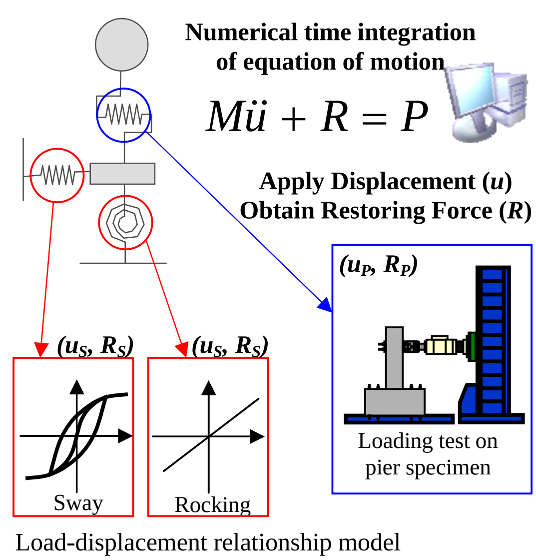

Fig. 6 Experimental set up of PSD test

Table 1 Details of case A (full size)

Dimension (cm)	Flexural Steel		Factor	
			Force	Disp
500x250	410 cm^2		0.2296	0.0202
Mass	m_1 (kg)	m_2 (kg)	I (kg-m^2)	
	1,077,580	486,000	13,074,690	
Stiffness (Initial)	k_{pier}	k_S	k_R	
	1.40x10^8	2.89x10^8	1.94x10^{14}	
Yield	P = 5,503 kN, d = 3.90 cm			

Table 2 Details of case B (full size)

Dimension (cm)	Flexural Steel		Factor	
			Force	Disp
500x250	1914 cm^2		0.4126	0.0202
Mass	m_1 (kg)	m_2 (kg)	I (kg-m^2)	
	1,077,580	486,000	13,074,690	
Stiffness (Initial)	k_{pier}	k_S	k_R	
	2.86x10^8	2.89x10^8	1.94x10^{14}	
Yield	P = 9,001 kN, d = 3.15 cm			

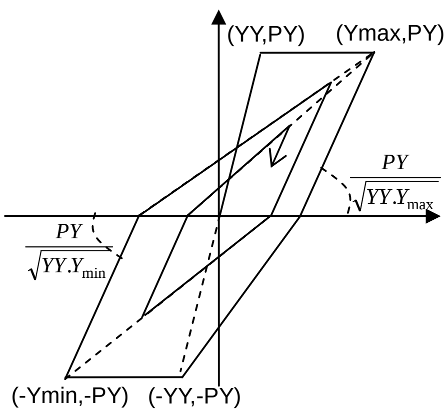

Fig. 7 Bi-linear model used in the analysis of pier

3. RESULTS AND DISCUSSION

The PSD test results of both pier cases are compared with their corresponding analysis as shown in Fig. 8. The PSD test results exhibit a good agreement with the analysis. Therefore, the stability of PSD test is acceptable.

It should be noted at first about the means of obtaining the responses in this PSD test in order to avoid confusion. Only the deformation and restoring force of pier spring (u_P and R_P) were obtained from the experiment; however, they were finally adjusted according to Operator Splitting Method. This makes the hysteresis curve of pier in analysis not exactly bi-linear. The other displacement and restoring force for both sway and rocking springs (u_S, R_S, u_R, R_R) were calculated according to the load-displacement relationship discussed in section 2.3.

The peak response of pier top displacement is the summation of the displacements of three springs according to the relationship in Eq. 1. In both pier cases, the peak values of pier top displacement are not so different, which are around 10 cm. On the other hand, the peak displacement of sway spring in case of stronger pier (Case B) is observed 65% greater than that of weaker pier (Case A). And also, the difference in the peak of 47%, greater in Case B, is observed in the rotation of rocking spring. From the static analysis of foundation model, as mentioned in section 2.3, the lateral (sway) displacement of foundation that coincides with the yield and ultimate point of a single pile are 3.43 cm. and 5.60 cm. respectively. From Fig. 8, the peak displacement of sway spring (lateral displacement of foundation) in Case A falls in the range between static yielding displacement and ultimate displacement, whereas, the peak displacement of sway spring in Case B is greater than the static ultimate displacement.

Fig. 9 compares the hysteresis curve of each spring in Case A and Case B. For pier, the pier of Case A experienced maximum displacement about 2.5 times yield displacement, whereas, the pier of Case B has just of touched the yield point. On the contrary, the hysteresis curves of foundation springs (sway and rocking) in Case B have faced the wider path range than that of Case A. It should be emphasized that, in both cases, the pier posses the same mass and also subjected to the same ground acceleration.

The work done by strain energy for each restoring spring, calculated from area under the plot of Fig. 9, is also plotted with time in Fig. 10. It is noted that the structure also contains kinetic energy at all steps. In both pier cases, the sway spring, Fig. 10(b), gives the dominant contribution to the energy dissipation of system as it undergoes inelastic since the start of experiment. On the other hand, because of its elastic model, the rocking spring, Fig. 10(d), does not dissipate energy. Comparing two pier cases, the work done by strain energy of the weaker pier is not much higher than that of the stronger pier case, Fig. 10(b). However, a greater difference is observed in sway spring, Fig. 10(c).

From the results, with a bridge having a stronger pier or strengthened with enhanced flexural capacity, the larger portion of load is carried by the foundation. This reveals that, with an enhanced pier capacity, the engineer should also take a better care on the capacity of foundation. A greater load and energy is transferred into the foundation instead of being carried by pier stiffness or dissipated as plastic deformation of pier. Therefore, the capacity of foundation in bridges with strengthened piers should be investigated in order to check the possibility of foundation failure in the future earthquakes. However, this study has just investigated the case of highest severity, by assuming fully enhanced flexural capacity of pier provided by strengthening. The damping, which could be count positive, has been neglected in this study. Therefore, further study on the benefit of including the damping to the foundation response is required as to ensure the seriousness of the problem.

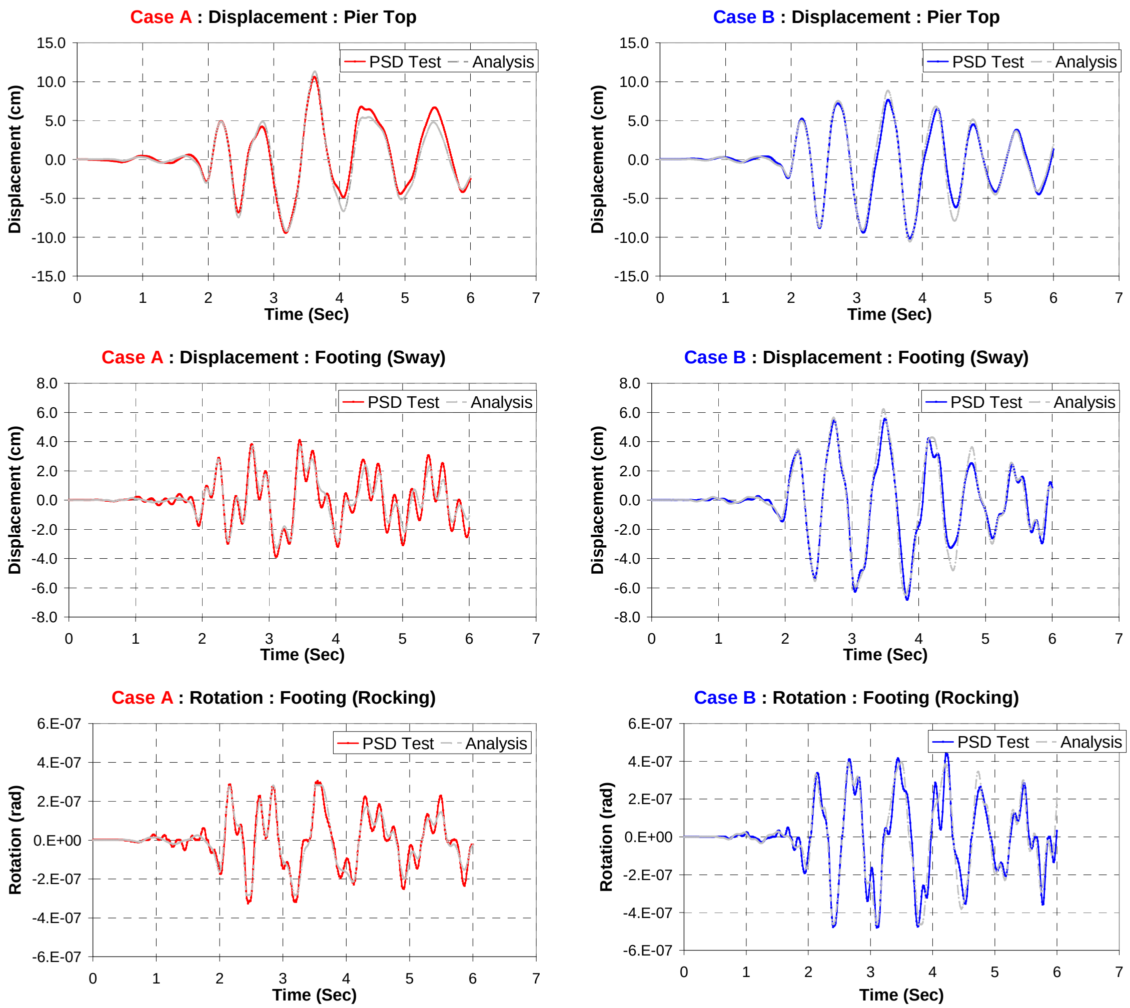

Fig. 8 Displacement results of Case A and Case B

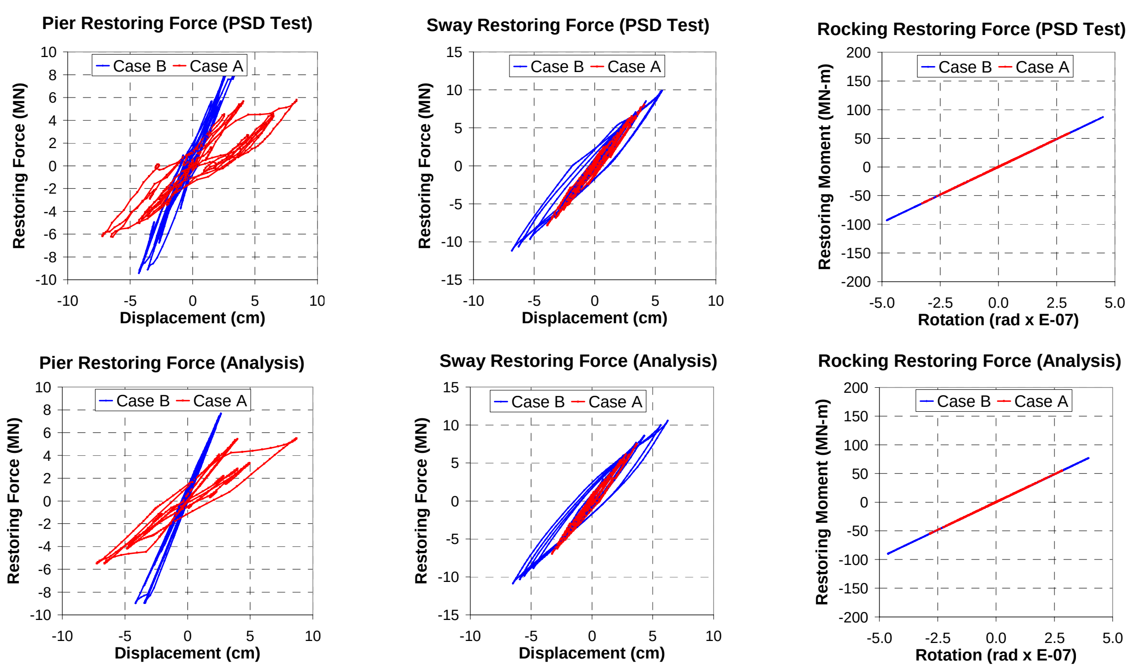

Fig. 9 Load-displacement hysteresis of Case A vs. Case B - PSD test (Upper) and Analysis (Lower)

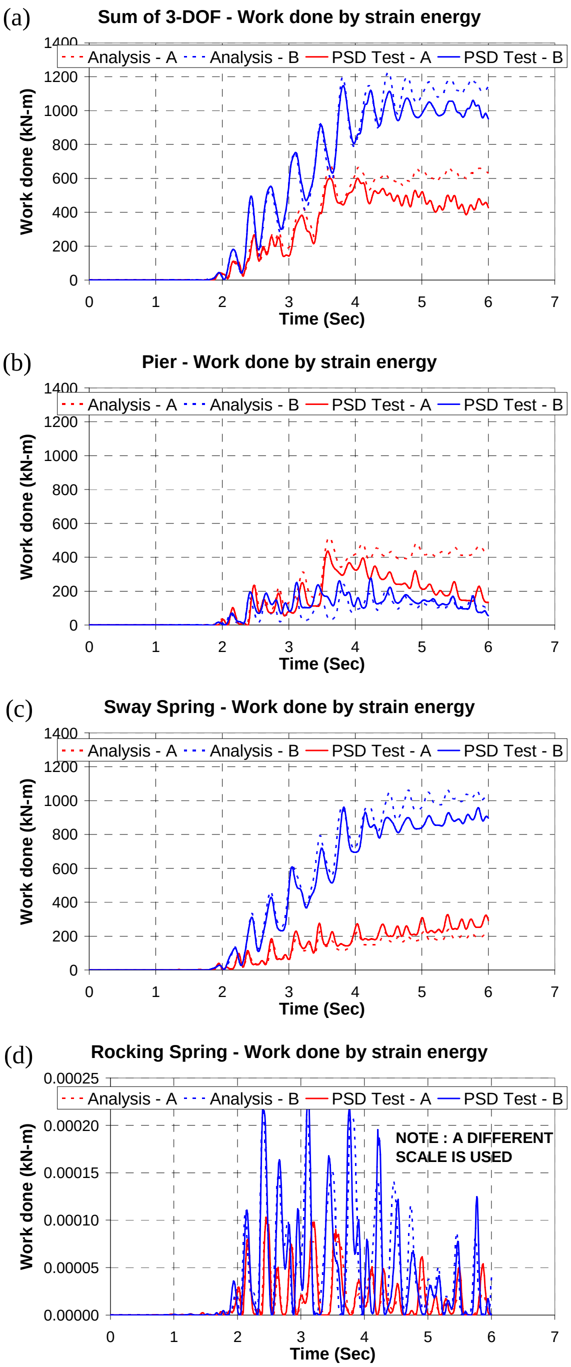

Fig. 10 Work done by strain energy

4. CONCLUSIONS

(1) The PSD test system for the behavior of RC bridge pier including the effects of soil-structure interaction was developed using a 3-DOF model.

(2) With the same mass, soil condition and ground acceleration, the foundation belonging to a pier with a higher stiffness and yield load (strengthened with flexural capacity enhanced) will subject to a greater load during an earthquake. Therefore, the possibility of failure to occur at foundation should be investigated and prevented.

REFERENCES

[1] Mylanokis, G. et al.: The Role of Soil on the Collapse of 18 piers of Hanshin Expressway in the Kobe Earthquake, 12WCEE, 1074(1-7) , 2000

[2] Mylanokis, G. and Gazetas, G.: Seismic Soil-Structure Interaction: Beneficial or Detrimental?, J. of Earthquake Engineering, Vol.4, No.3, pp. 277-301, 2000

[3] Hardin, B. O. and Drnevich, V. P.: Shear Modulus and Damping in Soils: Design Equations and Curves, Proc. ASCE, SM7, pp.667-692, 1972

[4] Nakashima, M. et al.: Integration Method Capable of Controlling Experimental Error Growth in Substructure Pseudo Dynamic Test, J. of Struct., Constr. Eng., AIJ, No.454, Dec., pp. 61-71, 1993

[5] Ministry of Transportation, Railway Division: Design Guideline and Commentary for Railway Structures (Foundation and Pile Structures), Maruzen, pp. 208-222, Jun. 2000 (Japanese)