



## **THE POWERFUL TMD FOR BRIDGES COMPULSORILY SHAKEN BY A REACTION FORCE FROM AN ABUTMENT**

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### **ABSTRACT**

A conventional TMD-type device does not necessarily give sufficient control force as the seismic control device for bridges. In order to highly improve damping capability of a conventional TMD, the Powerful TMD is newly developed. In this device, a TMD is compulsorily shaken by a reaction force from an abutment without any external energy. In this paper, the outline of the new device is introduced in the first step. Modeling, formulation and optimum tuning technique for the device are described next. Shaking table tests using a simple bridge model, to which the proposed device can be applied, are carried out, and the result of shaking table tests for a case in which the proposed device is applied is compared with other cases in which no control device is applied and a conventional passive TMD device is applied, both experimentally and numerically. Then, the effectiveness and the applicability of the Powerful TMD are examined.

### **KEYWORDS**

TMD-type seismic control device; Bridge; Formulation; Shaking table test; Numerical Simulation.

### **INTRODUCTION**

The fundamental theory and optimum tuning technique of a TMD for harmonic vibration were presented by Den Hartog (Den Hartog, 1956). Then, they were established, having been extended to various vibration conditions (Warburton, 1982). TMD-type devices have been recently applied to many civil engineering structures. Most of them are, however, adopted for structural control against wind vibration. The application of this passive device to structures for the purpose of seismic control is limited, because the device is not capable of following initial rapid motion and mass of the device should be large enough to control structural vibrations due to strong earthquake excitation. In order to highly improve damping capability of a TMD so as to be applied to seismic control for bridges, the Powerful TMD was developed. This new device adopts a principle of a lever, and is compulsorily shaken by a reaction force from an abutment without any external energy. Formulation of equation of motion, optimum tuning and numerical simulations were conducted for rigorous 2-DOF system (Kaneko et al., 1994). Then, the effectiveness of the proposed device were verified.

This paper focuses on the Powerful TMD for a simple 2-DOF system developed for design purposes. The mechanism of the new device is described first. Modeling, formulation and optimum tuning technique are introduced next. Then, the effectiveness are verified by vibration tests using a shaking table and numerical simulations.

## OUTLINE OF THE POWERFUL TMD

Fig. 1 illustrates a schematic diagram to represent a function of the Powerful TMD. The device should be located at the end of a bridge deck. Although the concept of the device is based on a conventional passive TMD, a TMD system is not simply fixed at a bridge deck directly but at a steel frame which is able to freely slide on a bridge deck in the longitudinal direction. The steel frame is connected to an abutment through a lever. Letting an arm ratio of the lever be  $1 : \alpha$  and the relative displacement between a bridge deck and an abutment due to earthquake excitation be  $u$  as shown in the figure, the TMD is compulsorily shaken with an amplitude of  $(1+\alpha)u$ , which is  $(1+\alpha)$  times larger than that of a bridge deck. When the control force,  $P$  is given by the TMD excitation, it is transmitted to a bridge deck being amplified by  $(1+\alpha)$  times. Therefore, we can obtain a seismic control force  $(1+\alpha)$  times larger than that obtained by a conventional TMD without any external energy adoption.

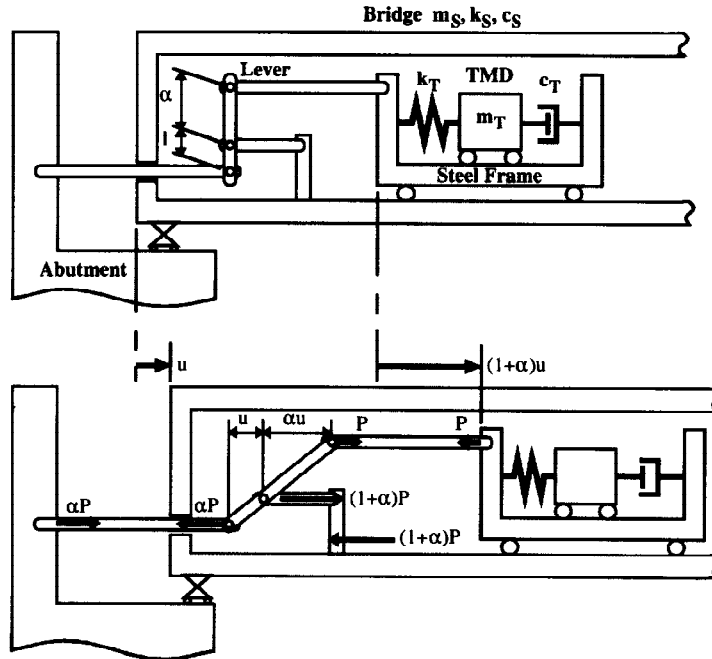


Fig. 1. A schematic diagram to represent a function of the Powerful TMD.

## FORMULATION OF EQUATION OF MOTION

The dynamic model for the Powerful TMD is illustrated in Fig.2. In this model, the displacement of the steel frame,  $u_{T'}$ , is constrained by that of a bridge structure,  $u_S$ , with a relation;  $u_{T'} = (1+\alpha)u_S$ . Using this relation, the formulation of equation of motion for the Powerful TMD can be given as follows (Kaneko et al., 1994):

$$\begin{bmatrix} m_S + (1+\alpha)^2 m_{T'} & 0 \\ 0 & m_T \end{bmatrix} \begin{Bmatrix} \ddot{u}_S \\ \ddot{u}_T \end{Bmatrix} + \begin{bmatrix} c_S + \alpha^2 c_{T'} + (1+\alpha)^2 c_T & -(1+\alpha)c_T \\ -(1+\alpha)c_T & c_T \end{bmatrix} \begin{Bmatrix} \dot{u}_S \\ \dot{u}_T \end{Bmatrix} + \begin{bmatrix} k_S + \alpha^2 c_{T'} + (1+\alpha)^2 k_T & -(1+\alpha)k_T \\ -(1+\alpha)k_T & k_T \end{bmatrix} \begin{Bmatrix} u_S \\ u_T \end{Bmatrix} = - \begin{bmatrix} m_S + (1+\alpha)m_{T'} & 0 \\ 0 & m_T \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \end{Bmatrix} \ddot{u}_G \quad (1)$$

where subscripts S, T, T' and G denote bridge structure, TMD, steel frame and ground, respectively, and  $u$ ,  $m$ ,  $c$  and  $k$  denote relative displacement, mass, damping and stiffness, respectively.

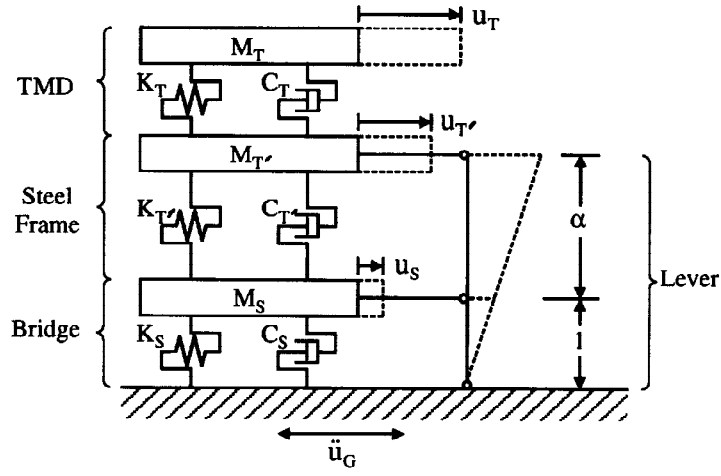


Fig. 2. A schematic diagram to represent a dynamic model for the Powerful TMD.

Having been discussed in the paper (Kaneko et al., 1994), the natural frequency of a spring-mass system consisting of the steel frame, at which the Powerful TMD acts effectively as a tuned mass damper, is limited in a range at most 2 or 3 times of that of a bridge structure. Therefore, the effect of the mass, stiffness and damping on the dynamic behavior of the whole system are considered to be negligible. Then, the equation of motion for the Powerful TMD can be simplified as Eq.(2).

$$\begin{aligned}
 \begin{bmatrix} m_S & 0 \\ 0 & m_T \end{bmatrix} \begin{Bmatrix} \ddot{u}_S \\ \ddot{u}_T \end{Bmatrix} + \begin{bmatrix} c_S + (1+\alpha)^2 c_T & -(1+\alpha)c_T \\ -(1+\alpha)c_T & c_T \end{bmatrix} \begin{Bmatrix} \dot{u}_S \\ \dot{u}_T \end{Bmatrix} + \begin{bmatrix} k_S + (1+\alpha)^2 k_T & -(1+\alpha)k_T \\ -(1+\alpha)k_T & k_T \end{bmatrix} \begin{Bmatrix} u_S \\ u_T \end{Bmatrix} \\
 = - \begin{bmatrix} m_S & 0 \\ 0 & m_T \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \end{Bmatrix} \ddot{u}_G
 \end{aligned} \quad (2)$$

### OPTIMUM TUNING OF FREQUENCY AND DAMPING

The method for optimum tuning for the Powerful TMD for harmonic excitations can be derived according to the method by Den Hartog (Den Hartog, 1962). The method of optimum tuning for Eq.(1) was so complicated that it was not adequate for design purposes. The optimum frequency,  $v_{opt}$ , and damping,  $h_{opt}$ , derived for Eq.(1), however, can be written in simple forms as shown in Eqs.(3) and (4), respectively.

$$v_{opt} = \frac{1}{1+(1+\alpha)^2 \mu} \quad (3)$$

$$h_{opt} = \sqrt{\frac{3(1+\alpha)^2 \mu}{8\{1+(1+\alpha)^2 \mu\}}} \quad (4)$$

Both of them are determined only by mass ratio,  $\mu$ , which is the ratio of mass of a TMD to that of a bridge structure, and arm ratio,  $\alpha$ . When  $\alpha$  equals to 0, these formulae are identical with those for a conventional TMD.

Fig.3 compares the optimum condition for the Powerful TMD derived from Eqs.(3) and (4) with that given from the previous method, when the frequency ratio of the steel frame to a bridge structure,  $v'$ , equals to 2. Within the range of mass ratios and arm ratios shown in the figure, they can be regarded as identical. Then, Eqs.(3) and (4) can be applicable for engineering purposes. This result denotes, in other words, that the spring connected between the frame and a bridge deck can be omitted.

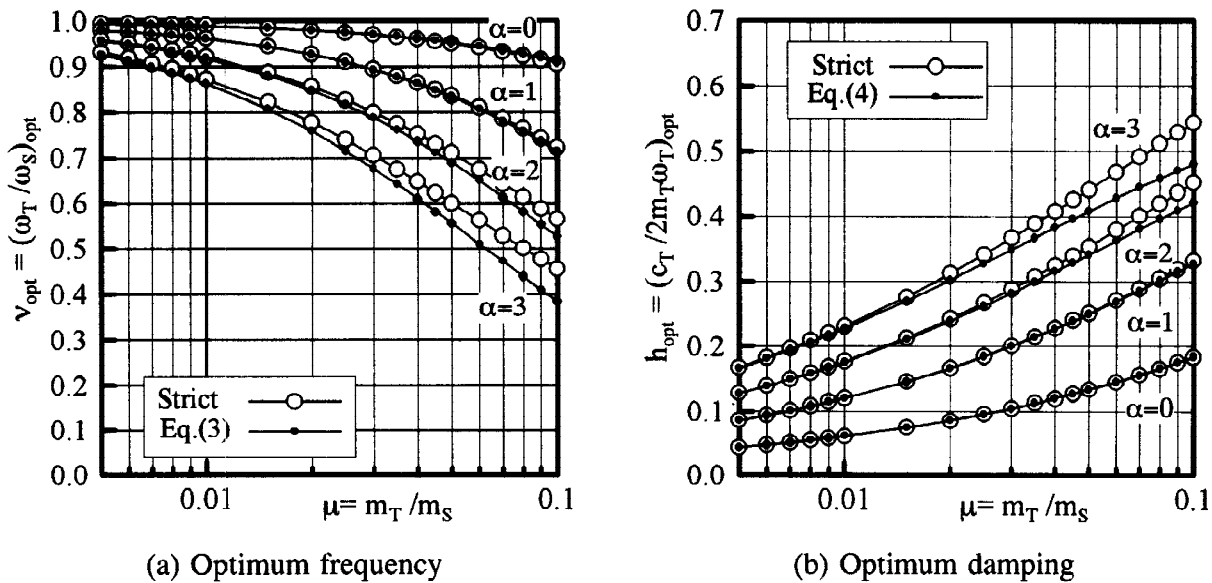


Fig. 3. Comparison of the optimum condition for the Powerful TMD derived from Eqs.(3) and (4) with that given from the previous method (Kaneko et al., 1994), when the frequency ratio of the steel frame to the bridge structure,  $v'$ , equal to 2, and the mass ratio of the steel frame to the structure,  $\mu'$ , is 0.1 times of that of the TMD to the structure,  $\mu$ .

## SHAKING TABLE TESTS AND NUMERICAL SIMULATIONS

### Outline and Testing Method

In order to verify the validity of formulation and the seismic control effect for the Powerful TMD, shaking table tests were carried out. Table 1 summarizes the similarity of a prototype and a shaking model of the bridge of concerns in this study. The model for shaking table tests were designed based on the similarity in Table 1 and the result of tuning. Parameters for the shaking table model are summarized in Table 2. The bridge model was consisted of a concrete slab and rubber bearings, which constitutes a SDOF system. The Powerful TMD was optimally tuned using the previous method based on a rigorous 2-DOF system for  $\mu = 0.043$  and  $\alpha = 2$ .

Four different strong motion records listed in Table 4 including the record at JMA Kobe station during the southern Hyogo earthquake of January 17, 1995, were used as the input motion of the test. Time duration of each input motion was shortened by 2/3 and the maximum acceleration was determined in consideration of the limitation of measuring instruments and capability of the model. Fig.4 illustrates the outline of the model and location of measuring instruments.

Table 1. Similarity of a prototype and a shaking model of the bridge.

	A. Prototype structure	B. Shaking model	B / A
Weight	7,500 tf	5.75 tf	1 / 1,300
Stiffness	16,000 tf/m	27.2 tf/m	1 / 600
Period	1.6 sec	0.923 sec	2 / 3
Remarks	A prestressed concrete bridge with the deck length of 250m.		

Table 2. Parameters for the shaking table model.

	Bridge model	TMD	Steel frame
Weight	$W_S = 5.75$ tf	$W_T = 0.25$ tf	$W_{T'} = 0.065$ tf
Parts	Concrete Slab	Steel Lump	Light Steel
Mass ratio	1	$\mu = 0.043$	$\mu' = 0.011$
Stiffness	$K_S = 27.2$ tf/m	$K_T = 0.611$ tf/m	$K_{T'} = 0.307$ tf/m
Parts	Lubber Bearing	Coil Spring	Coil Spring
Frequency	$f_S = 1.08$ Hz	$f_T = 0.78$ Hz	$f_{T'} = 1.08$ Hz
Frequency ratio	1	$\nu = 0.72$	$\nu' = 1.00$
Damping Ratio	$h_S = 0.03$	$h_T = 0.32$	$h_{T'} = 0.01$
Part		Oil Damper	
Arm Ratio of the Lever		$\alpha = 2$	

- |                          |                              |                                |
|--------------------------|------------------------------|--------------------------------|
| ① Concrete slab          | ⑤ Oil damper of TMD          | ⑨ Lever of Powerful TMD        |
| ② Rubber bearing         | ⑥ Lever of TMD               | ⑩ Outer support of the lever ⑨ |
| ③ Steel lump of TMD mass | ⑦ Steel frame                | ⑪ Inner support of the lever ⑨ |
| ④ Coil spring of TMD     | ⑧ Coil spring of steel frame |                                |

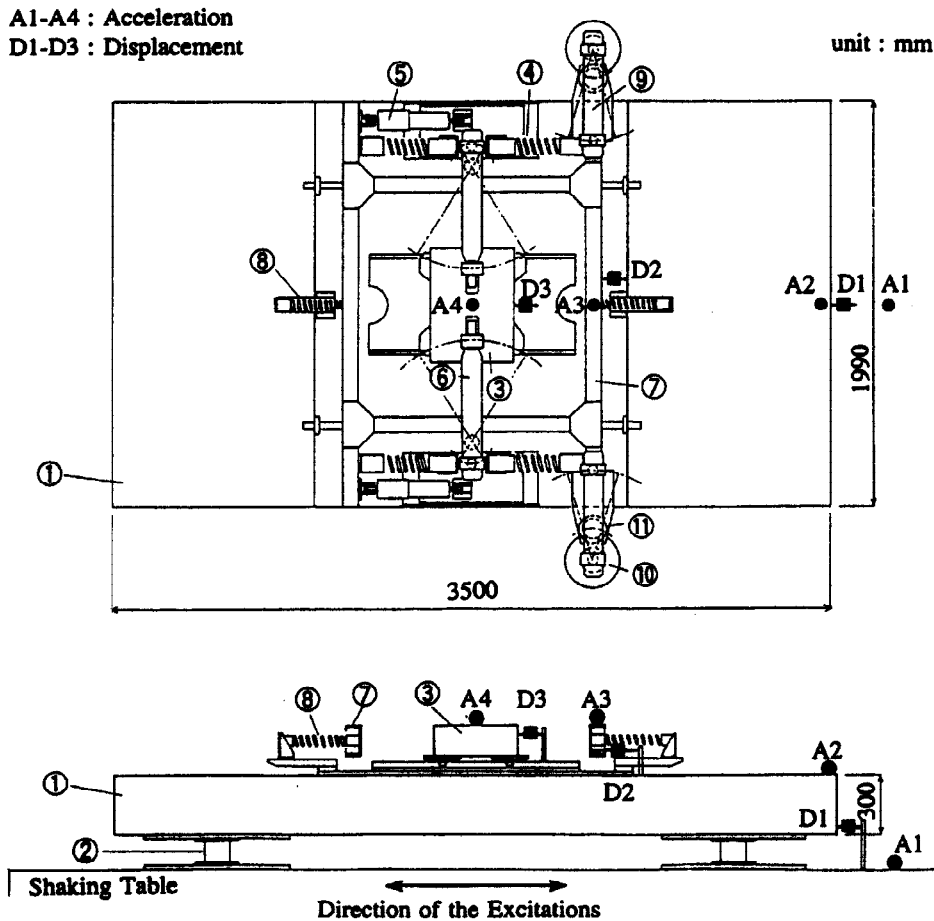


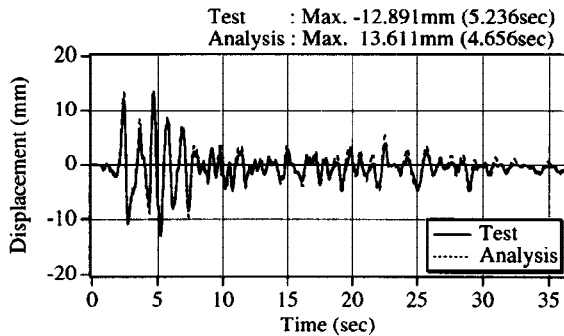
Fig. 4. Outline of the model and location of measuring instruments.

Table 3. Strong motion records used as the input motion of the test.

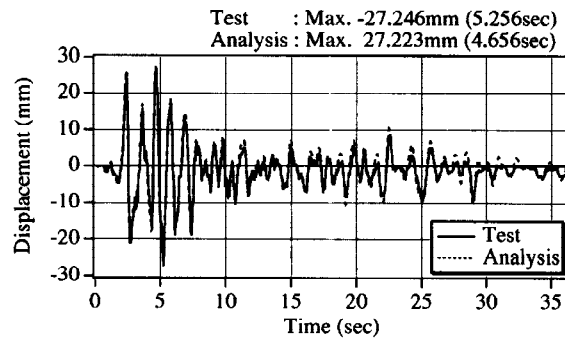
Name of strong ground motion record		Maximum acceleration	
1	Taft (EW comp.)	1952.7.21	150 gal
2	El Centro (NS comp.)	1940.5.18	150 gal
3	JMA Kobe (NS comp.)	1995.1.17	150 gal
4	Kobe University (NS comp.)	1995.1.17	50 gal

### Comparison of Test Results and Analyses

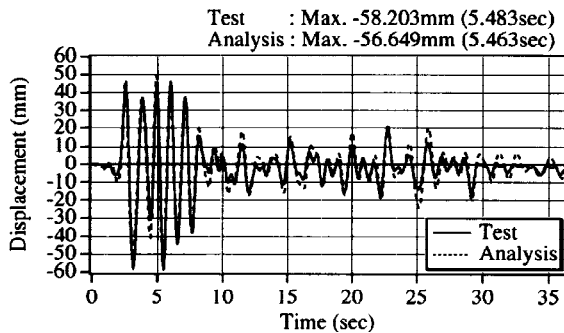
Numerical simulations for shaking table tests were also conducted solving Eq.(1) with parameters as shown in Table 2 for 4 cases in which the Powerful TMD was installed. Fig.5 compares the response time histories of relative displacement and acceleration between the shaking table test and the numerical analysis, when the Taft-EW component was applied as the input motion. As shown in the figure, the analysis successively simulates the test result, which validates a formulation of the dynamic model for the Powerful TMD. The comparison between the test result and the analysis subjected to other input motions are summarized in terms of maximum values in Table 4. The table denotes that the analysis gives the almost complete simulation of the shaking table test.



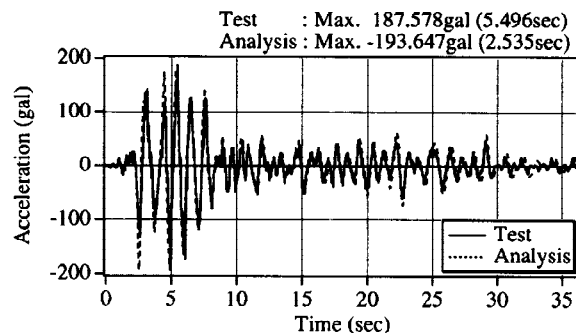
(a) Relative displacements between the bridge model and the shaking table.



(b) Relative displacements between the Steel frame and the bridge model.



(c) Relative displacements between the TMD mass and the bridge model.



(d) Accelerations at the TMD mass.

Fig. 5. Comparison of the response time histories between the shaking table test and the numerical analysis, when the Taft-EW component was applied as the input motion.

Table 4. Ratio of the maximum values of the test result to that of analysis.

		Taft	El Centro	JMA Kobe	Kobe Univ.
Acceleration	TMD mass	1.032	0.987	1.004	1.010
Displacement	Bridge model - shaking table	1.056	1.145	0.904	0.970
	Steel frame - bridge model	0.999	1.063	0.916	0.971
	TMD mass - bridge model	0.973	0.935	0.908	0.884

### Effectiveness of the Powerful TMD

Table 5 summarizes the maximum displacement of the bridge deck model among three cases which are no control, a conventional TMD installation and the Powerful TMD installation. The results of two cases of no control and the Powerful TMD installation are given by shaking table tests and that of a conventional TMD installation which was optimized using the method by Den Hartog was given by the numerical simulation. The effect of the Powerful TMD on the reduction of a bridge deck displacement is 30 through 60 % compared to the case in which no device is installed. On the contrary, a conventional TMD gives only

Table 5. Comparison of the maximum displacements of the bridge deck model, which were normalized by that of the case of no control, between the cases of the Powerful TMD installation and the conventional TMD installation.

	Taft	El Centro	JMA Kobe	Kobe Univ.
Powerful TMD	0.498	0.691	0.366	0.463
Conventional TMD	0.876	1.052	0.668	0.699

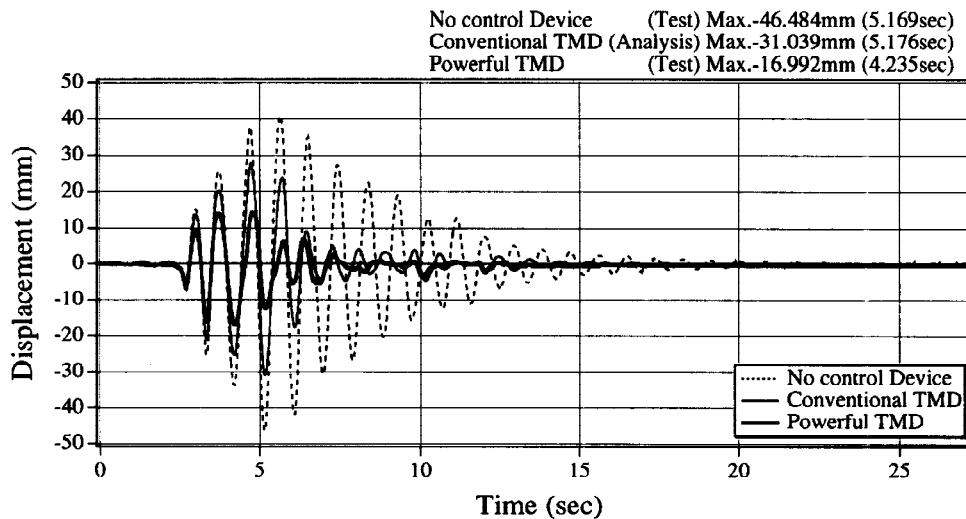


Fig. 6. Comparison for time histories of displacement of the bridge deck model among three cases which are no control, the conventional TMD installation and the Powerful TMD installation in case of the excitation using the JMA Kobe input motion.

0 through 30 % reduction in terms of the bridge displacement. Therefore, much higher seismic control can be attained by the proposed device than by a conventional device. Fig.6 illustrates an example of comparison for time histories of relative displacement between the shaking table and the bridge deck model among the three cases in case of the excitation using the JMA Kobe input motion. Although the peak of displacement response was given at 5 second in case of a conventional TMD, It was given one cycle earlier at 4 second in case of the Powerful TMD. Therefore, the proposed device acts effectively in the early stage of the earthquake excitation.

## CONCLUSIONS

Conclusions derived in this paper are summarized in the followings:

- 1) Simplification of the dynamic model for the bridge on which the Powerful TMD is installed was made for design purposes. Then, the optimum tuning for the model was proposed. The validity of the simplification and the tuning technique was verified by the comparison with those for rigorous model previously presented by the authors.
- 2) The formulation for the dynamic model for the bridge on which the Powerful TMD is installed was verified by the comparison of the response time histories between the shaking table tests and numerical simulations.
- 3) Shaking table tests revealed that the seismic response of bridges can be reduced up to almost a half by the application of the Powerful TMD, in a case the mass ratio is 0.043 and the arm ratio is 2.

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