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SEISMIC RESPONSES OF AN ISOLATED BRIDGE MODEL BY ROLLING-TYPE BEARINGS

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ABSTRACT

This paper investigates the seismic behavior of a 1/7.5 scaled bridge model isolated by rolling-type bearings. An approximate expression is derived for predicting the peak acceleration response of a structure isolated by the rolling-type bearing. Shaking table tests of the scaled bridge model have been conducted to verify the effectiveness of the rolling-type bearing as a seismic isolation device. Test results reveal that the seismic force transmitted by the rolling-type bearing is independent of the earthquake intensities when the sloped rolling mechanism is completely triggered. Comparison between the measured and estimated peak acceleration responses is made to evaluate the appropriation of the derived formula on predicting the maximum structural acceleration. Also, As an appropriate option, viscous dampers may be incorporated to reduce the larger displacement and eliminate unfavorable oscillation of the bridge deck, which may occur under strong earthquakes.

Keywords: seismic isolation, rolling-type bearing, shaking table test

INTRODUCTION

Seismic isolation technology has been applied to protect civil structures from earthquake damage for over two decades. Observed performance of several practical cases has revealed the success of seismic isolation technique in reducing earthquake-induced forces (Celebi 1994, Clark *et al*. 1997, Stewart *et al*. 1999, Nagarajaiah *et al*. 2000, Lee *et al*. 2000). On the growing need of practical application, many seismic isolation devices have been developed (Naeim and Kelly 1999, Kunde and Jangid 2003). Among those well-developed seismic isolation devices, the lead-rubber bearing (LRB), high-damping rubber bearing (HRB), and friction pendulum system (FPS) are three major devices for practical use.

The original idea of the friction-type isolation bearings was based on its constant horizontal force transmitted by a flat sliding surface under a fixed normal reaction. Later, it was realized that undesired permanent displacement might occur if no re-centering force was provided. FPS is one of the sliding-friction systems that have re-centering ability. However, the curved sliding surface may result in increased horizontal force with larger displacement. One effective approach to further reduce the transmitted forces by the FPS is to adopt a rolling mechanism instead of sliding, since rolling friction is less than sliding friction. An isolation method using free rolling rods under the basement of structures was proposed (Lin and Hone 1993). Shaking table tests were conducted to investigate a one-story frame isolated by the free rolling-rod system (Lin *et al*. 1995). Also, dynamic behavior of a rolling-ball bearing named Ball-N-Cone isolation system, which consists a steel ball sandwiched

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between two conical steel load plates, has been studied and tested (Kasalanati *et al*. 1997, CERF 1998, Hanai 2004). Effectiveness of the rolling mechanism in reducing the response of the superstructure was demonstrated. In fact, rolling bearing systems have been applied to practical shaking isolation of equipments (ISO-Base, CRS).

A sloped rolling-type bearing (RTB), which utilizes the concept of a steel cylinder rolling on a V-shape surface, has been proposed (Lee *et al*. 2003, Lee and Liang 2003, Wu *et al*. 2004). In this paper, shaking table tests have been conducted to investigate the seismic behavior of a 1/7.5 scaled bridge model isolated by the RTBs. The scaled bridge model is designed to simulate one vibration unit of a multi-span, simply-supported highway bridge. The RTB is composed of a steel cylinder (roller) and two V-shape steel plates. An approximate expression is derived first to estimate the maximum acceleration of structures isolated by the RTBs. Dynamic characteristics and seismic behavior of the RTB are evaluated from the test results. Comparison between the measured and estimated peak acceleration responses is made to assess the accuracy of the derived formula. Furthermore, viscous dampers (VDs) are combined with the RTBs to reduce the bearing displacement under earthquake excitations. Seismic responses of the bridge model isolated by the RTBs with and without VDs are compared to examine the effect of the added VDs.

FUNDAMENTALS OF SLOPED ROLLING-TYPE BEARINGS

The basic RTB unit consists of three components: an upper plate, a solid roller, and a lower plate, as shown in Fig. 1. Both the upper and lower plates are made with one flat surface and one concave surface with constant slopes. The roller is sandwiched between the concave sides of those two plates. The flat surfaces of both upper and lower plates are respectively connected to the superstructure and substructure. Thus, a sloped rolling mechanism of the bearing is expected when the isolated structure is subjected to horizontal excitations. Constant horizontal force is transmitted through the RTB when the sloped rolling mechanism is triggered. Since rolling resistance is always less than sliding resistance on the same surface, the horizontal force transmitted by the RTB shall be less than that by a sliding bearing. Also, restoring force of the RTB may be provided by the parallel component of the gravity load on the roller to the sloped surface.

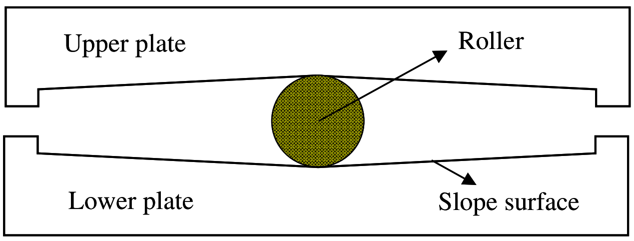


Figure 1 A schematic of the RTB unit.

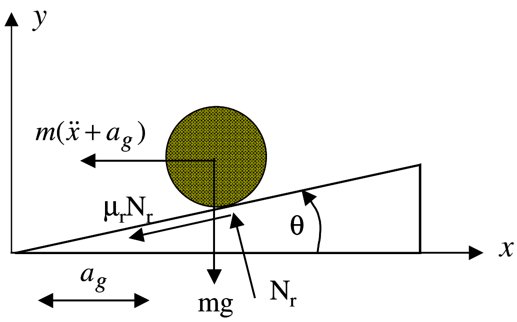


Figure 2 External forces acting on the roller.

Rigorous derivations for the real dynamic behavior of a structure isolated by rolling bearings are involved with the translation and rotation of the rollers (Lin and Horn 1993, Wang 2005). In fact, the mass of a bearing may be negligible as compared with its supported superstructure. Also, the transmitted force by the bearing is usually the major concern for a seismically isolated structure. Hence, an approximate formula is proposed here for estimating the dynamic response of the structure isolated by the RTB.

Considering a structure supported by the RTB with a sloping angle θ under horizontal excitations, external forces acting on the roller structure are the gravity load from the superstructure mg, inertia force $m(\ddot{x}+a_g)$, normal reaction from the sloped surface N_r, and the rolling friction force $\mu_r N_r$, as shown in Fig. 2. g is the gravitational constant. μ_r is defined as the ratio of the coefficient of rolling resistance to the radius of the roller (Shame 1996). Under the action of those external forces, at an instantaneous moment, the equation of sloped rolling motion of the roller structure in the horizontal direction may be described by

$$m(\ddot{x}+a_g)+N_r \sin\theta\cdot \text{sgn}(x)+\mu_r N_r \cos\theta\cdot \text{sgn}(\dot{x})=0 \tag{1}$$

where "sgn" denotes the sign of the parameter in parentheses. m is the mass of the superstructure. $\ddot{x}$ and a_g represent the relative horizontal acceleration of the roller to the lower plate and the horizontal ground acceleration, respectively. The reaction force on the sloped surface may be expressed as

$$N_r = m(g - a_g \tan\theta \cdot \text{sgn}(x))\cos\theta \tag{2}$$

Now, considering the situation of $\dot{x} \geq 0$ and $x \geq 0$, the peak acceleration response of the roller structure, $S_{a,RTB}$, may be estimated by

$$S_{a,RTB} = \cos^2\theta(\mu_r + \tan\theta)(g + a_{g,\max} \tan\theta) \tag{3}$$

It is observed from Eq.3 that the peak acceleration transmitted through the RTB is dependent on the peak ground acceleration (PGA) $a_{g,\max}$, sloping angle θ and μ_r. Also, it is recognized that for any friction-type bearing, there is a minimum horizontal force for triggering the frictional motion of the bearing. This holds for the RTB. It may be derived from the equation of motion that the requirement for triggering the rolling motion of the RTB is expressed as

$$|a_g| \geq \frac{g\cos^2\theta(\mu_r + \tan\theta)}{1-\sin\theta\cos\theta(\mu_r + \tan\theta)} \tag{4}$$

Figs.3(a) and 3(b) show the normalized acceleration responses of the roller structure ($S_{a,RTB}$/g) with respect to the PGA values for different sloping angles and μ_r ratios. It is seen that the effects of $a_{g,\max}$ and μ_r on the structural acceleration response are minor for practical application (Wu *et al.* 2004). The sloping angle is the governing parameter for the peak acceleration transmitted by the RTB.

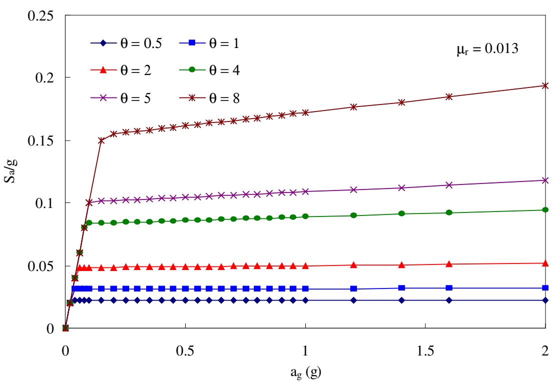


Figure 3(a) Effect of θ on $S_{a,RTB}$

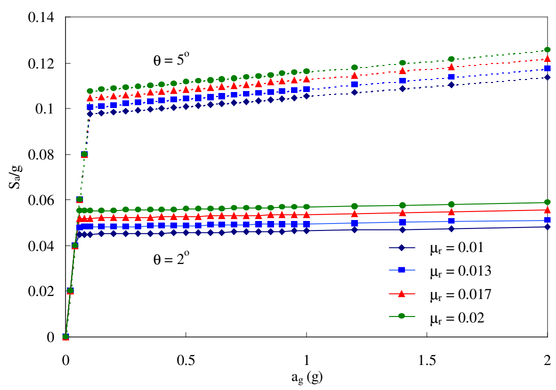


Figure 3(b) Effect of μ_r on $S_{a,RTB}$

BRIDGE MODEL

A single-span, simply supported bridge model, which consisted of a concrete π-girder and two piers, was constructed for the shaking table test, as shown in Fig. 4. The longitudinal direction of the bridge model was isolated by the RTBs installed on the top of each pier. The bridge model was scaled down from one vibration unit of a standard, simply supported PCI bridge adopted by the Directorate General of Highways in Taiwan. The span length and deck width of the prototype bridge were equal to 30m and 9m, respectively. Each bridge pier was composed of a tapered circular column with linearly varying diameters from 2.0m to 2.4m and an 8.5m cap beam in length. Cross section of the cap beam had a width of 2m and linearly varying depths from 1m to 1.8 m. Total pier height was equal to 11m, including the cap beam.

Figure 4 The 1/7.5 isolated bridge model

Considering the shaking table capacity, a scaling factor of 1/7.5 was determined for the bridge model. Center-to-center span length of the bridge model was changed from the theoretically scaled length of 400cm to 450cm for matching the locations of the anchorage holes on the shaking table. Since the bridge deck was expected to exhibit rigid-body motion under horizontal excitations, the mass similarity was the major concern for the deck model. Plan dimensions of the deck model were determined to be 500cm in length and 175cm in width. Lead blocks were placed on the π-girder to result in a total weight of 193.3 kN for the deck model.

On the other hand, the stiffness similarity was the design criteria for the pier models. To preclude stiffness degradation due to possible concrete cracks, concrete-filled steel columns were used in the pier models. Based on a scaled equivalent transformed section, thickness and exterior diameter of the steel pipe were determined to be 8mm and 21.6cm, respectively. Also, the concrete cap beams were jacketed with steel plates to prevent from cracks. Design compressive strength of the concrete used in the bridge model was 20600kN/m^2 (3000psi). Each pier model consisted of a 1.23m concrete-filled steel column in height and a 1.13m cap beam in length. Correspondences between the prototype and model are shown in Table 1.

Table 1 Correspondences between the prototype and the bridge model

Items	Prototype	Bridge model
Span length (m)	30	4.5 (4)*
Deck width (m)	9	1.75
Pier height (m)	11	1.47
Cap beam (L×W×D m^3)	8.5×2×1~1.8	1.13×0.27×0.13~0.24
Deck weight (kN)	10124	193.3 (213.1)
Pier weight (kN)	2845	17.5 (50.6)

*: Theoretical values from the similarity rule are shown in the parentheses.

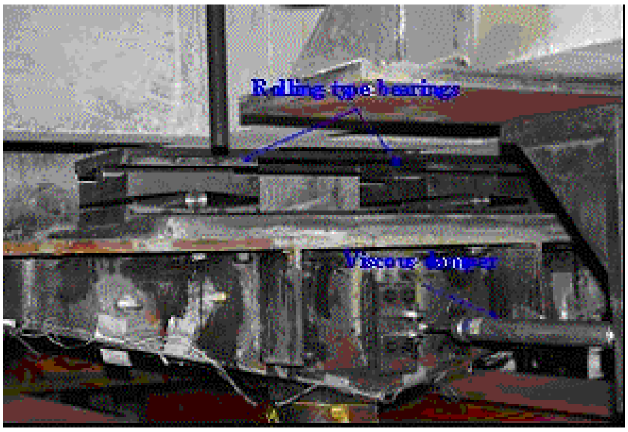


Figure 5 The RTB assembly and viscous damper

TEST SETUP AND TEST PROGRAMS

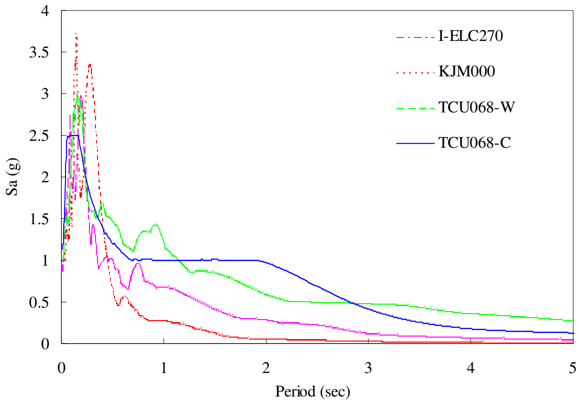
Two series of shaking table tests were carried out in this study. Test Series I was aimed at evaluating the dynamic characteristics and seismic responses of the bridge model isolated by the RTBs. Based on preliminary analytical studies, a sloping angle of 5°and a roller of 38mm in diameter were determined for the RTBs. Instead of an angled transition, an arced surface between two slopes of each plate was made for the RTBs to diminish the impact of the roller during cyclic motions. Plan dimensions of the upper and lower plates for each RTB were 27cm×27cm. The transmitted peak acceleration by the RTB was estimated to be 0.1g. Two RTBs were assembled into one isolation unit and located at the top of each pier, as shown in Fig. 5. Two-layer RTBs are shown in the figure. The lower layer was installed for future transverse isolation studies.

In Test Series II, linearly viscous dampers (VDs) were adopted to increase the damping ratio of the isolated bridge model for suppressing the bearing displacement, which might be excessive under strong earthquakes. The VD had a maximum damper force of 9.0 kN in capacity and was connected from the cap beam to the bottom of the π-girder, as shown in Fig. 5. Component test results revealed that the viscous damping coefficient of the VD was approximate to 11.8 kN-s/m. In addition, test setup and instrumentation were designed to measure the acceleration and displacement responses of the deck and cap beams. Also, strain gauges were arranged and distributed on the jacket plates of the two piers to monitor the stress conditions for ensuring their elastic responses. Sampling rate was 200 Hz for all signal processes in both test series.

White noise excitations with a bandwidth of 20 Hz and different intensities were used to investigate the dynamic characteristics of the isolated bridge model in each series. Furthermore, three earthquake records and one code-compatible acceleration history (TCU068-C) were used to evaluate the seismic behavior of the isolated bridge model. Fig. 6 shows the 5% normalized response spectra of those scaled input accelerations. For each seismic input, the PGA value was increased gradually and reached its maximum based on the estimated proportional limit of the piers or tolerable deformation of the RTBs. Table 2 shows the specified PGA values of each input acceleration and the white noise excitations.

Table 2 Specified PGA values of input excitations

Input excitations	Specified PGA (g)	
	RTB (Series I)	RTB + VD (Series II)
White noise	0.05 ~ 0.2 @ 0.05	0.1 ~ 0.25 @0.05
El Centro/I-ELC270 (Imperial Valley 1940/05/19)	0.05 ~ 0.3 @ 0.05	0.05 ~ 0.5 @ 0.05
KJMA/KJM000 (Kobe 1995/01/16)	0.05 ~ 0.6 @ 0.05	0.05 ~ 0.6 @ 0.05
Chichi/TCU068-W (Chi-chi, Taiwan 1999/09/21)	0.05 ~ 0.2 @ 0.05	0.05 ~ 0.25 @ 0.05
Chichi/TCU068-C (Chi-chi, Taiwan 1999/09/21)	0.05 ~ 0.2 @ 0.05	0.05 ~ 0.25 @ 0.05



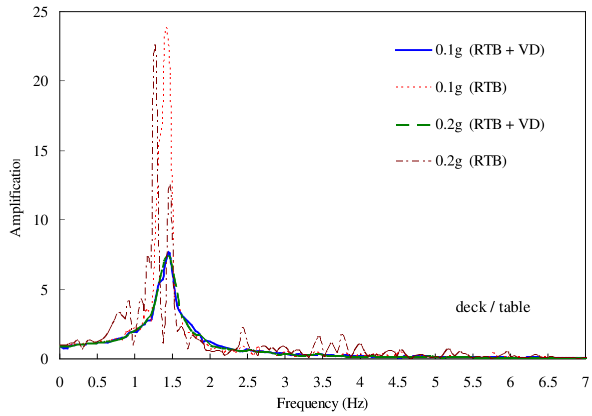


Figure 6 Normalized input response spectra.

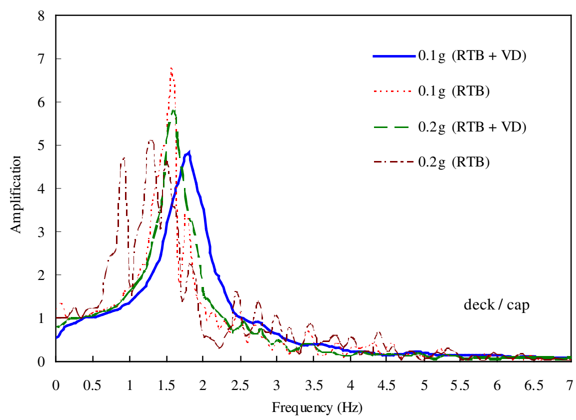
Figure 7 The deck-to-table transfer functions.

TEST RESULTS

Dynamic Characteristics

As nonlinear behavior of the RTB is revealed by its equation of motion (Eq.1), it is expected that the dynamic characteristics of the isolated bridge model may be dependent on the intensity of the ground motion. Fig. 7 shows the deck-to-table transfer functions of the isolated bridge model with or without VDs. It is seen that the dominant frequency of the model without VDs changes as the PGA increased from 0.1g to 0.2g. Nevertheless, the nonlinear characteristics are suppressed by the added VDs. This is due to the fact that the displacement of the RTBs with VDs is significantly reduced. Under the 0.1g excitation, the dominant vibration frequencies of the bridge model with and without VDs are 1.465 Hz and 1.416 Hz, respectively. Also, if estimated by the half-band power method, the corresponding equivalent damping ratios are approximated to 8.0% and 4.0%.

Furthermore, dynamic characteristics of the RTBs are investigated from the deck-to-cap beam transfer functions, as shown in Fig. 8. Different from the deck-to-table transfer functions, the nonlinear behavior of the RTBs without VDs is significant even under the 0.1g excitation. The extent of nonlinearity is mitigated as the VDs are added to the RTBs.



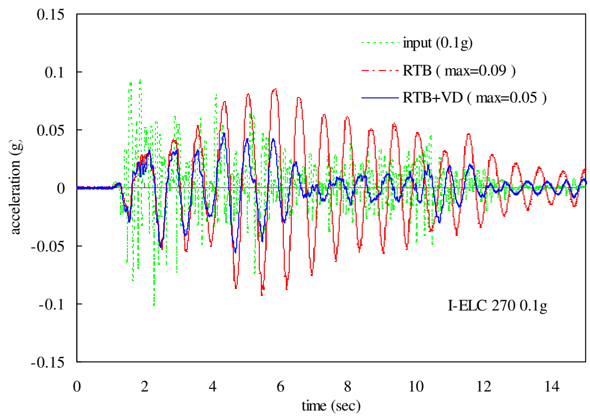


Figure 8 The deck-to-cap beam transfer function.

Figure 9(a) Acceleration time histories.

Earthquake Responses

The longitudinal deck acceleration ($\ddot{x}_a$) and bearing displacement (Δ_b) histories of the isolated bridge model with or without VDs under the scaled 0.1g and 0.3g I-ELC270 records are shown in Figs. 9(a), 9(b), 10(a), and 10(b). Quite different acceleration histories are observed for the bridge model without VDs. Wave form of the deck acceleration has been changed from smooth sinusoidal shapes to a series of rectangular pulses as the PGA increases from 0.1g to 0.3g. Under the 0.3g ground motion, the peak acceleration is less than or approximated to 0.1g, which is the estimated maximum acceleration transmitted by the RTBs. The deck displacement is significantly reduced by the added VDs even though the acceleration responses may be slightly increased. Fig. 11 shows the force-displacement loops of the RTBs with or without VDs. Similar to the test results by Kasalanati *et al*. (1997), the arced transition area results in initial elastic stiffness of the bearing. A constant force is reached as the cylinder rolls on to the sloped surfaces. Also, it is observed that the equivalent damping ratio of the RTBs is so small that damper devices may be necessary to reduce their excessive displacement response.

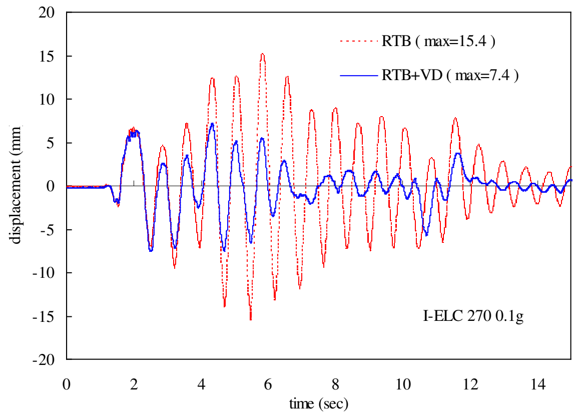


Figure 9(b) Displacement time histories.

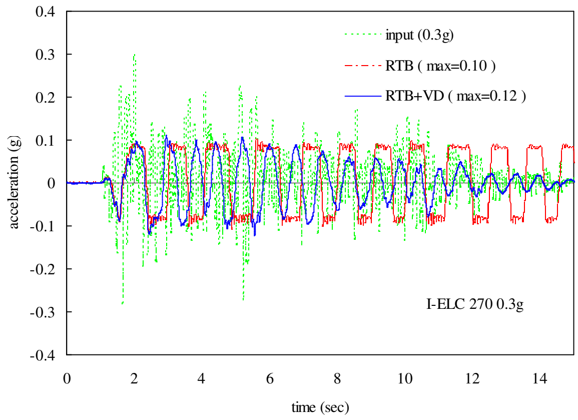


Figure 10(a) Acceleration time histories.

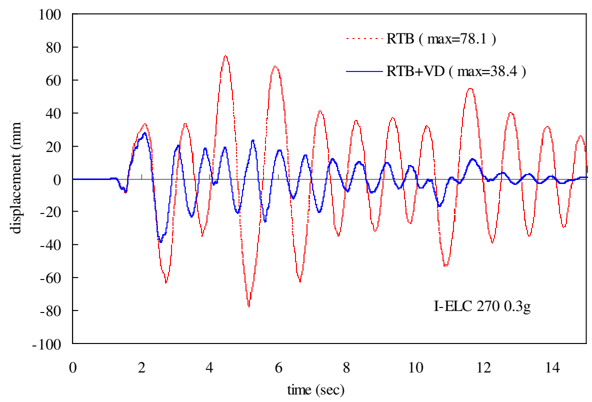


Figure 10(b) Displacement time histories.

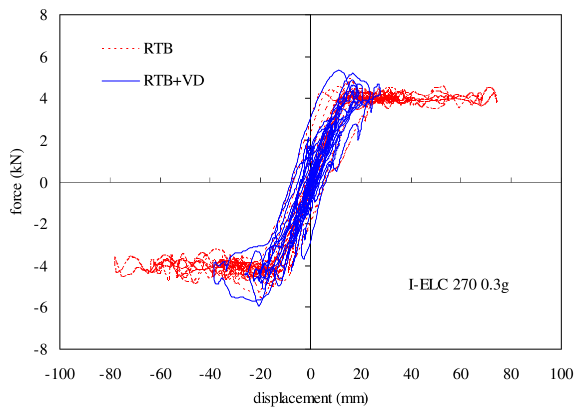


Figure 11 Force-displacement loops.

Figs. 12(a) and 12(b) show the deck acceleration and bearing displacement histories under the scaled 0.2g TCU068-C excitation. It is seen that the RTBs without VDs presents considerable oscillation even if the excitation becomes negligible. This illustrates that the deck acceleration of the isolated bridge model without VDs may decay quite slowly under pulse-type seismic excitations. Evidently, it will be an important issue to mitigate such unfavorable oscillation for the application of the RTBs. Adding dampers may be a good option for eliminating the undesired oscillation.

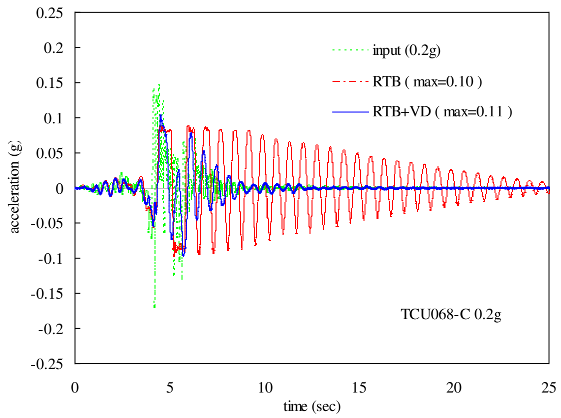


Figure 12(a) Acceleration time histories.

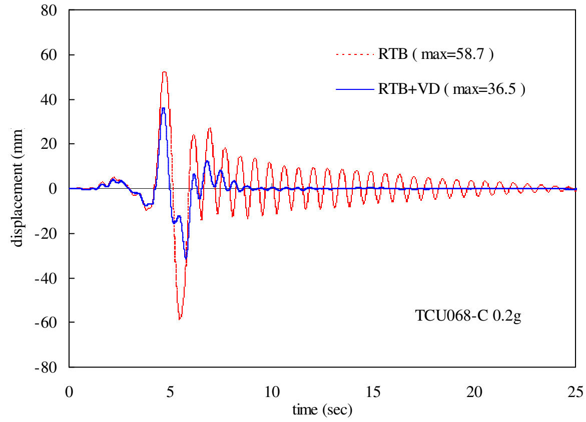


Figure 12(b) Displacement time histories.

The transmitted acceleration by the RTBs is one major concern of the seismic responses of the isolated bridge. Figs. 13 and 14 respectively present the measured peak deck acceleration and bearing displacement with respect to the measured PGA for the isolated bridge model without VDs. It is seen that, at the beginning, the peak acceleration response increases gradually with the PGA and then it attains the estimated maximum transmitted acceleration 0.1g. This reveals that there is a lower-bound PGA for triggering the sloped rolling mechanism. The PGA values of those input earthquakes for triggering the sloped rolling mechanism are around 0.13g except for the KJM000 record, which has a higher value of 0.2g. This happening may be attributed to the fact that the bearing displacement under the KJM000 excitations is less than that under others. In Fig. 14, it is seen that, with the exception of the KJM000 record, all other seismic inputs have similar displacement variations with the PGA. This indicates that different seismic inputs may lead to different relationships between the peak bearing displacement and the PGA. Also, it is found that those displacement responses corresponding to the trigger PGA values are equal to a same value of 20mm approximately. This value is very close to the radius of the roller. It appears that the sloped rolling mechanism is completely triggered only when the bearing displacement is larger than the radius of the roller. Below the trigger PGA values, the deck acceleration and bearing displacement exhibit a nearly linear variation with the PGA. Hence, earthquake-dependent acceleration responses may occur under small excitations.

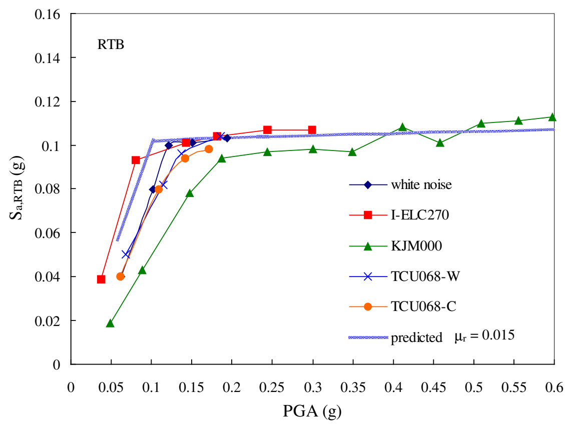


Figure 13 Peak deck acceleration (RTB).

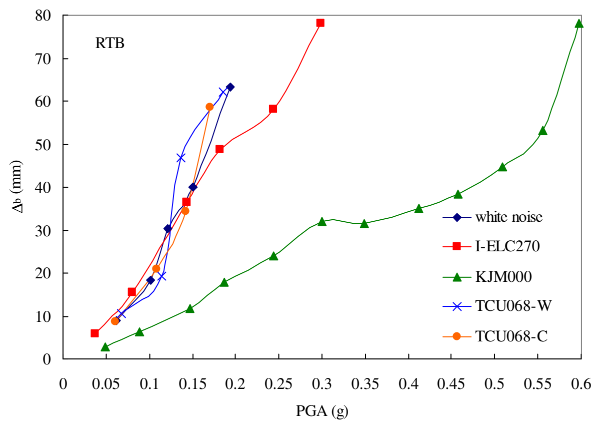
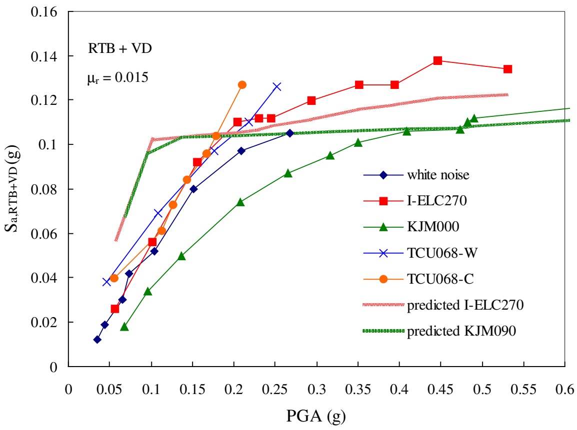


Figure 14 Peak bearing displacement (RTB).

In Test Series II, the deck acceleration responses are different from that in Test Series I, as seen in Fig. 15. Instead of an apparent asymptotical limit, the acceleration responses increase steadily with the PGA at a decreasing rate. Moreover, the bearing displacement in all cases has been significantly reduced as shown in Fig. 16. As compared to the results of Test Series I, adding VDs will lead to lesser bearing displacement responses without altering the trend of variation with the PGA, although more scattered responses may be induced by different earthquakes. If the radius of the roller is used as a displacement index for triggering the sloped rolling mechanism of the isolated bridge with VDs, then the corresponding PGA values are around 0.20g, 0.48g, 0.13g, and 0.14g, respectively for the I-ELC270, KJM000, TCU068-W, and TCU068-C earthquake records. These trigger PGA values are in general larger than those of the isolated bridge without VDs. Also, the VDs help to reduce the acceleration response of the isolated bridge as the PGA is less than the trigger value, although its peak acceleration may be slightly increased under intensive excitations.



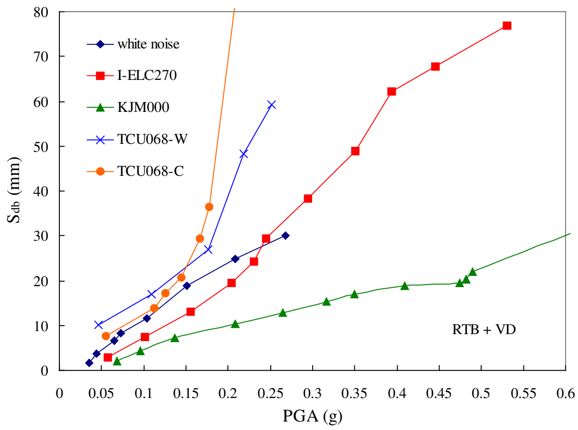


Figure 15 Peak bearing displacement (RTB+VD). Figure 16 Peak deck acceleration (RTB+VD).

ANALYTICAL PREDICTION

Estimation of the peak acceleration or the maximum transmitted force is an important issue for the application of the RTB. As derived earlier in this paper, Eq.3 may be used to estimate the peak acceleration of the superstructure supported by the RTBs. Usually, the sloping angle is so small such that Eq.3 may be simplified as

$$S_{a,RTB} = (\mu_r + \theta)(g + a_{g,\max}\theta) \tag{5}$$

The value of μ_r is determined from the coefficient of rolling resistance (δ) and the roller radius (r). According to Shames (1996), the value of δ may vary from 0.007in to 0.015in for a steel interface. With r equal to 0.748in (19mm), μ_r is between 0.00936 and 0.0201 for the RTBs. Since no test has been conducted for estimating the coefficient of rolling resistance in this study, an average value of 0.015 for μ_r is used in the numerical analyses.

As shown in Fig. 13, the predicted acceleration responses by Eq.5 agree well with the measured peak values when the sloped rolling mechanism is triggered. The trigger PGA value is determined by Eq.4. The peak acceleration of the superstructure is considered to be equal to the PGA when it is less than the trigger value. However, probably due to the arced transition area, test results reveal earthquake-dependent responses when the PGA is less than the trigger value. Hence, Eq.4 may not be appropriate for every seismic input in the experiment. In Fig. 13, better estimation is obtained for the I-ELC270 earthquake record than for others.

When the RTBs are combined with the VDs, the equation of sloped rolling motion at an instantaneous moment may be written as

$$m(\ddot{x} + a_g) + C\dot{x} + N_r \sin\theta \cdot \mathrm{sgn}(x) + \mu_r N_r \cos\theta \cdot \mathrm{sgn}(\dot{x}) = 0 \tag{6}$$

where C is the damping coefficient of the VDs. Based on the equivalently linear concept, the peak acceleration response, $S_{a,RTB+VD}$, is roughly estimated by

$$S_{a,RTB+VD} = \sqrt{S_{a,RTB}^2 + (F_{VD}/m)^2} \tag{7}$$

where $F_{VD} = C\omega_{eq}S_{db}$ represents the maximum damper force. ω_{eq} is the dominant frequency in radian obtained from the transfer function of measured acceleration history. S_{db} is the maximum bearing deformation calculated from the measured displacement responses. Predicted peak acceleration responses under the I-ELC270 and KJM000 earthquake excitations are shown in Fig. 15. Obviously, the rough estimation may capture the peak acceleration response only when the sloped rolling mechanism is triggered.

CONCLUSIONS

Shaking table tests of a 1/7.5 scaled bridge model isolated by sloped rolling-type bearings with or without viscous dampers have been carried out. Nonlinearly dynamic characteristics of the roller bearings were confirmed under white noise excitations. Seismic behavior of the bridge model was investigated under four earthquake excitations scaled to various peak ground accelerations. Test results have verified that the sloped rolling-type bearing is an effective seismic isolation device. The seismic force transmitted by the bearings was independent of the earthquake intensities when the sloped rolling mechanism was completely triggered. Also, the maximum structural acceleration may be estimated by the proposed expression (Eq.5). In addition, due to the small damping ratio of the roller bearings, large displacement and undesired deck oscillation may be induced. Adding viscous dampers represents an option to reduce the bearing displacement and eliminate the unfavorable oscillation although the acceleration response may be increased slightly. Comparison between the seismic responses of the isolated bridge model with and without the viscous dampers has demonstrated that, regardless of the peak ground acceleration, the bearing displacement may be significantly reduced by the added dampers.

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