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DYNAMIC BEHAVIOR AND SEISMIC PERFORMANCE OF BASE-ISOLATED BRIDGES IN OBSERVED SEISMIC RECORDS

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SUMMARY

The purpose of this paper is to analyze the dynamic behavior of base-isolated bridges and to quantify the performance of isolation bearings in actual earthquakes using recent recordings obtained at instrumented base-isolated bridges in Japan, including the behavior of viaducts of Hanshin-Expressway Matsunohama district during the 1995 Hyogoken-Nanbu Earthquake. Three bridges with lead rubber bearing, high damping rubber bearing, and natural rubber bearings are selected for detailed study. Their seismic performance is evaluated through comparison between identified stiffness and damping values from observed records and predicted values from loading test of each device. The analysis revealed that base-isolation effect is present in all bridges, while contribution of minor friction element can significantly influence the performance.

INTRODUCTION

Application of base-isolation by lead rubber bearings and high damping rubber bearings became popular in Japan especially after 1995 Hyogoken-Nanbu earthquake to enhance seismic performance of highway system. In base-isolation design, dynamic effects, such as longer natural period and higher energy dissipation/damping are positively employed to reduce seismic inertia force. However, base-isolated bridges had not been exposed to disastrous earthquakes, and its actual dynamic behavior and performance during strong earthquakes has remained unknown. The purpose of this paper is to analyze the dynamic behavior of base-isolated bridges and to quantify the performance of isolation bearings in actual earthquakes, using recent recordings at instrumented base-isolated bridges. Table 1 shows the list of instrumented base-isolated bridges in Japan. All of them are designed before the codification of base-isolation (MENSHIN) design [Japan Road Association, 1991], and the base-isolation effect is not expected to reduce seismic force but is considered as additional contribution to seismic performance improvement. Table 2 shows the list of measured earthquakes records. It can be seen moderate earthquake ground motions are observed in several sites. Three bridges with typical features are selected for detailed investigation here. The first two examples are the behavior of Matsunohama Viaduct A and B during 1995 Hyogo-ken Nanbu (Kobe) Earthquake. The Viaduct A is supported by natural rubber bearings (NRB), while the Viaduct B is supported by lead rubber bearings (LRB). The third example is Yamaage Bridge, which is supported by high damping rubber bearings (HDR).

The method employed here to evaluate seismic performance is as follows: (1) equivalent stiffness and damping of base-isolated structure are identified in frequency domain transfer function method; (2) expected equivalent stiffness and damping are predicted independently based on loading test of base-isolation bearings conducted prior to installation, (3) the actual performance is evaluated by comparing these observed and predicted values with consideration of inherent uncertainty in measurement and prediction.

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Table 1. List of base isolated bridges with field measurement [Subcommittee on Structural Control, 1998]

	Onneto Bridge	Marukibashi Bridge	Yamaage Bridge	Miyagawa Bridge	Matsunohama Viaduct A	Matsunohama Viaduct B
Location	Hokkaido	Iwate	Tochigi	Shizuoka	Hanshin Expressway	Hanshin Expressway
Length	102.5 [m]	92.8 [m]	246.3 [m]	105.5 [m]	211.5 [m]	211.5 [m]
Span	4 x 25.5 [m]	3 x 30.5 [m]	6 x 40.8 [m]	32.8 + 39.0 + 32.85 [m]	44.5 + 2x45.0 + 44.5 [m]	46.0 + 2x60.0 + 44.5 [m]
Superstructure type	Steel girder	Prestressed concrete girder	Prestressed concrete girder	Steel girder	Steel I girder	Steel box girder
Foundation type	Pile foundation	Spread footing	Spread footing	Spread footing	Pile foundation	Pile foundation
Bearing type	LRB NRB for both ends	LRB	HDR	LRB	NRB Bearing plate bearing for both ends	LRB Pivot-roller for both ends
Direction of base-isolation	Longitudinal	Longitudinal +transverse	Longitudinal	Longitudinal	Longitudinal	Longitudinal
Measurement	Under-ground, surface, pier top, girder	Under-ground, pier top, girder	Under-ground, pier top, girder	Under-ground, pier top, girder	Underground, footing top, pier top, girder	Underground, footing top, pier top, girder

LRB: lead rubber bearing, HDR: high damping rubber bearing, NRB: natural rubber bearing. Measurement installation is only at one pier for each bridge.

Table 2. Measured earthquakes [Subcommittee on Structural Control, 1998]

	Date	PGA (longitudinal)	Magnitude	Earthquake
Onnneto Bridge	1994.10.4 1994.8.31 1994.10.9 1995.1.21	316 [cm/sec^2] 191 [cm/sec^2] 46.7[cm/sec^2] 43.7[cm/sec^2]	8.1 6.5 7.3 6.2	1994 Hokkaido Touhou Oki 1994 Hokkaido Touhou Oki: after shock 1994 Hokkaido Touhou Oki: after shock
Marukibashi Bridge	1994.12.28 1995.1.7	64.1[cm/sec^2] 48.1[cm/sec^2]	7.5 6.9	1994 Sanriku Haruka Oki
Yamaage Bridge	1996.2.17 1997.5.12 1997.2.20 1994.10.4	29.3[cm/sec^2] 12.4[cm/sec^2] 10.1[cm/sec^2] 9.08[cm/sec^2]	6.6 5.7 5.3 8.1	 1994 Hokkaido Touhou Oki
Miyagawa Bridge	1991.4.25	4.3[cm/sec^2]	4.9	
Matsunohama Viaduct A	1995.1.17 1995.1.17 1995.1.17 1995.1.17	144[cm/sec^2] 19.6[cm/sec^2] 17.7[cm/sec^2] 12.5[cm/sec^2]	7.2 - - -	1995 Hyogo-ken Nanbu (Kobe) 1995 Hyogo-ken Nanbu: after shock 1995 Hyogo-ken Nanbu: after shock 1995 Hyogo-ken Nanbu: after shock
Matsunohama Viaduct B	1995.1.17 1995.1.17 1995.1.17 1995.1.17	169 [cm/sec^2] 20.7[cm/sec^2] 20.6[cm/sec^2] 12.8[cm/sec^2]	7.2 - - -	1995 Hyogo-ken Nanbu (Kobe) 1995 Hyogo-ken Nanbu: after shock 1995 Hyogo-ken Nanbu: after shock 1995 Hyogo-ken Nanbu: after shock

METHOD OF SYSTEM IDENTIFICATION

Dynamic properties are identified through comparing the theoretical transfer functions with the measured one obtained from accelerograms. Here, linear system is assumed and equivalent linearizedd stiffness and damping are employed as the indicators of performance. The stiffness and damping are identified by minimizing the following error function:

$$E = \int_{w1}^{w2} \left| H_M - H_T \right|^2 dw \tag{1}$$

where H_M is measured, and H_T is the theoretical prediction of transfer function. The domain of integration is selected according to the frequency band of interest. To obtain theoretical transfer function, appropriate physical modeling which reflects number of measured modes is made. Since the main purpose of the investigation is the verification of bearing performance, transfer function between pier cap and girder is used for identification.

ANALYSIS OF MATSUNOHAMA VIADUCTS

One of the strongest motion records are obtained at Matsunohama Viaducts, which locate about 35 [km] east of epicenter of 1995 Hyogoken-Nanbu Earthquake. Table 2 gives the peak ground acceleration (PGA) values at free surface ground. The elevation view of the viaducts are shown in Figures 1 and 2. The instrumented pier is P-400 for Viaduct A, and P-408 for Viaduct B in the figures. The equivalent stiffness and damping of the bearings are identified from the transfer function between the pier cap and the girder during each earthquake. Because the bridges are isolated only in longitudinal direction and transverse motion is restrained, and the stiffness of the girder in longitudinal direction is much higher than that of bearings, the superstructure is modeled as a single degree of freedom lumped mass system with longitudinal motion [Chaudhary, et. al., 1999].

The identified stiffness and damping ratios are shown in Figures 3, 4, and 5 denoted by identified value. The identified results has been found to be highly reliable according to Bootstrap analysis [Abé, et.al. 1998]. Damping of Viaduct A is not studied because the NRB in this case is not designed to dissipate energy but to distribute horizontal inertia force. The experimental values correspond to the prediction based upon actual restoring force characteristics of the bearings tested prior to installation [Hiromatsu, et. al., 1994]. It can be observed that the identified values and predicted values agree well for main shock but identified values are much higher than experimental values for aftershocks. To understand this discrepancy, the friction forces introduced by the bearings at each end support of the girder, which are the bearing plate bearings for Viaduct A and pivot-rollers for Viaduct B, are also plotted as lines with arrow in the figures. Because the experimental results on similar friction bearing revealed scatter of friction coefficients depending on each testing [Shimoda, 1990], the line with arrow shows the possible deviation of stiffness/damping values introduced by friction of end bearings. It can be seen that the identified values are within this possible range of deviation. It should be noted that friction effect becomes relatively smaller for larger displacement because Coulomb friction force is constant with respect to displacement while rubber restoring force is increasing with displacement. Because of this effect, friction force contribution is marginal for main shock.

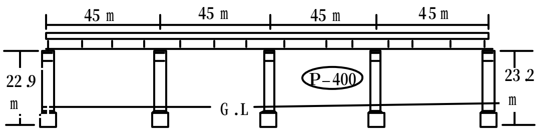

Figure 1. Matsunohama Viaduct A

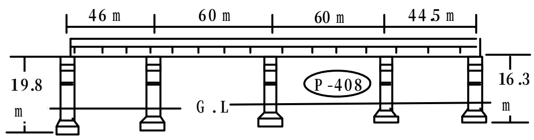

Figure 2. Matsunohama Viaduct B

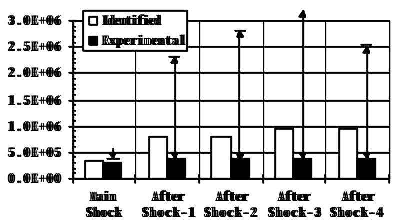

Figure 3. Equivalent stiffness of Viaduct A.

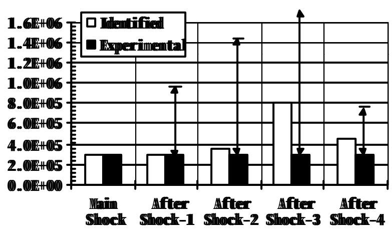

Figure 4. Equivalent stiffness of Viaduct B.

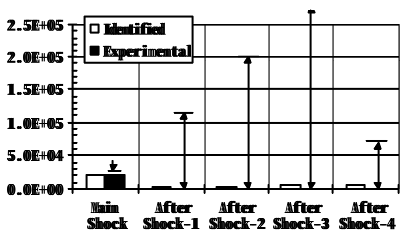

Figure 5. Equivalent damping of Viaduct B.

ANALYSIS OF YAMAAGE BRIDGE

Yamaage Bridge, which is a continuous prestressed girder bridge supported by HDR, is instrumented at P5 as shown in Figure 6. The bridge is. In longitudinal direction, the behavior of the girder is found to be close to the rigid body and identification is successfully made assuming the single degree of freedom lumped mass model as in the previous Matsunohama case. However, in the transverse direction, another distinct peak is found as shown in Figure 7. This peak is found to be the contribution of out of plane bending mode of the superstructure. Therefore, the girder is modeled as a beam with spring supports at the place of bearings, assuming that the girder stiffness and mass are those of the design values. Because damping is localized at bearing supports, the system becomes non-proportionally damped. To accurately reflect this behavior, identification is conducted using transfer function of non-proportionally damped system [Kaito, et. al., 1999]. Figure 7 shows the comparison of measured and identified transfer function between pier cap and girder. It is seen that this modeling with non-proportional damping can express the behavior of the structure with reasonable accuracy.

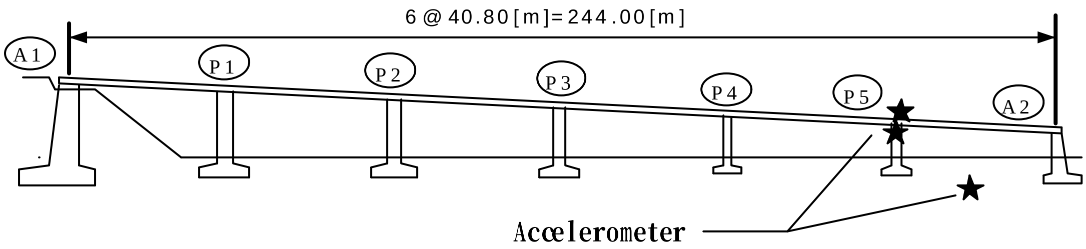

Figure 6. Yamaage Bridge

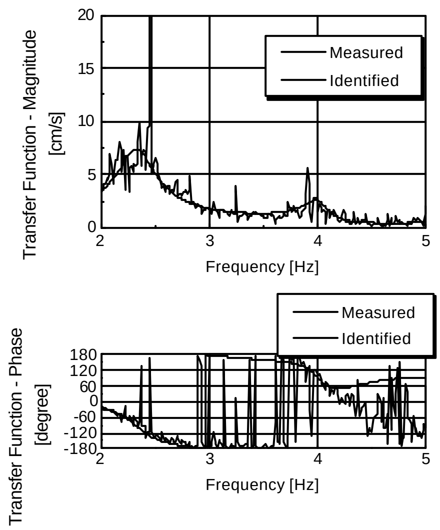

Figure 7. Transfer function in transverse direction of Yamaage Bridge.

Figure 8 and 9 show the comparison between the identified dynamic properties and prediction by test results. Typical reaction force displacement relationship of HDR is shown in Figure 10. HDR has higher stiffness at the first loading cycle, followed by stable loop from the second cycle. Therefore, to evaluate performance, the upper limit stiffness of HDR is taken from the first loop and lowest possible value is taken from the second loop. Since all the ground motions observed at the location was relatively weak, the friction effect introduced by restrainers is found to have significant contribution in this case. Nevertheless, the identified values do not contradict with the expected bearing performance.

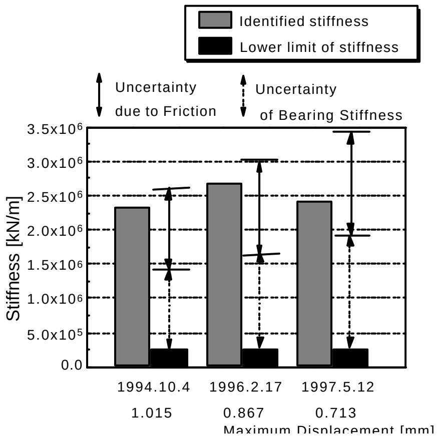

Figure 8. Equivalent stiffness

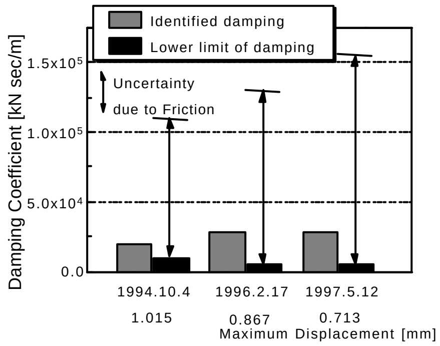

Figure 9. Equivalent damping

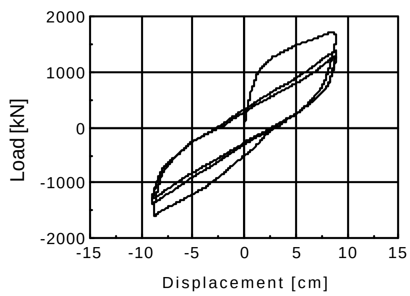

Figure 10. Reaction force-displacement relationship of HDR.

Performance of three instrumented base-isolated bridges, supported by LRB, HDR and NRB, during actual earthquake is investigated in this paper. Major conclusions are as follows:

(1) The isolated girders are found to behave rigid body in longitudinal direction and flexible beam for transverse direction, which indicates the base-isolation effect to isolate girder and pier is present in all bridges.

(2) Bearing equivalent stiffness and damping are successfully identified in frequency domain method with appropriate modeling the dynamics of girder. The performance of isolation bearings identified from actual earthquake agrees with predicted performance based on loading test conducted prior to installation within the possible range of modeling uncertainties, i.e., the effect of friction.

(3) Friction force introduced by minor structural elements can affect the dynamic behavior of superstructure and increase the modeling uncertainty considerably, which may degrade the base-isolation effects. This effect is found particularly prominent for smaller earthquakes. Although contribution of these minor details is usually neglected in current design and analysis, careful treatment is essential to achieve the design performance.

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