IABSE-JSCE Joint Conference on Advances in Bridge Engineering-II, August 8-10, 2010, Dhaka, Bangladesh. ISBN: 978-984-33-1893-0
Amin, Okui, Bhuiyan (eds.) www.iabse-bd.org

Effect of modeling approaches on seismic response prediction of base isolated highway bridges

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ABSTRACT: This study is devoted towards evaluating the effects of modeling of isolation bearings on seismic responses of a highway bridge. To this end, a nonlinear dynamic analysis of a multi-span continuous seismically isolated highway bridge is carried out with two natural and high damping rubber bearings. Three analytical models of the isolation bearings are employed in the analysis for comparison: the conventional equivalent linear and bilinear models as specified in Japan Road Association (JRA) and American Association of State Highways and Transport Officials (AASHTO) and the proposed strain-rate dependent rheology model by the authors. The proposed rheology model is capable of reproducing the nonlinear viscosity and the elasto-plastic behavior along with strain hardening of the bearings. A tri-linear hysteretic model is employed in the analysis for representing the nonlinear mechanical behavior of the bridge pier. A solution algorithm for solving the first order governing differential equation of the rheology model is developed to implement the proposed model into nonlinear time history analysis software. Two design earthquake ground motions as recommended by JRA, applied in the longitudinal direction, are used in the analysis. The dynamic responses of the isolation bearings and the rotation responses of the plastic hinge in concrete piers are compared for different modeling of isolation bearings. Finally, a comparative assessment of the bridge responses shows the sensitivity of modeling of isolation bearings in evaluating seismic responses of the bridge.

1 INTRODUCTION

Since the severe damage due to the Kobe earthquake occurred in Japan in 1995 laminated rubber bearings have been adopted widely as an isolation system of important structures for the few years. In seismically isolated systems, the superstructure is decoupled from the earthquake ground motion by introducing a flexible interface between substructure and superstructure. Thereby, the isolation system shifts the fundamental time period of the structure to a large value and/or dissipates the energy in damping, limiting the amount of force that can be transferred to the superstructure such that inter-story drift and floor accelerations are reduced drastically. The matching of fundamental frequencies of base-isolated structures and the predominant frequency contents of earthquakes is also consequently avoided, leading to a flexible structural system more suitable from earthquake resistance viewpoint. Three types of laminated rubber bearings are available for the base isolation devices: natural rubber bearings (RBs), lead rubber bearings (LRBs) and high damping rubber bearings (HDRBs).

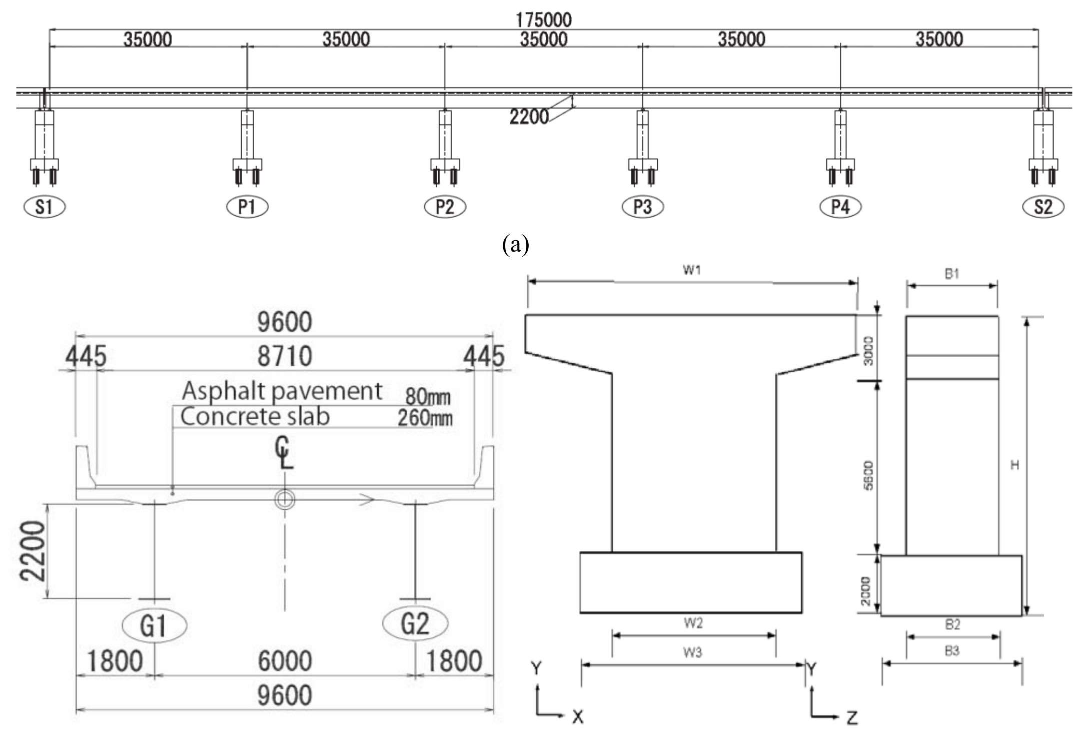
The dominating mechanical behavior of HDRBs has been recognized to be of rate-dependence as documented in several published works (i.e. Bhuiyan et al. 2009 and Hwang et al. 2002). Emphasizing the above mentioned mechanical behavior of HDRBs as evident in the experimental observations an elasto-viscoplastic rheology model has been proposed by the authors (Bhuiyan et al., 2009). This model is well capable of reproducing the rate-dependent along with strain hardening behavior of HDRBs at room temperature. On the other hand, RBs and LRBs exhibit nonlinear elasto-plastic and strain hardening behavior along with somewhat weak rate-dependent mechanical behavior (Robinson 1982 and Bhuiyan, 2009). Considering some aspect of the experimental observations of LRBs, Robinson (1982) has proposed a bilinear model for representing the hysteresis behavior of LRBs, which is conceptually the same as that recommended for isolation bearings in specification of highway bridges (JRA 2002). However, the strain hardening features of the bearings as evidently observed in the experiments (Abe et al., 2004; Bhuiyan, 2009; Kikuchi and Aiken, 1997) cannot be well reproduced by the available models. In order to improve the performance of the existing models for LRBs and RBs, a rheology has been proposed (Bhuiyan, 2009) by simplifying the earlier rheology model for HDRBs (Bhuiyan et al. 2009).

The objective of the current study is to evaluate effects of modeling of isolation bearings on seismic responses by conducting the nonlinear dynamic analysis of a multi-span continuous highway bridge. Three types of isolation bearings, i.e. RB, LRB and HDRB, are considered in this study. The isolation bearings are modeled by the equivalent linear, the bilinear models specified by JRA (2002) and the rheology model proposed by the authors (Bhuiyan et al. 2009 Bhuiyan, 2009) for comparison

2 MODELING OF BRIDGE

2.1 *Physical Model*

Figure 1 shows the details of the physical model of the bridge comprised of a five-span continuous composite deck supported by a number of isolation bearings. The superstructure consists of 260 mm continuous composite slab with 80 mm of asphalt supported on two continuous steel girders. The depth of the continuous steel girder is 2200 mm. The substructures consist of RC piers and footings supported on pile foundations. The isolation bearings are placed between the steel girders and top of the piers. As laminated rubber bearings, three types of isolation bearings are considered: high damping rubber bearings (HDRBs), lead rubber bearings (LRBs) and natural rubber bearings (RBs). The dimensions and material properties of the bridge deck, piers with footings are given in Table 1 and those of the isolation bearings are presented in Table 2.



(b)

Figure 1. Physical model of a six-span continuous seismically isolated highway bridge (a) longitudinal sectional elevation of the bridge, and (b) transverse sectional elevation of the bridge; all dimensions are in [mm]

Table 1: Geometric and material properties of the bridge

Properties	Specifications	
	Piers with RBs	Piers with LRBs and HDRBs
Cross-section of the pier cap (mm^2), (B1x W1)	1500x9000	1800x9000
Cross-section of the pier body (mm^2), (B1xW2)	1500x6000	1500x5000
Cross-section of the footing (mm^2), (B3xW3)	5000x8000	5000x8000
Number of piles in each pier	4	
Young's modulus of elasticity of concrete(N/mm^2)	25000	
Young's modulus of elasticity of steel (N/mm^2)	200000	

Table 2: Properties of the isolation bearings

Dimension	Specifications		
	RBs	LRBs	HDRBs
Length (mm)	650.0	650.0	650.0
Width (mm)	650.0	650.0	650.0
Thickness of rubber layers (mm)	81.3	81.3	81.3

2.2 *Analytical Model*

The analytical model of the bridge is shown in Figure 2. The entire system of the bridge is approximated by a 2-D model bridge. The bridge deck is idealized as a rigid body ignoring flexibility of the bridge deck. The piers were restricted to participate in energy absorption in the entire bridge system to some extent in addition to the isolation bearings. The secondary plastic behavior was expected to be lumped at bottom of the piers where plastic hinges are occurred. The plastic hinges of the piers are modeled by nonlinear spring elements. The nonlinear spring elements are modeled using the tri-linear Takeda model (Takeda et al., 1970). The steel girder, the pier cap, the pier body, the footing, and the two ends of the plastic hinge are modeled using the simple elastic beam elements. The foundation is modeled by linear translational and rotational springs (soil-springs elements) to simulate the soil-foundation-structure interaction. The superstructure and substructure of the bridge are modeled as a lumped mass system divided into a number of small discrete segments. Each adjacent segment is connected by a node and at each node two degrees of freedom are considered: horizontal translation and rotation. The masses of each segment are assumed to be distributed between the two adjacent nodes in the form of point masses. The vertical displacement of the piers is restrained as no significant axial shortening is expected. In order to describe the mechanical behavior of isolation bearing, two types of analytical models of the bearings are used in the study: the rate dependent rheology model as developed by the authors (Bhuiyan et al. 2009; Bhuiyan, 2009) and the design models including the bilinear model and the equivalent linear model specified in JRA (2002). These two models are briefly discussed in the following sub-sections.

2.2.1 *Rheology Model*

The rheology model (Bhuiyan et al. 2009; Bhuiyan, 2009) employed in the subsequent numerical analysis is illustrated in Figure 3, where τ and γ are the average shear stress and shear strain of rubber layers, respectively. In this model, the total shear stress is decomposed into three contributions associated with a nonlinear elastic stress, an elasto-plastic stress and finally a viscosity induced overstress. The mathematical description of the model is briefly stated in Eq.(1).

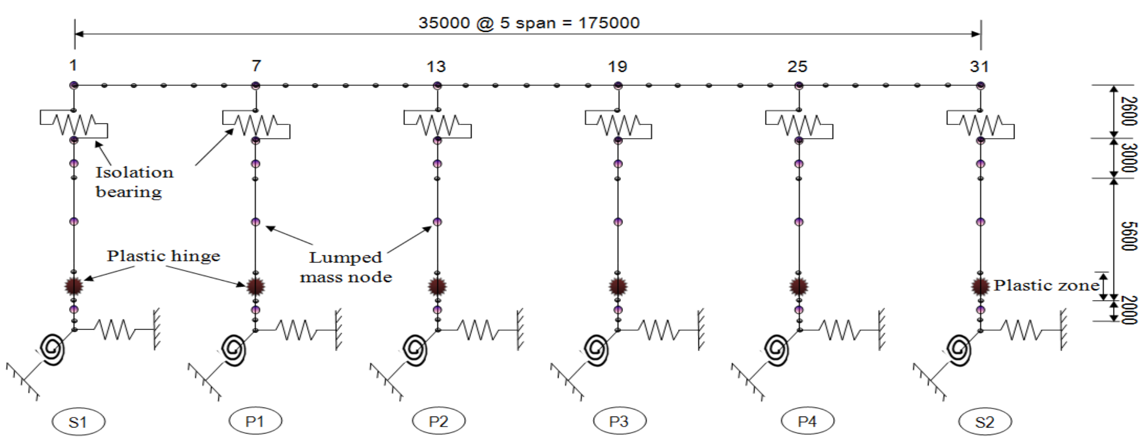


Figure 2. Analytical model of the bridge

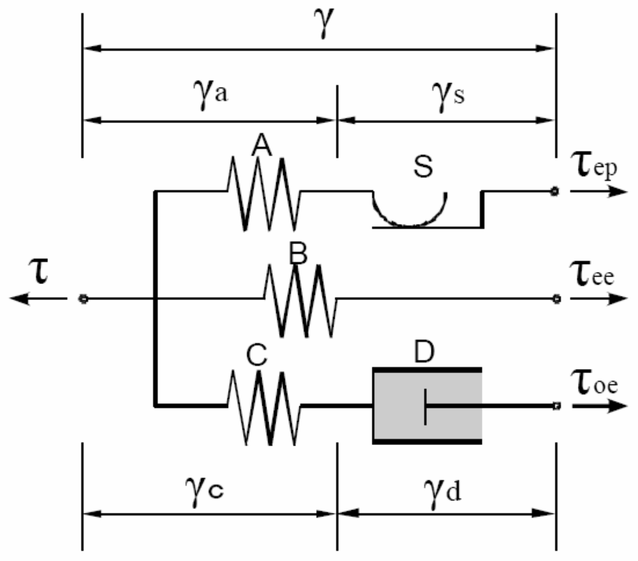


Figure 3. Rheology model of the isolation bearings

(a) Rheology model for HDRBs

$$\tau = \tau_{ep}(\gamma_a) + \tau_{ee}(\gamma) + \tau_{oe}(\gamma_c) \qquad (1a)$$

$$\tau_{ep} = C_1\gamma_a \quad \text{with} \quad \begin{cases} \dot{\gamma}_s \neq 0 & \text{for} \quad \left|\tau_{ep}\right| = \tau_{cr} \\ \dot{\gamma}_s = 0 & \text{for} \quad \left|\tau_{ep}\right| < \tau_{cr} \end{cases} \qquad (1b)$$

(1c)

(1d)

$$\tau_{ee} = C_2\gamma + C_3\left|\gamma\right|^m \mathrm{sgn}(\gamma)$$

$$\tau_{oe} = A\left|\frac{\dot{\gamma}_d}{\dot{\gamma}_o}\right|^n \mathrm{sgn}(\dot{\gamma}_d) \qquad \tau_{oe} = C_4\gamma_c \qquad (1e)$$

$$\text{with} \quad A = \frac{1}{2}\left(A_l \exp(q\left|\gamma\right|) + A_u\right) + \frac{1}{2}\left(A_l \exp(q\left|\gamma\right|) - A_u\right)\tanh(\xi\tau_{oe}\gamma_d)$$

where C_i (i = 1 to 4), τ_{cr}, m, A_l, A_u, q, n, and ξ are parameters of the model to be determined from experimental data .

(b) Rheology model for LRBs and RBs

For the rheology model LRBs and RBs, the parameter "A" in Eq. (1d) is understood to be constant on the basis of experimental results (Bhuiyan, 2009). The remaining equations (1a, to 1e) are the same as those for HDRs. The values of the parameters for HDR, LRB and RB used in the numerical analysis are listed in Table 3 and 4 respectively.

2.2.2 *Bilinear Model*

It is recognized that the isolation bearing has generally nonlinear inelastic hysteretic property. Some specifications have specified guidelines for using the bilinear model in order to represent the nonlinear inelastic hysteretic property of the HDRB and the LRB (AASHTO 2000; JRA 2002). In this case, three parameters are required to represent the hysteresis loop of HDRBs and LRBs: initial stiffness k_1, post yield stiffness k_2 and the yield strength of the bearings Q_d as shown in Figure 4. In the subsequent numerical study, these parameters are assigned for HDRB and LRB in accordance with the manual of bearings for highway bridges (JRA 2004) as given in Table 5.

2.2.3 *Equivalent Linear Model*

From experimental observations of RBs, it has been found that the force-displacement hysteresis loop of RBs can be approximated by the equivalent linear model (JRA 2002). Accordingly, the equivalent linear model is employed for RBs in the numerical analysis. The equivalent stiffness of the RB can be evaluated based on the nominal shear modulus G_e of the rubber material and the damping constant of the bearing is set to be 3.0%.

Table 3. Rate-independent response parameters of the bearings

Isolation bearing	C_1 MPa	C_2 MPa	C_3 MPa	C_4 MPa	τ_{cr} MPa	m
HDR1	2.401	0.535	0.002	2.805	0.205	8.182
LRB2	4.181	0.779	0.010	2.352	0.230	6.684
RB1	2.051	0.883	0.006	0.402	0.112	7.234

Table 4. Rate-dependent viscosity parameters of the bearings

Bear-ing /Pier	A_l MPa	A_u MPa	q	n	ξ
HDR 1	0.302	0.204	0.532	0.205	1.221
LRB2	0.322	0.322	--	0.302	--
RB1	0.082	0.082	--	0.232	--

Table 5. Parameters of the Bilinear model

Bearing	K_1 kN/mm	K_2 kN/mm	Q kN
HDR	17.60	1.67	34.2
LRB	11.31	1.74	34.5
RB	$K_B = 2.30$; $\xi = 3.0$ %		

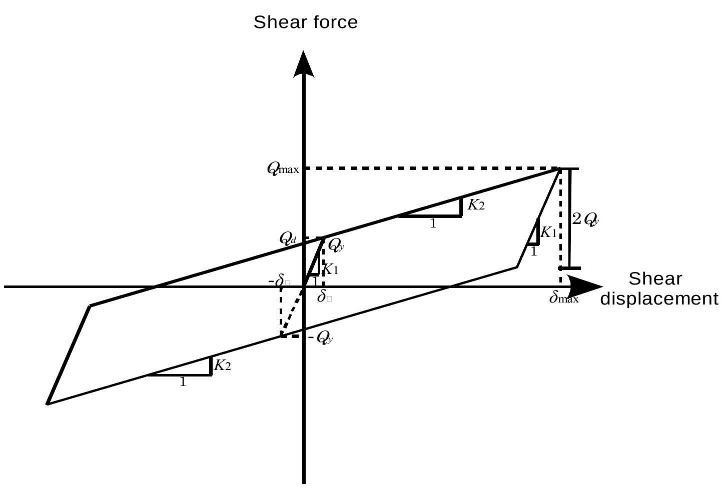


Figure 4: Bilinear force-displacement relationship of the bearings (JRA, 2002)

4 EQUATIONS OF MOTION AND SOLUTION ALGORITHM

Equations that govern the dynamic response of the bridge can be derived by considering the equilibrium of all forces acting on it using the d'Alermbert's principle. In this case, the internal forces are the inertia forces, the damping forces, and the restoring forces, while the external forces are the earthquake induced forces. The equations of motion in incremental form can be written as

$$\begin{aligned}&[\mathbf{M}]\{\Delta\ddot{\mathbf{U}}\}_{t+\Delta t}+[\mathbf{C}]\{\Delta\dot{\mathbf{U}}\}_{t+\Delta t}+[\mathbf{K}]\{\Delta\mathbf{U}\}_{t+\Delta t}+\{\Delta\mathbf{R}\}_{t+\Delta t}=\{\mathbf{P}\}_{t+\Delta t}-[\mathbf{M}]\{\ddot{\mathbf{U}}\}_t-[\mathbf{C}]\{\dot{\mathbf{U}}\}_t\\&-[\mathbf{K}]\{\mathbf{U}\}_t-\{\mathbf{F}_b\}_t-\{\mathbf{F}_s\}_t\end{aligned} \tag{2}$$

where [**M**] is the mass matrix;[**C**],the damping matrix;[**K**], the tangent stiffness matrix; $\{\Delta\mathbf{U}\}$, the vector of the increment of displacement over the time integration; $\{\Delta\dot{\mathbf{U}}\}_{t+\Delta t}$, the vector of the increment velocity over the time increment; $\{\Delta\ddot{\mathbf{U}}\}_{t+\Delta t}$, the vector of the increment of acceleration over the time increment; $\{\mathbf{U}\}_t$, the vector of the displacement at the beginning of the time step t; $\{\dot{\mathbf{U}}\}_t$, the vector of the velocity at the beginning of time step t; $\{\ddot{\mathbf{U}}\}_t$, the vector of the acceleration at the beginning of time step *t*; $\{\mathbf{F}_s\}_t$, the internal force of the bridge excluding isolation bearing at the time step *t*; $\{\Delta\mathbf{R}\}_{t+\Delta t}$, the total unbalanced force vector, and $\{\mathbf{P}\}_{t+\Delta t}$, the external force vector at the end of time step $t+\Delta t$; $\{\mathbf{F}_b\}_t$, the internal force vector derived from the isolation bearings at the beginning of the time step *t*. A solution algorithm comprised of the solution of equations of motion using the unconditionally stable Newmark's constant-average-acceleration method and the solution of the differential equation governing the strain-rate dependent behavior of isolation bearings is developed (Bhuiyan, 2009). The proposed algorithm has been successfully implemented in nonlinear dynamic time history analysis software. Furthermore, the Newton-Raphson iteration procedure consisting of corrective unbalanced forces is employed within each time step until equilibrium condition is achieved.

3. STRUCTURAL DAMPING

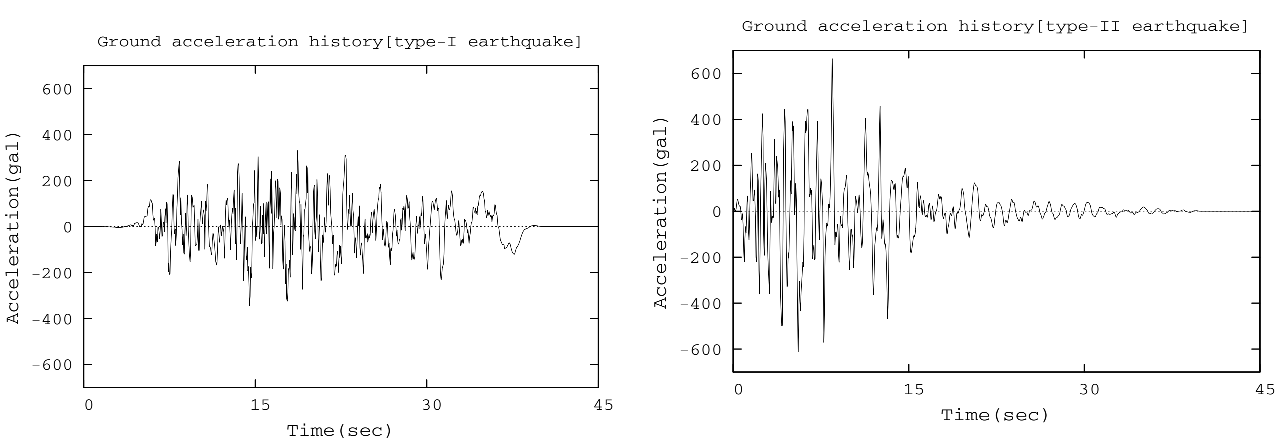
The damping constant matrix C for the bridge system is evaluated using the stiffness proportional damping model. The damping constant matrix is calculated by summing all the elements' equivalent damping constants. The elemental stiffness proportional coefficient is determined by using the elemental equivalent damping constant and first natural circular frequency of the system.

$$[\mathbf{C}] = \sum_{j=1}^{N} \frac{2h_j}{\omega_1}.k_j \tag{3}$$

where hj and kj are, respectively, the damping constant and stiffness matrix of the jth element and N is the number of elements of the bridge system. The elemental damping constants for the steel girder are taken as 0.02, for the concrete part and the foundation soil taken as 0.05 and 0.2, respectively (JRA 2002).

4. EARTHQUAKE GROUND MOTIONS

Two types of earthquake ground motions as recommended and specified in JRA (2002) are employed in the subsequent analysis. These two ground motions refer to type-I and type-II earthquake ground motions, which correspond to, respectively, a plate boundary type earthquake with large amplitude and long duration, such as the Kanto earthquake (Tokyo, 1923),and an inland direct strike type earthquake with low probability of occurrence, strong acceleration and short duration, such as the Kobe earthquake (Kobe, 1995). In order to consider the variation of the amplitude, phase characteristics of the ground motions, three design ground motion records for each type of earthquakes of moderate ground condition are applied in the longitudinal direction of the model bridge to evaluated the seismic responses. Figure 5 shows typical ground acceleration time histories for two types of earthquakes.



(a) (b)

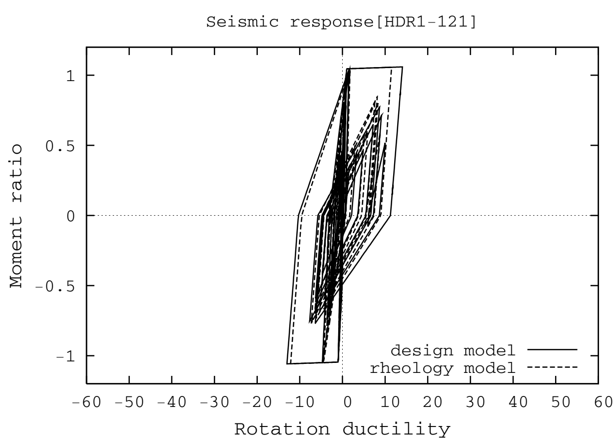
Figure 5. Ground acceleration histories used in the seismic analysis (a) the 123 Kanto earthquake ground motion, (b) the 1995 Kobe earthquake ground motion

5 SEISMIC RESPONSES OF BRIDGE

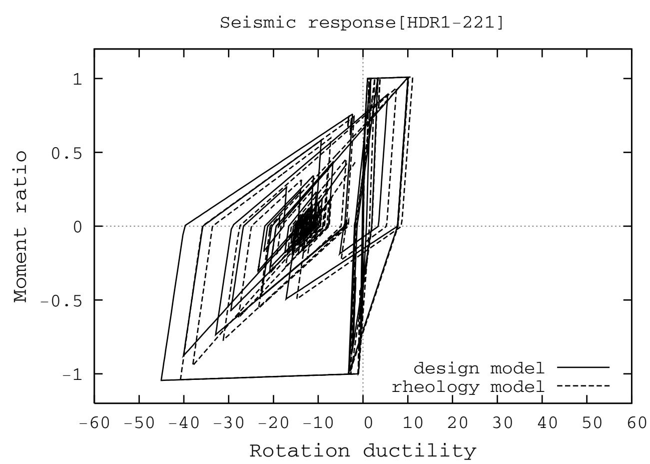
Before conducting nonlinear time history analysis of the bridge system, an eigenvalue analysis has been carried out to compute the vibration properties (natural frequencies and mode shapes of the bridge). Using the first natural frequency properties of the system, the damping matrix in Eq. (3) is obtained. A solution algorithm proposed by the authors (Bhuiyan, 2009) has been successfully implemented in commercially available software (Resp-T, 2006) in order to compute the seismic responses of the bridge using rate-dependent rheology model for isolation bearings. Due to symmetry of the bridge structure shown in Figure 1 and due to brevity, only one pier's results as obtained using three isolation bearings (HDR, LRB and RB) are graphically presented and discussed herein. Figures 6, 7 and 8 represent the moment-rotation relations of the plastic hinges of the pier for level-2 type-I and type-II earthquakes, respectively. The similar trend of the responses is obtained from the shear stress-strain relations of the bearings as shown in Figures 9, 10 and 11. The effect of modeling of isolation bearings on the responses of the bridge have been clearly appeared in comparisons of maximum shear strain (γ_{max}) occurred in the isolation bearings and the ratio of the maximum rotation to the allowable rotation of the plastic hinge experienced for type-I and type-II earthquakes waves respectively are shown in Table 5 and 6.

6 CONCLUDING REMARKS

Effect of modeling of bearings on the seismic responses of the isolated bridge is evaluated by conducting nonlinear dynamic analyses. Two different analytical models of the isolation bearings are used in the study for conducting a comparative assessment of the seismic responses of the isolated bridge system. These two models are design model specified in manual of bearings for highway bridges (JRA 2004) and the proposed rheology model. As the design model, the bilinear model is employed for modeling LRB and HDRB; and, the equivalent linear model for RB. It should be noted that a set of parameters corresponding to design models are estimated using the design equations as specified in JRA (2004), whereas the parameters of the proposed rheology model are estimated using experimental data conducted by the authors. In this paper, the bridge responses are discussed in terms of the moment-rotation relations of the plastic hinges and the shear stress-strain relations of the bearings, since these responses are very crucial for seismic design of bridge systems. The effect of modeling the bearings is significantly observed in the responses indicating that a careful selection of the models of isolation bearings is very important for seismic design of an isolated bridge system

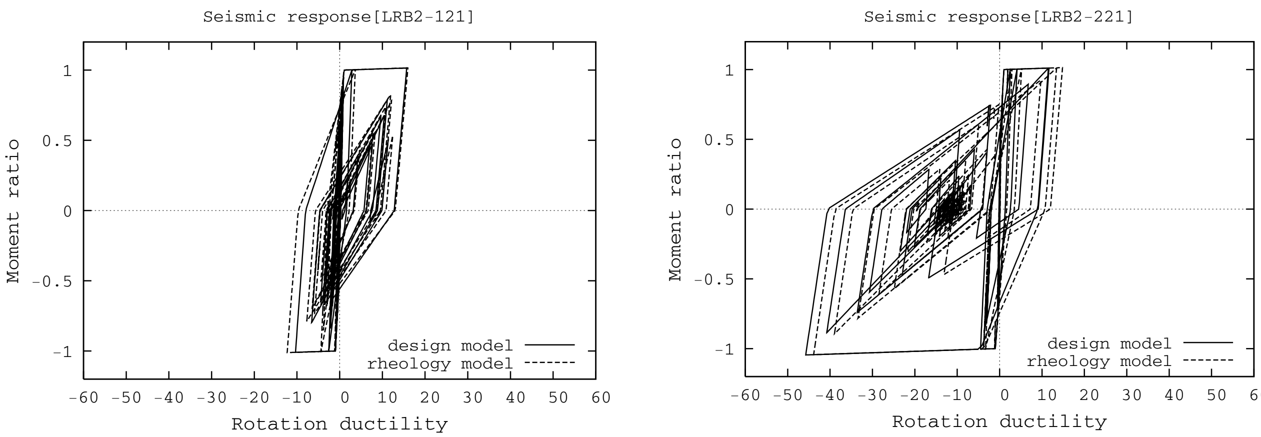


(a)



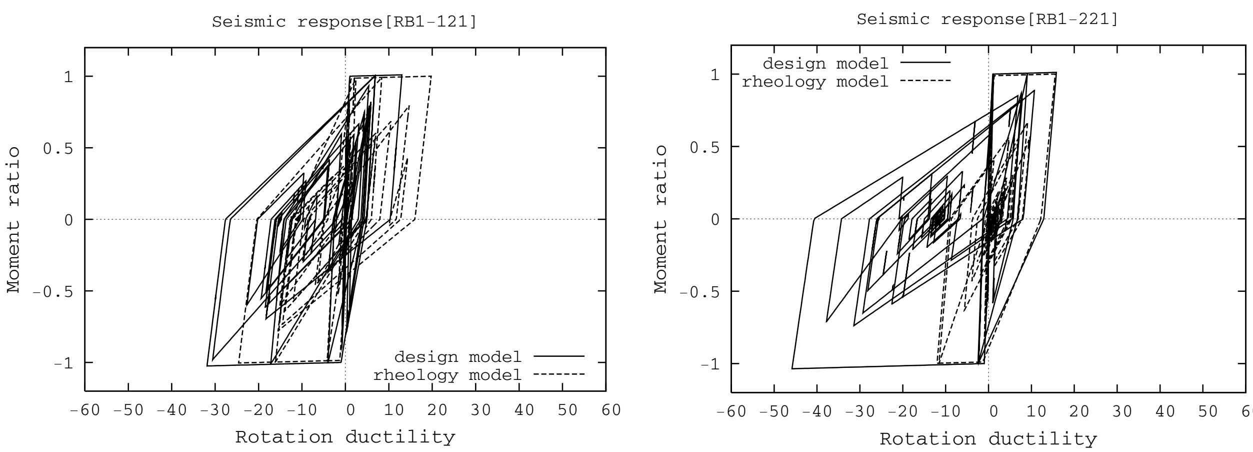
(b)

Figure 6. Moment-rotation responses as obtained HDR1 isolation bearing at the plastic hinge of the pier P1 (=P4) for level 2 (a) type-I and (b) type-II earthquake ground motions; moment ratio is the bending moment at the level of the plastic hinge divide by the yield moment and rotation ductility refers to the rotation of the pier at plastic hinge level divide by the yield rotation.



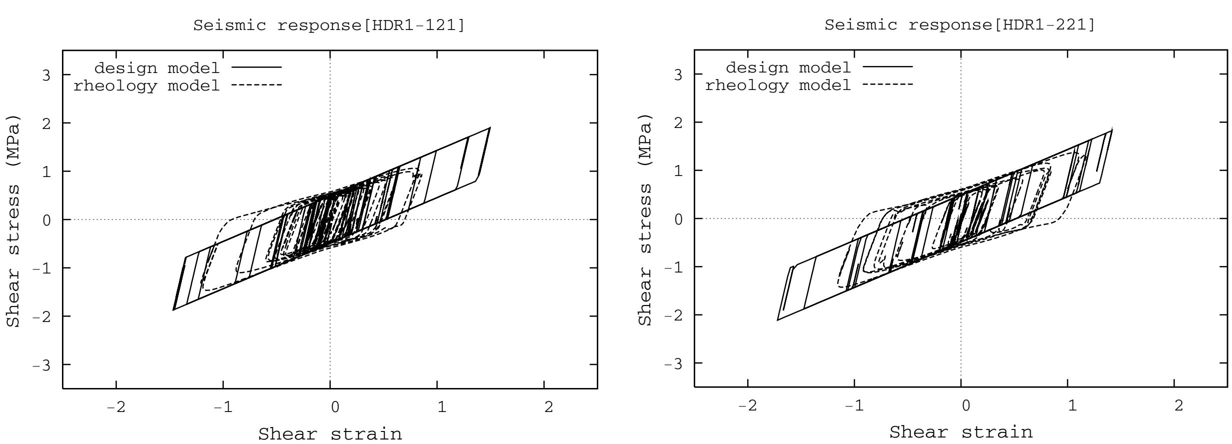
(a) (b)

Figure 7. Moment-rotation responses as obtained LRB2 isolation bearing at the plastic hinge of the pier P1 (=P4) for level 2 (a) type-I and (b) type-II earthquake ground motions; moment ratio is the bending moment at the level of the plastic hinge divide by the yield moment and rotation ductility refers to the rotation of the pier at plastic hinge level divide by the yield rotation



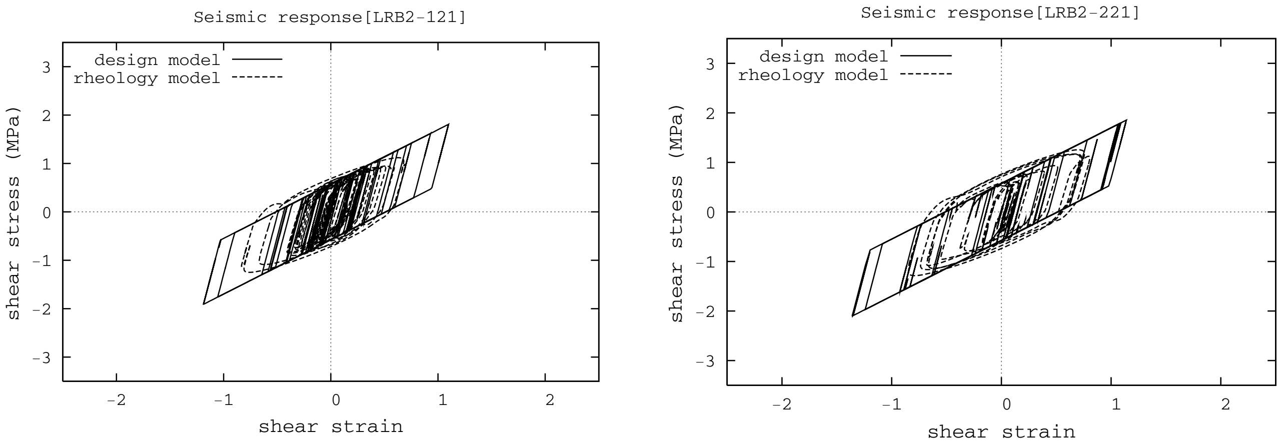
(a) (b)

Figure 8. Moment-rotation responses as obtained RB1 isolation bearing at the plastic hinge of the pier P1 (=P4) for level 2 (a) type-I and (b) type-II earthquake ground motions; moment ratio is the bending moment at the level of the plastic hinge divide by the yield moment and rotation ductility refers to the rotation of the pier at plastic hinge level divide by the yield rotation.



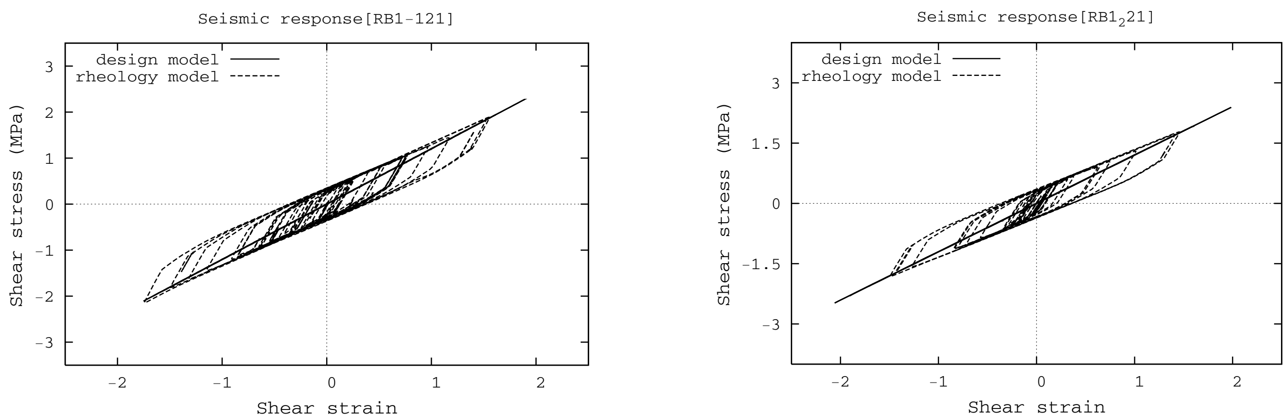
(a) (b)

Figure 9. Shear stress-strain responses as obtained for HDR1 at top of the bearing of the pier P1 (=P4) for level 2 (a) type-I and (b) type-II earthquake ground motions; shear stress is the horizontal shear force divided by the cross-sectional area of the bearings and shear strain is relative displacement of the top of the bearing divided by the total height of the rubber layers of the bearings.



(a) (b)

Figure 10. Shear stress-strain responses as obtained for LRB2 at top of the bearing of the pier P1 (=P4) for level 2 (a) type-I and (b) type-II earthquake ground motions; shear stress is the horizontal shear force divided by the cross-sectional area of the bearings and shear strain is relative displacement of the top of the bearing divided by the total height of the rubber layers of the bearings



(a) (b)

Figure 11. Shear stress-strain responses as obtained for RB1 at top of the bearing of the pier P1 (=P4) for level 2 (a) type-I and (b) type-II earthquake ground motions; shear stress is the horizontal shear force divided by the cross-sectional area of the bearings and shear strain is relative displacement of the top of the bearing divided by the total height of the rubber layers of the bearings

ACKNOWLEDGEMENTS

The experimental works were conducted by utilizing the laboratory facilities and bearings-specimens provided by Japan Rubber Bearing Association. The authors indeed gratefully acknowledge the kind cooperation extended by them.

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