Seismic Design and Performance of Two Isolation Systems Used for Reinforced Concrete Bridge Construction

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SUMMARY:

An analytical study is performed to evaluate the seismic performance of typical California reinforced concrete overpass bridges enhanced by seismic isolation using bearings located between the bridge superstructure and the columns. The two seismic isolation bearing technologies considered are: lead rubber bearings and friction pendulum bearings. For the friction pendulum system, single and double concave bearings are investigated. A typical five-span, single column-bent, reinforced concrete bridge is redesigned using these two technologies and modeled using the OpenSees framework. Nonlinear response history analyses using three-component ground motions from past and recent earthquakes around the world are carried out for the different bridge systems. The dynamic analysis results demonstrate that both bridge isolation systems are effective in reducing the displacement and force demands on the bridge piers and the supporting foundations. The seismic performance of the two isolation systems is compared in terms of force and displacement demands on key bridge components.

Keywords: Lead Rubber Bearings; Friction Pendulum Bearings; Bridge Isolation; Reinforced Concrete Bridge.

1. INTRODUCTION

The focus of conventional bridge design practices is to provide sufficient strength and ductility capacities to the substructure components (columns, foundation, bearings, expansion joints and abutments) to meet the seismic performance requirements. Despite better detailing and confinement in modern bridges leading to enhanced damage tolerance and reduced collapse susceptibility, significant damage to bridge infrastructure has occurred following large earthquakes in the U.S., Japan, and other countries around the world. Such damage requires extensive repair or complete replacement of the bridge columns and superstructure. To guarantee post-earthquake serviceability and reduce the repair costs of highway bridge systems following earthquake disasters, research efforts in recent years have been directed towards the development and implementation of innovative materials, supplemental damping and energy-dissipation mechanisms, and seismic response modification techniques for new and existing bridge structures (Grant et a. 2004; Constantinou et al. 1999; Mosqueda et al. 2004; Buckle et al. 2006; Kelly and Konstantinidis 2011). One of these seismic performance enhancement strategies, particularly effective for sites with high seismicity or directivity effects, is bridge seismic isolation, which, when properly designed nearly eliminates damage in the bridge substructure. This system allows for continuous operation of critical transportation routes and circulation of emergency traffic after a major event with zero downtime and zero direct or indirect post-earthquake repair costs.

The rapid growth of seismic isolation technology led to the development of specific guidelines for design, construction, and testing of different isolation devices, as well as the analysis of isolated bridge structures (AASHTO 1999; Buckle et al. 2006). Among the seismic isolation devices commonly used are elastomeric bearings such as natural rubber, high-damping rubber, or lead-plug rubber bearings, and sliding bearings such as flat, single, or multi-concave friction-pendulum (FP) bearings. The effectiveness of isolation devices in uncoupling the bridge substructure from the horizontal components of ground motion excitation and therefore reducing its force and displacement



demand has been thoroughly assessed through numerous experimental and analytical research studies (Zayas et al. 1987; Mosqueda et al. 2004; Grant et al. 2004; Warn and Whittaker 2006). The lead-plug rubber bearing device is the most commonly used system in bridge applications worldwide due to its relatively simple design, low fabrication cost, and low maintenance requirements. The FP bearings also have good energy dissipation characteristics by means of friction and a desirable gravitational restoring force (due to uplift of the structure) that minimizes residual displacements.

An analytical study is carried out to evaluate and compare the improved seismic performance of typical reinforced concrete bridges in California. Enhancement of seismic performance is achieved by isolating the bridges using two common seismic isolation bearing types, lead rubber bearings and friction pendulum bearings, located beneath the bridge superstructure. For the FP system, single and double concave bearings are investigated. A typical five-span, single column-bent, reinforced concrete bridge is redesigned using these two technologies and then modeled using the OpenSees structural analysis software. The target performance states of the isolated bridges include elastic column and elastic deck behavior, zero uplift at the columns, adequate gap size at the abutments, and stable response of the isolators. The design procedure and modeling approaches of these bridges are also described in this paper.

Nonlinear response history analyses using three-component ground motion records from a wide range of past and recent earthquakes around the world are carried out for the different bridge systems. The seismic performance of the bridges is assessed and compared in terms of different engineering demand parameters on major bridge components.

2. BRIDGE DESIGN AND MODELING

The following section summarizes the general design scheme, basic assumptions, final dimensions, and material properties used for the three-dimensional, nonlinear OpenSees models of the fixed-base conventionally-reinforced concrete bridge (RC), seismically isolated bridge with lead rubber bearings (LRB1), and seismically isolated bridges with single concave (FP1) and double concave (FP2) friction pendulum bearings. The bridge models are implemented using the OpenSees structural analysis framework following the modeling and analysis recommendations by Aviram et al. (2008; 2010a).

2.1. Benchmark RC Bridge

The RC bridge consists of an Ordinary Nonstandard reinforced concrete bridge with box-girder superstructure, typical column bent details, and simple geometric regularity (symmetry, zero skew, and uniform column height), designed according to AASHTO Standard Specifications for Highway Bridges (AASHTO 1996) and Caltrans Seismic Design Provisions (Caltrans 2004). The geometry of the RC bridge corresponding to bridge Type 1A by Ketchum et al. (2004) is presented in Fig. 1. The fixed-base periods of the RC bridge, obtained from the OpenSees model (with uncracked column and effective superstructure properties), are 0.95 seconds for the transverse translation mode, 0.53 seconds for the longitudinal translation mode, and 0.56 seconds for the global torsion mode of the bridge deck about a vertical axis.

The RC bridge superstructure is modeled using elastic beam-column elements and effective crosssection properties per Caltrans SDC (2004). The continuous superstructure is modeled using two segments for each span. It is rigidly connected to the columns, transmitting gravity loads, lateral forces and bending moments. The OpenSees model of the column is a single beam-column finite element with a distributed plasticity fiber model, nonlinear force formulation and five integration points. Expected material strengths specified in Caltrans SDC (2004) are used for all steel and concrete elements and fibers. The concrete fiber constitutive model is *Concrete02*, which implements the Kent-Scott-Park behavior and includes tensile strength. The steel fibers utilize *Steel02*, which implements the Giuffre-Menegotto-Pinto behavior with ultimate strains specified according to Caltrans SDC (2004) and softening post-yield behavior. Rigid offsets are defined at the top of the column element to account for the column-to-superstructure rigid connection. Lumped translational and rotational tributary masses are assigned to each node of the substructure and superstructure. The self-weight of the bridge and P-Delta effects are considered in the dynamic analysis. The column foundations are modeled as fixed boundary conditions. An elaborate abutment model (Aviram et al. 2010a), denoted as the *SpringAbutment* model, is used for the deck ends. This abutment model includes complex longitudinal, transverse, and vertical nonlinear abutment response, as well as a participating mass corresponding to the concrete abutment and mobilized embankment soil.



Figure 1. Geometry of the fixed-base RC bridge Type 1A (Ketchum et al. 2004)

2.2. General Design and Modeling Scheme for Isolated Bridge

Seismic isolation is a response modification technique that allows reducing the force and displacement demand on the bridge substructure through stable lateral deformation of the isolation bearings. The elastomeric and FP isolation systems considered in this study minimize the interaction between the superstructure and substructure by providing lateral flexibility (period shift) and energy dissipation through hysteretic behavior. The Type 1A bridge was redesigned by inserting the isolation bearings beneath the deck. Two bearings are placed at each column, redesigned with increased diameter and longitudinal reinforcement to ensure elastic behavior. A column cap beam is used to facilitate bearing installation, as shown in Fig. 2. For the LRB1 bridge, three bearings are specified at the abutment, one underneath each web of the box-girder superstructure. For FP1 and FP2 bridges, only two bearings are used at the abutments in order to maintain a minimum level of axial load on the bearing and reduce uplift.

In general, seismic isolator design is an iterative procedure, where the structural performance determines the isolator parameters, which in turn affect the overall structural performance. The preliminary design of the isolated bridge is carried out following the AASHTO Guide Specifications for Seismic Isolation Design (AASHTO 1999), the AASHTO Standard Specifications for Highway Bridges: Division IA-Seismic Design (AASHTO 1996), and the Caltrans Seismic Design Criteria V1.3 (Caltrans 2004). The simplified analysis of the isolated bridge is performed using the Uniform Force Method of the AASHTO (1999) code to determine of the seismic demand values on the columns and the isolation devices. The SDC (Caltrans 2004) was used for the preliminary design of the columns. The target design performance of the bridge is to achieve a period shift to 3.0 seconds in both the transverse and longitudinal directions, an equivalent viscous damping coefficient of 20%, adequate bearing displacements, and column behavior not exceeding the elastic limit under a high seismic

demand per SDC 2004. The DIS manuals (Dynamic Isolation Systems 2007) are used for the selection and design of the elastomeric bearings. For the Uniform Force Method analysis procedure, the effective stiffness and damping characteristics of the isolators are used to design of the system. The final designs of the different isolated bridge systems is obtained by adjusting the parameters defining the nonlinear hysteretic behavior of the isolators using nonlinear dynamic response history analyses of the bridges, which included a large representative suite of three-component ground motions scaled to different seismic hazard levels.

The modeling scheme of the isolated bridges in OpenSees, presented in Fig. 2, includes a system of rigid links and top and bottom cap beams connecting the superstructure, isolation bearings, and columns. Their location is defined according to the centerline of these elements and the spacing of the isolators. The centerline location of the isolators is determined in accordance with their horizontal dimension and the superstructure bottom flange width, allowing for clear distance from the deck edges. The torsional stiffness of the top bent cap accounted for the monolithic construction of the superstructure and the cap beam system, while its remaining dimensions are consistent with the superstructure cross section (Aviram et al. 2010b). The bottom cap beam is checked for shear, flexure, and torsion. No releases are provided in the model for the superstructure-to-substructure system. Tributary translational and rotational masses are assigned to each node of the bridge system.



Figure 2. Schematic configuration of the isolated bridge models in OpenSees

The OpenSees abutment model for the isolated bridges, denoted as the *IsolatorAbutment* model, is similar to the *SpringAbutment* model used for the RC bridge. The uncoupled elastomeric bearings are replaced by the bearing elements in OpenSees corresponding to each isolation system. These are described in further detail in the following sections. To allow lateral displacement of the deck, the size of the longitudinal gap is increased and an additional compression-only gap is provided in the transverse direction, defined according to the maximum lateral displacement, d_{max} specified for the isolators. The shear keys and embankment mobilization in the transverse direction interacts with the superstructure and contributes to the shear resistance following gap closure. For the LRB1 bridge, the shear capacity of the isolators at the abutments is defined as 2/3 of the capacity of the isolators at the piers (since three isolators are used instead of two) to obtain a similar shear strength at all isolator locations. For FP1 and FP2 bridges, two isolators are used so that the axial loads per bearing are not too low. The abutment isolator geometry and mechanical properties at the remaining degrees of freedom is defined similarly to the column isolators for the LRB and FP bearings.

2.3. Base Isolated Bridge LRB1 with Lead Rubber Bearings

The first type of bridge isolation bearings considered in this study, commonly used for bridge isolation in North America, are the lead-plug rubber bearings (LRBs). LRBs consist of alternating layers of

steel and rubber providing horizontal flexibility while maintaining sufficient vertical stiffness. The lead core in the center of the bearing provides supplemental damping.

The LRB isolators used in LRB1 bridge are modeled in OpenSees using the *elastomericBearing* element developed and implemented for this study. This element has a bilinear force-deformation response envelope and circular interaction surface for horizontal shear forces. The remaining degrees of freedom of the element are assigned distinct uniaxial materials. The nonlinear parameters for the LRBs are selected from the manufacturer design values (Dynamic Isolation Systems 1997). They include the force, Q_d at zero displacement under cyclic loading, the post-yielding isolator stiffness, K_d (set equal to the stiffness of the rubber bearing alone), the elastic stiffness, K_e at monotonic loading (equal to the unloading stiffness at cyclic loading), and the yield force, F_y under monotonic loading. The resulting LRB column isolator dimensions and hysteretic parameters are presented in Fig. 3.



Figure 3. (a) Cross section configuration and (b) idealized force-deformation relationship for the LRB column isolator of LRB1 bridge model

For this study, the failure displacement of the bearings was defined at 300% shear strain, equal to 49.5 inches. The targeted equivalent damping coefficient for the LRB1 bridge model is equal to 20%. The equivalent damping coefficient is calculated as the energy, E_D dissipated per cycle at the design displacement, d_{max} (equal to the area enclosed by the actual hysteresis loop), divided by the energy dissipated by an equivalent elastic system with effective stiffness, $K_{eff;max}$. For an axial load ratio, P/P_{cr} of the isolator with respect to its critical buckling load not exceeding a value of 0.3, no significant reduction in the shear resistance is expected (Kelly and Konstantinidis 2011). Therefore, horizontal-vertical interaction is not included in the *elastomericBearing* element in OpenSees, consistent with the computed conditions of low axial load levels on the LRB1 isolators (P/P_{cr} < 0.01).

The axial, rotational, and torsional degrees of freedom are defined using linear-elastic material behavior. The axial stiffness is defined as the compression stiffness of the bearing, $K_v = 11,000$ kip/in, according to the design values provided by the manufacturer (Dynamic Isolation Systems 1997). The rotational stiffness, $K_{\theta} = 834$ kip-in/rad, and torsional stiffness, $K_T = 18,770$ kip-in/rad, are approximated following the recommendations by Kelly and Konstantinidis (2011). The *elastomericBearing* element used for the isolated bridge models is defined with a finite length corresponding to the actual height, h_i of the bearing, excluding the steel end plates. P-Delta considerations are included for this element for finite-length isolator segments. More details on the LRB design for the LRB1 bridge can be found elsewhere (Aviram et al. 2010b).

2.4. Base Isolated Bridges FP1 and FP2 with Friction Pendulum Bearings

The friction pendulum (FP) bearing is an axisymmetric concave sliding device that combines high energy dissipation characteristics, and a gravitational restoring force mechanism that allows minimizing residual displacements of the supported structure under ground shaking (Zayas et al. 1987). Energy dissipation occurs through friction between two or more sliding interfaces in the bearing, while the restoring force is produced by the self-centering action of the structure sliding on

the concave spherical surface under gravity. The FP bearing parameters defining its dynamic behavior such as period, damping, vertical load capacity, and displacement capacity, can all be selected independently. The period of the FP bearing depends solely on the effective radius of curvature, R_{eff} of the concave surfaces and is independent of the weight of the supported structure and vertical seismic loads. The effective radius for the bearings in FP1 and FP2 bridges is selected to attain a target isolated period of 3.0 sec in both transverse and longitudinal directions, which is approximately three times the fundamental transverse translation period of the fixed-base structure. Typical coefficients of friction, μ , between the unlubricated PTFE slider interface and the highly polished stainless steel concave surface material, commonly used in FP bearings, range between 0.04 and 0.12 for high sliding velocities.

The compression stiffness of the FP bearings is typically about 7 to 10 times greater than the one of elastomeric bearings and is maintained even at their design lateral displacement. The vertical stiffness of the single FP (SFP) and double FP (DFP) bearings in bridges FP1 and FP2, is set at $K_v = 100,000$ kip/in. The higher vertical stiffness of the FP bearings results in a shorter vertical period and reduced uplift displacement demand on the bearings. The FP bearings maintain the vertical support at the center of the columns. A minimum and maximum pressure of 2 and 10 ksi are defined for the FP bearings in this study. This is required to ensure stability of the friction coefficient per manufacturer recommendations. Zero stiffness is assigned to the bearings in the three rotational degrees of freedom. Because the friction force and lateral stiffness of the bearing are directly proportional to the supported weight, their center always coincides with the center of mass. Thus, the FP bearings automatically adjust for accidental mass eccentricities and minimize torsion in the supported superstructure.

2.4.1 Single friction pendulum (SFP) bearing in the FP1 bridge model

Fig. 4 show the cross section of the SFP bearing designed for the FP1 bridge. The SFP isolators used in FP1 bridge are modeled in OpenSees using the *singleFPBearing* element developed and implemented for this study. The *singleFPBearing* element is defined using a bilinear lateral forcedeformation response, as shown in Fig. 4b under tributary weight, with circular yield surface. Because the yield (friction) force and the hardening stiffness depend on the time-varying axial load in the bearing, vertical and lateral degrees of freedom are coupled. Rayleigh damping was not included in the element, only hysteretic damping.



Figure 4. (a) Cross section configuration and (b) idealized force-deformation relationship for the single-concave FP isolator of the FP1 bridge model

The friction force at the sliding interface depends mainly on the PTFE composite type, sliding velocity, displacement path, pressure, and temperature. A simple modeling, shown in Eqn. 2.1, approach was used to define the dynamic coefficient of friction in terms of the sliding velocity (Constantinou et al. 1990). This modeling approach was experimentally investigated to verify that it adequately captures the velocity dependent behavior of FP bearings (Mosqueda et al. 2004).

$$\mu = f_{\max} - (f_{\max} - f_{\min}) \cdot e^{-a|v|}$$
(2.1)

In this equation: f_{max} and f_{min} are the coefficients of friction at high and low velocities, respectively, *a* is a constant that describes the rate of transition from low to high velocities, and |v| is the magnitude of the instantaneous velocity vector. For a coefficient of friction μ of 0.08, an effective damping coefficient of 15% is calculated at the design displacement, d_{max} for the FP1 bridge model. The friction force is defined using the velocity-dependent friction model in OpenSees with $f_{max} = 0.08$ and $f_{min} = 0.02$. The yield deformation of the composite PTFE liner when friction is overcome and sliding occurs is estimated to be on the order of 0.01 inches (Constantinou et al. 1999).

The SFP bearings can be installed with the concave surface facing either up or down. The articulated joint allows relative rotations between the structure above and below the isolators, and reduces the isolator moment loads on the structure. In the selected configuration of the SFP bearing in the FP1 bridge shown in Fig. 4, where the concave surface is facing down, no P-Delta moments are transferred to the structural members below the isolator. This reduces the seismic forces transmitted to the columns and foundations, which are the most vulnerable components of the bridge. The superstructure, typically designed to remain elastic with large safety factors, will absorb the P-Delta moments produced in the SFP bearing.

The effect of the rotation of the concave sliding surface on the hysteretic loop of the bearing is also included in the isolator model. For the SFPs in the FP1 bridge mode, the rotation of the superstructure results in a shift of the equilibrium position of the bearing, since the slider tends towards the surface location tangent to the horizontal. For small angle approximations, the normal force on the bearing can be approximated using the vertical force. In the bearing local axial (vertical) direction, a gap behavior is defined to simulate the zero tensile capacity of the FP bearings, and supplemental viscous damping is provided for numerical stability.

2.4.2 Double friction pendulum (DFP) bearing in the FP2 bridge model

The double pendulum bearing incorporates two pendulums in one bearing. The properties of each of the pendulums are chosen to become sequentially active at different earthquake intensities. The properties of the first top concave dish and second bottom concave dish are chosen to protect the bridge substructure under design and maximum considered earthquake levels, respectively. Thus, the first top pendulum is selected with a lower coefficient of friction to absorb seismic displacements at lower earthquake intensities. Sliding on the second concave surface is triggered during low frequency, large displacement ground motions produced by stronger earthquakes. The DFP bearing typically results in plan dimensions that are nearly half those of the SFP bearing, and allows for easier design of the isolation system. However, P-Delta moments in the DFP bearing are distributed to both the superstructure and the column top, proportional to the amount of sliding on each concave surface.

Fig. 5 shows the cross section of the double friction pendulum (DFP) bearing designed for the FP2 bridge. It can be modeled using two *singleFPBearing* elements with different radii and coefficients of friction, or using a single element with an effective radius and coefficient of friction.



Figure 5. (a) Cross section configuration and (b) idealized force-deformation relationship for double concave FP isolator for the FP2 bridge model

The friction coefficients and radii selected for FP2 bridge are displayed in Fig. 5a for the different concave surfaces, and the idealized piece-wise linear lateral force-deformation relationship under tributary weight is presented in Fig. 5b. The effective pendulum length, L_n of each concave surface is computed as the corresponding radius, R_n subtracted by the distance, h_n between the sliding surface and the bearing mid-height. An effective damping coefficient of 18% is calculated at the design displacement, d_{max} for the FP2 bridge model.

3. NONLINEAR RESPONSE HISTORY ANALYSIS RESULTS

Nonlinear response history analyses are carried out on three-dimensional OpenSees models of the fixed-base RC bridge and the isolated bridges LRB1, FP1, and FP2. A uniform ground motion excitation is applied at the base of the bridges using 140 three-component records covering a wide range of earthquake magnitudes and fault distances, as well as different faulting mechanisms (Aviram et al. 2010b). The seismic response of the different bridge models is compared by relating selected Engineering Demand Parameters (EDP)s, obtained from nonlinear response history analyses, to a period-independent intensity measure (IM) peak ground velocity (PGV) for each ground motion record. A natural logarithm fit is used to describe this relation. The PGV was selected as the IM used in this study because base-isolated bridge structures are vulnerable to large pulse-like ground motions (characterized by high incremental PGV values) recorded at near-fault locations (Makris and Chang 2000). The PGV value for each three-component ground motion, obtained as the SRSS combination of the PGV values of the two orthogonal horizontal components of the record, is an adequate IM for structures with fundamental first-mode period in the constant-velocity range of the response spectra.

Selected median EDP-IM relations for the different bridges are presented in Fig. 6. The dispersion of the data is also presented. The selected range of PGV values comprises low to high earthquake intensities. The regressions provide important insight into the effect of using different isolation devices on the overall behavior of the bridge response parameters as a function of earthquake intensity. Due to the large scatter of the nonlinear dynamic analysis results, the regressions provide a general tendency for bridge response, not the exact relations between these parameters and earthquake intensity.

As seen in Fig. 6, due to the lower yield strength of the SFP and DFP bearings, the column is better protected in the FP1 and FP2 bridges in comparison to the LRB1 bridge. This is reflected by lower displacement ductility and shear force demands. Conversely, more energy is dissipated in the LRB1 bridge than the FP bridges. The increase in the displacement demand with increasing PGV in the FP bearings is more pronounced than in the LRB bearings. This trend is due to a combination of two effects. First, at smaller ground motion intensities the FP bearings displace less than the LRB bearings because of the large initial stiffness of the FP bearings. Second, at larger ground motion intensities the FP bearings experience uplift, which reduces resistance and increases sliding displacements. The columns in the fixed-based RC bridge are designed as a fuse to limit the force demand and thus protect the remaining bridge components, primarily the bridge superstructure. The yielding and corresponding formation of plastic hinges in the column at relatively low earthquake intensities results in a nearly constant shear force demand for all levels of ground shaking. The isolated systems approach the plastic strength of the column as the column displacement ductility demand approaches the elastic limit, suggesting column yield is possible for stronger motions. The column in all seismic isolation bridges is redesigned using larger column diameters and longitudinal reinforcement ratios to ensure elastic behavior.

The FP bearing design in the FP2 bridge model has a similar yet more desirable seismic performance as compared to the FP1 bridge. The DFP bearing design is more compact, provides more flexibility in adjusting the hysteretic behavior, and has higher damping than the SFP design. Despite some P-Delta moments to the top of the column, the overall behavior of the FP2 bridge results in lower column top rotations than in the FP1 bridge. The two seismic isolation systems are similarly effective in significantly reducing vertical curvatures and vertical accelerations in the bridge deck. Nearly identical compression demands are computed for the two seismic isolation systems. Because bearing tension forces are produced in the LRB design and bearing uplift is generated in the FP designs, the stability of the lateral resistance of the two systems differs significantly.



Figure 6. EDP-IM relations for the analyzed bridge models

4. CONCLUSIONS

A typical five-span, single-column bent reinforced concrete bridge in California with box-girder superstructure and no geometric irregularities is redesigned using two isolation systems placed beneath the superstructure, i.e., elastomeric lead rubber bearings and friction pendulum bearings. Single and double concave pendulum bearing designs are developed. The target performance of the isolated bridges includes elastic column behavior and controlled bearing displacements under high seismic

intensity, as well as a significant shift in the translational periods of the bridge systems. Detailed nonlinear three-dimensional models of these bridge systems were implemented in the OpenSees structural analysis software. The comparison of the enhanced performance of the isolated bridge systems is carried out in terms of key engineering demand parameters on different bridge components obtained from nonlinear response history analysis results using 140 three-component ground motions.

In general, the efficiency of bridge isolation in protecting the vulnerable substructure is evident for the elastomeric and friction pendulum designs. Significant reductions in force and displacement demands are obtained at the substructure, redesigned to accommodate the isolation system. Thus, both bridge isolation systems lead to trivial post-earthquake repair efforts in an actual bridge, allowing for continuous operation of the structure and negligible indirect costs resulting from down time. However, important differences are observed between the two isolation systems in terms of the geometry, hysteretic behavior, energy dissipation characteristics, dependency on fluctuating axial load, tension resistance, vertical and rotational stiffness, distribution of P-Delta moments, and stability of lateral resistance with pressure and ground motion velocity. These differences in isolator behavior affect the response of the remaining bridge components and the overall seismic performance of the bridges.

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