Study on Nonlinear Seismic Response of Curved Highway Viaducts with Different Cable Restrainers

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This paper presents an in-depth analysis to evaluate the efficiency of using cable restrainers connecting isolated and non-isolated spans for preventing collapse of curved highway viaducts. For this purpose, the overall three-dimensional nonlinear bridge response is examined in detail under the action of four near-fault earthquake ground motions. The expected seismic vulnerability of bridge structures with curved deck geometries has been demonstrated, providing a refined estimation of seismic demands on most critical bridge components. The advantage of using a precise three-dimensional model has revealed the concentration of large seismic forces to specific steel bearing supports that greatly increase their possibility to failure. Moreover, the significantly unbalanced distribution of pounding forces found across the expansion joint may cause local damage to colliding girders and high impact forces transmitted to the bearing supports. It is concluded that the combination of longitudinal and transverse cable restrainers is the most effective configuration in order to minimize the possibility of deck unseating and reducing the pounding forces at the expansion joint, without significantly increasing ductility demands in the bridge piers.

Keywords: Curved highway viaduct, unseating prevention system, isolation bearings, near-fault earthquake

1. Introduction

Recent strong earthquakes have repeatedly demonstrated the seismic vulnerability of multi-span simply-supported bridge structures[1]. It is widely recognized that one of the primary causes of bridge collapse during earthquakes is due to unseating of deck superstructures[2]. This catastrophic result occurs when the seismically induced relative displacement between the deck and the supporting substructure exceeds the available seat width. Furthermore, the rupture of continuity of the superstructure at expansion joints substantially increases the susceptibility of simply-supported bridges to structural damage associated with pounding between adjacent spans due to the transfer of large seismic force from deck to deck, which results in damage of bearing supports and piers[3].

As a result of the implementation of modern seismic protection technologies, bridges can be seismically upgraded through the installation of cable restrainers that provide connection between adjacent spans. The purpose is to prevent the unseating of decks from top of the piers at expansion joints by limiting the relative movements of adjacent bridge superstructures. Moreover, cable restrainers provide a fail-safe function by supporting a fallen girder unseated in the event of a severe earthquake[4].

In addition, another commonly adopted earthquake protection strategy consists of replacing the vulnerable steel bearings with isolation devices. Among the great variety of seismic isolation systems, lead-rubber bearing (LRB) has found wide application in bridge structures. This is due to their simplicity and the combined isolation-energy dissipation function in a single compact unit. LRB bearings are steel reinforced elastomeric bearings in which a lead core is inserted to provide hysteretic damping as well as rigidity against minor earthquakes, wind and service loads. The lead core yields at relatively low shearing stress resulting in significant dissipation of seismic energy and reduction of earthquake response[5]. By using hydraulic jacks, the superstructure of the bridge can be lifted to remove the original bearings, replacing them with suitable LRB bearings. In practice, isolated bridges with LRB bearings have been proven to perform effectively reducing bridge seismic responses during earthquake shaking, like the Thjorsa River Bridge that survived two major earthquakes of magnitudes 6.6 and 6.5 (M_w) without serious damage and was open for traffic immediately after the earthquakes[6].

During the last decades horizontally curved viaducts have become an important component in modern highway systems as a viable option at complicated interchanges or river crossings where geometric restrictions and constraints of limited site space make extremely complicated the adoption of standard straight superstructures. Curved alignments offer, in addition, the benefits of aesthetically pleasing, traffic sight distance increase, as well as economically competitive construction costs with regard to straight bridges. On the contrary, bridges with curved configurations may sustain severe damage owing to rotation of the superstructure or displacement toward the outside of the curve line due to complex vibrations occurring during an earthquake[4]. For this reason, curved bridges have suffered severe damage in past earthquakes. The South Fork Eel River Bridge, a curved steel girder bridge located 49 km from the epicenter of the 1992 Petrolia earthquake sustained considerable damage at hinge locations with a large impact on its service capacity[7]. And the partial collapse during the 1994 Northridge earthquake of two curved bridges at the Interstate 5/ Route 14 interchange[8] is another example to corroborate the seismic vulnerability of curved bridge structures.

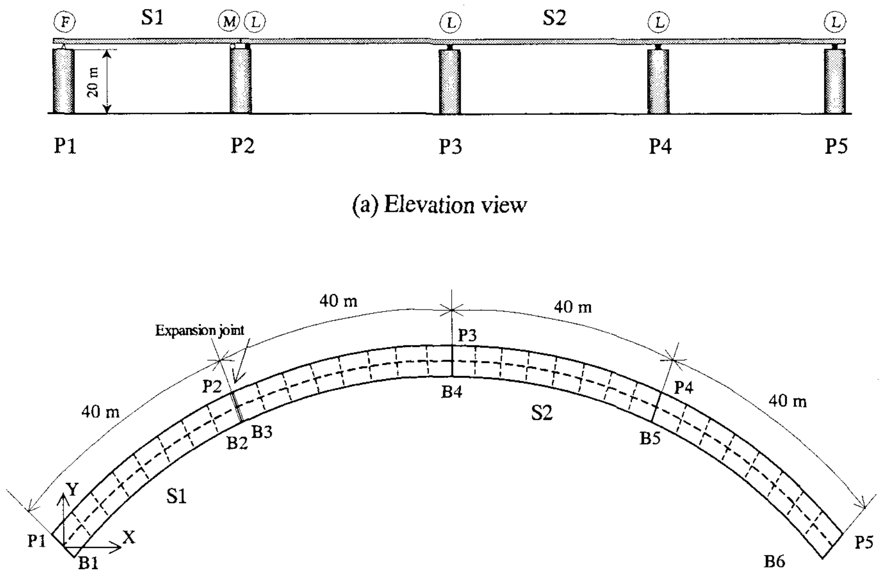
The considerable complexity associated with the analysis of curved viaducts, especially under earthquake loading, requires a realistic prediction of the structural response. For that reason, the mitigation of earthquake damage for such structures requires the comprehensive understanding of all important aspects of the complex problem involving both, a precise three-dimensional modeling and nonlinear dynamic analysis. Therefore, this paper presents a detailed analysis of the seismic response a substantially adverse case of highway viaduct configuration which concentrates various significant seismic hazards, including curved alignment, the presence of an expansion joint, and adjacent bridge sections with different sizes and bearing supports. The study combines the use of nonlinear dynamic analysis with a detailed three-dimensional nonlinear bridge model to evaluate the global dynamic behaviour of the bridge system with particular emphasis focused on the expansion joint. In order to perform a complete investigation of the role of cable restrainers on the performance of the viaduct, the original configuration is analyzed to provide a refined estimation of seismic demands on critical components. Then, the direct comparison of these responses with those for which the highway viaduct is equipped with seismic cable restrainers in two different configurations is evaluated.

2. Analytical Model of Highway Viaduct

The highway viaduct considered in the analysis is composed by a three-span continuous seismically isolated section connected to a single simply supported non-isolated span. The overall viaduct length of 160 m is divided in equal spans of 40 m, as represented in Fig. 1. The bridge alignment is horizontally curved in a circular arc with a radius of curvature of 100 m, measured from the origin of the circular arc to the centerline of the deck superstructure. Tangential configuration for both piers and bearing supports is adopted, respect to the global coordinate system for the bridge, shown in the figure, in which the X- and Y-axes lie in the horizontal plane while the Z-axis is vertical.

2.1 Deck Superstructure and Piers

The bridge superstructure consists of a concrete deck slab that rests on three I-shape steel girders, equally spaced at an interval of 2.1 m. The girders are interconnected by end-span diaphragms as well as intermediate diaphragms at uniform spacing of 5.0 m. Full composite action between the slab and the girders is assumed for the superstructure model, which is treated as a three-dimensional grillage beam system presented in Fig. 2. The deck weight is supported on five hollow box section steel piers of 20 m height designed according to the seismic code in Japan[4]. Cross sectional properties of deck and bridge piers are summarized in Table 1. Characterization of structural pier elements is based on the fiber element modelization in which the inelasticity of the flexure element is accounted by the division of the cross-section into a discrete number of longitudinal and transversal fiber regions with constitutive model based on uniaxial stress-strain relationship for each zone. The element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. At the pier locations the bridge deck is modelled in the transverse direction as a rigid bar of length equal to the deck width. This transverse rigid bar is used to model the interactions between deck and pier motions[9].



(a) Elevation view

(b) Plan view

Figure 1 Model of curved highway viaduct

Table 1 Cross sectional properties of deck and piers

	A (m^2)	I_x (m^4)	I_y (m^4) [(1)]
P1	0.4500	0.3798	0.3798
P2	0.4700	0.4329	0.4329
P3	0.4700	0.4329	0.4329
P4	0.4700	0.4329	0.4329
P5	0.4500	0.3798	0.3798
G1	0.2100	0.1005	0.0994
G2	0.4200	0.1609	0.2182
G3	0.2100	0.1005	0.0994

[(1)] I_z in case of G1, G2 and G3

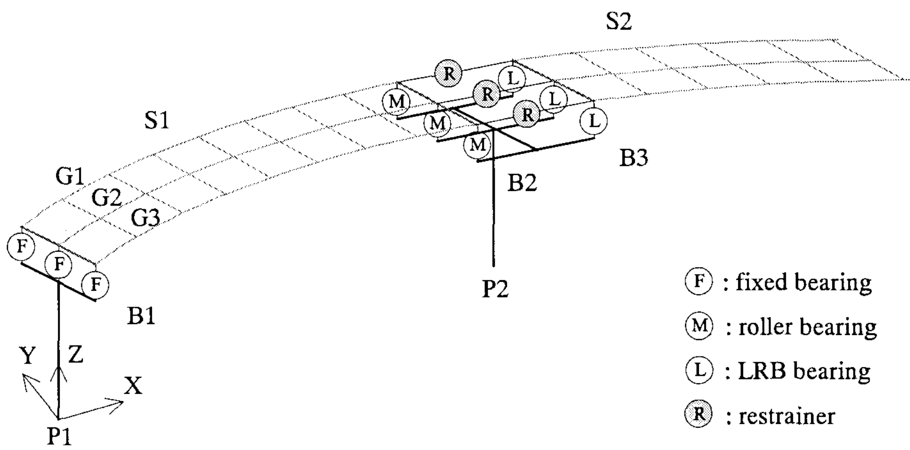


Fig. 2 Highway viaduct finite element model

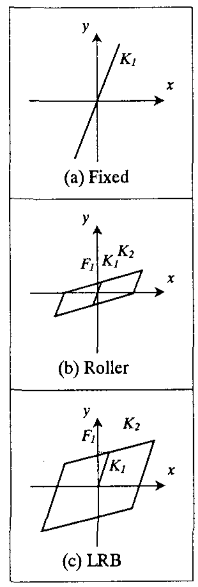


Fig. 3 Bearings analytical models

2.2 Bearing Supports

Steel fixed bearing supports (Fig. 3-a) are installed across the full width on the left end of the simply-supported span (S1), resting on the Pier 1 (P1). Steel roller bearings at the right end on the Pier 2 (P2) allow for movement in the longitudinal (tangent to the curved superstructure) direction while restrained in the transverse radial direction. Coulomb friction force is taken into account in numerical analysis for roller bearings, which are modelled by using the bilinear rectangle displacement-load relationship, shown in Fig. 3-b.

The isolated continuous section (S2) is supported on four pier units (P2, P3, P4 and P5) by LRB bearings. The left end is resting on the same P2 that supports S1, and at the right end on the top of P5. Orientation of LRB bearings is such as to allow for longitudinal and transverse movements.

LRB bearing supports are represented by the bilinear force-displacement hysteresis loop given in Fig. 3-c. The principal parameters that characterize the model are the pre-yield stiffness K_1, corresponding to combined stiffnesses of the rubber bearing and the lead core, the stiffness of the rubber K_2 and the yield force of the lead core F_1. The devices are designed for optimum yield force level to superstructure weight ratio ($F_1/W = 0.1$) and pre-yield to post-yield stiffness ratio ($K_1/K_2 = 15.0$), which provide maximum seismic energy dissipation capacity as well as limited maximum deck displacements[10, 11].

2.3 Expansion Joint

The isolated and non-isolated sections of the viaduct are separated, introducing a gap equal to the width of the expansion joint opening between adjacent spans, to allow for contraction and expansion of the road deck from creep, shrinkage, temperature fluctuations and traffic without generating constraint forces in the structure. In the event of strong earthquakes, the expansion joint gap of 0.1 m could close resulting in collision between the deck superstructures. Pounding phenomenon, defined as taking place at the three girder ends, is modelled using impact spring elements for which the compression-only bilinear gap element is provided with a spring of stiffness K_i = 980.0 MN/m that acts when the gap between the girders is completely closed.

On the other hand, in order to prevent excessive opening of the expansion joint gap thus providing additional fail-safe protection against extreme seismic loads, longitudinal and transverse cable restrainers are anchored to the girder ends connecting both adjacent superstructures across the expansion joint. The seismic restrainers, illustrated in Fig. 4, have been modelled as tension-only spring elements provided with a slack of 0.025 m, a value fitted to accommodate the expected deck thermal movements limiting the activation of the system specifically for earthquake loading. The seismic performance of the viaduct has been evaluated using four different stiffness values of restrainers (K_r = 9.8, 49.0, 98.0, and 490.0 MN/m) based on cross-sectional characteristics and the modulus of elasticity of the cables. In order to simplify, the effects of the expansion joint in the transverse direction as well as the shear forces acting on cable restrainers are neglected.

3. Nonlinear Analytical Method

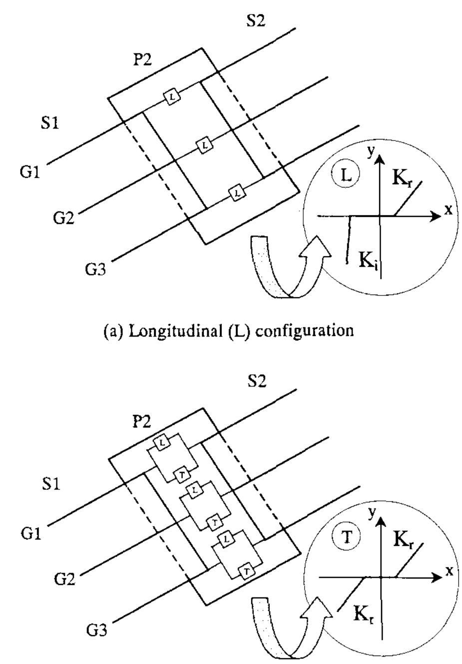
The analysis on the highway bridge model is conducted using an analytical method based on the elasto-plastic finite displacement dynamic response analysis. The governing nonlinear equation of motion can be derived by the principle of energy that the external work is absorbed by the work of internal, inertial and damping forces for any small admissible motion that satisfies compatibility and essential boundary conditions[12]. Hence, the incremental finite element dynamic equilibrium equation at time $t+\Delta t$ over all the elements, can be expressed in the following matrix form:

$$[\mathbf{M}]\{\ddot{u}\}^{t+\Delta t} + [\mathbf{C}]\{\dot{u}\}^{t+\Delta t} + [\mathbf{K}]^{t+\Delta t}\{\Delta u\}^{t+\Delta t} = -[\mathbf{M}]\{\ddot{z}\}^{t+\Delta t} \qquad (1)$$

where [M], [C] and $[K]^{t+\Delta t}$ represent respectively the mass, damping and tangent stiffness matrices of the bridge structure at time $t+\Delta t$. While $\ddot{u}$, $\dot{u}$, Δu and $\ddot{z}$ denote the structural accelerations, velocities, incremental displacements and earthquake accelerations at time $t+\Delta t$, respectively.

The incremental equation of motion accounts for both geometrical and material nonlinearities. Material nonlinearity is introduced through the bilinear elastic-plastic stress-strain relationship of the beam-column element, incorporating a uniaxial yield criterion and kinematic strain-hardening rule. The yield stress is 235.4 MPa, the elastic modulus is 200 GPa and the strain hardening in plastic area is 0.01.

The Newmark's step-by-step method of constant acceleration is formulated for the integration of equation of motion. The Newmark's integration parameters (β =1/4, γ = 1/2) are selected to give the required integration stability and optimal result accuracy. The equation of motion is solved for the incremental displacement using the Newton-Raphson iteration scheme where the stiffness matrix is updated at each increment to consider geometrical and material nonlinearities and to speed to convergence rate. The damping mechanism is introduced in the analysis through the Rayleigh damping matrix, expressed as a linear combination of the mass matrix and the stiffness matrix. The particular values of damping coefficients are set to ensure a relative damping value of 2% in the first two natural modes of the structure.



(a) Longitudinal (L) configuration

(b) Longitudinal (L) and transverse (T) configuration

Fig. 4 Cable restrainer configurations and analytical models

4. Input Earthquake Ground Motions

To assess the seismic performance of the viaduct, the nonlinear bridge model is subjected to the longitudinal (L), transverse (T) and vertical (V) components of four different sets of strong ground motion records given in Fig. 5. The longitudinal earthquake component shakes the highway viaduct parallel to the X-axis of the global coordinate system, while the transverse and vertical components are acting in the Y- and Z-axes, respectively.

The large magnitude events used in this study, classified as near-fault earthquakes, are characterized by the presence of high peak accelerations and strong velocity pulses with a long period component as well as large ground displacements[13, 14]. These exceptionally strong earthquakes have been selected due to the destructive potential of long duration pulses on flexible structures equipped with isolation systems that can lead to a large isolator displacement, probably exciting the bridge into its nonlinear range as well as inducing opening and pounding phenomenon at the expansion joint.

5. Numerical Results

5.1 Natural Periods and Mode Shapes

Valuable information regarding to the highway viaduct structural behaviour during seismic events is provided by calculation of its dynamic characteristics. For this reason, natural vibration analysis has been preliminary performed in order to obtain the natural periods and mode shapes as well as to determine the parameters required for defining Rayleigh damping in the subsequent nonlinear time-history analysis.

For the natural vibration analysis, the equation of motion for the highway viaduct can be written in form of frequency equation as follow,

$$\|[\mathbf{K}] - \omega^2[\mathbf{M}]\| = 0 \tag{2}$$

The solution of this equation yields the circular frequencies ω_i and hence the dominant periods T_i can be calculated.

According to recommendations of the Specifications for Highway Bridges in Japan[4], characteristics of LRB isolation bearings, based on the effective stiffness at the maximum design displacement, have been selected to obtain periods slightly larger than twice the natural period of the bridge when all bearings are assumed to be fixed (T_1 = 0.63 seconds). As a result, the viaduct shifts its fundamental natural period with the objective of reflecting a major portion of the earthquake energy. On the other hand, the obtained moderate period shift is expected to limit the increased displacements experienced by the bridge deck during strong earthquake ground motions.

The fundamental mode of the bridge structure consists predominantly of rigid body translation of the isolated deck along the in-plane direction, while the non-isolated section essentially rotates and translates in out-plane direction. The bridge fundamental natural period is T_1 = 1.31 seconds. Therefore, the viaduct may be severely damaged by the action of near-fault earthquake excitations considered in this study, which concentrates most of their seismic energy near this range of long periods.

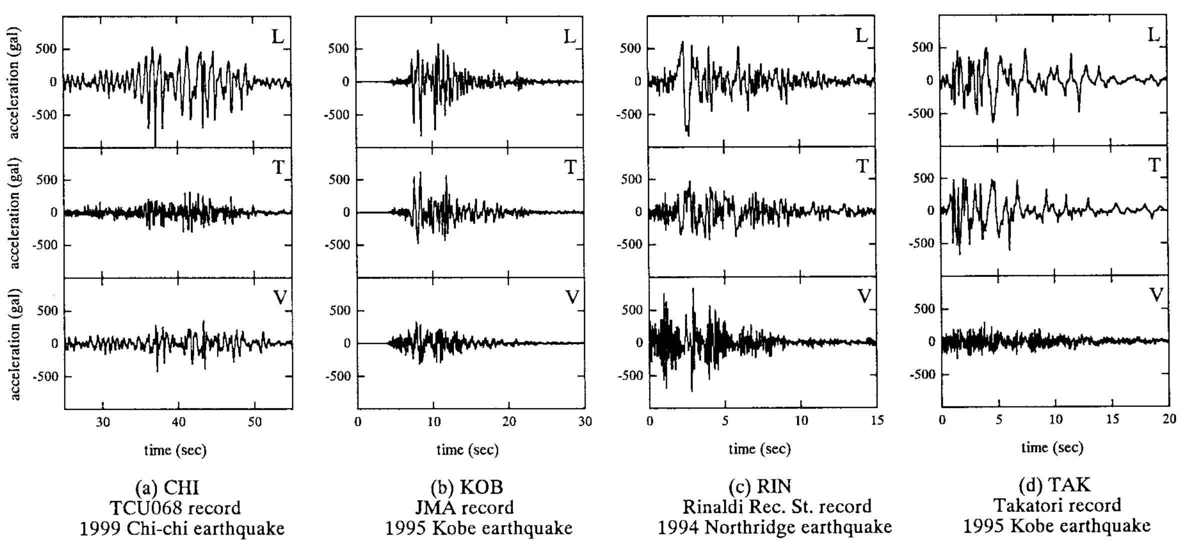


Fig. 5 Input earthquake ground motions

5.2 Nonlinear Dynamic Response

The overall three-dimensional response of the viaduct is investigated in detail through nonlinear dynamic response analysis. Particular emphasis has been focused on the expansion joint behaviour due to the extreme complexity associated with connection between isolated and non-isolated sections in curved viaducts. In order to improve its seismic performance, the viaduct is implemented with unseating prevention cable restrainers for different position of installation (Case II, Case III). The effectiveness of both different solutions has been investigated by comparing their seismic response with the corresponding response obtained for the original configuration (Case I):

Case I: original configuration without cable restrainers

Case II: installation of longitudinal cable restrainers

Case III: installation of longitudinal and transverse cable restrainers

(1) Case I

The seismic response evaluation of the original bridge configuration clearly indicates that the ability of the bridge to withstand near-fault seismic loads is not satisfactory. The weakest link is found at the expansion joint that connects the isolated and non-isolated sections of the viaduct. Time-history response of the relative displacement between adjacent decks, presented in Fig. 6, demonstrates that maximum opening of the expansion joint can be as large as 0.36 m for CHI input earthquake. Such extremely peak separation between adjacent superstructures, caused by the large accelerations developed at predominant periods similar to the fundamental period of the bridge, induces relative negative displacements up to 0.20 m to the steel roller bearings that would probably result in unseating of the non-isolated deck superstructure from the top of P2. Moreover, small residual displacements at the expansion joint can be observed at the conclusion of the earthquakes. The separation or compression damage to the joint is due to the final position of the roller bearings as well as the relative inclination between P1 and P2, which is caused by plastic damage at the bottom of the piers. However, the slight permanent longitudinal offset observed for the four earthquake loadings is not excessively large, and it is not expected to interfere the post-earthquake highway viaduct serviceability. On the other hand, the maximum out-plane opening at the expansion joint, shown in Fig. 7, is 0.13, 0.10, 0.09, and 0.14 m for CHI, KOB, RIN and TAK input waves, respectively. These values are significantly smaller than those in the longitudinal direction since steel roller bearings are transversally fixed.

It is also important to note that pounding between girders of adjacent superstructures takes place at the expansion joint during the earthquake excitations. For all ground motions the trend of collisions is quite similar with impact force values considerably varying depending on their position across the expansion joint, as plotted in Fig. 8 for RIN earthquake. The exterior girder G3 first starts the collision, as it is illustrated in Fig. 9, and consequently absorbs most of the impact energy.

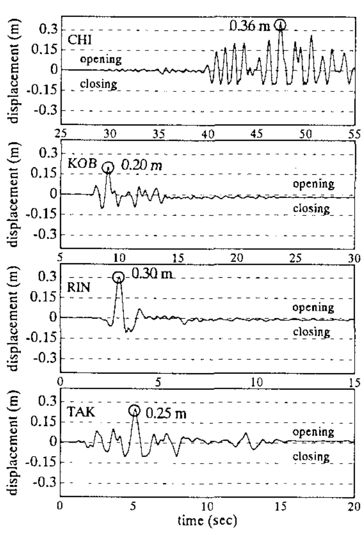


Fig. 6 In-plane displacement time-history of expansion joint (Case I)

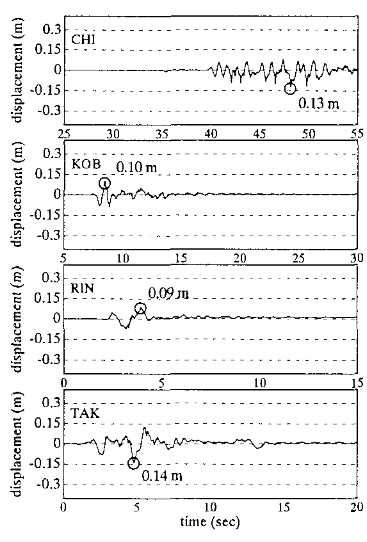


Fig. 7 Out-plane displacement time-history of expansion joint (Case I)

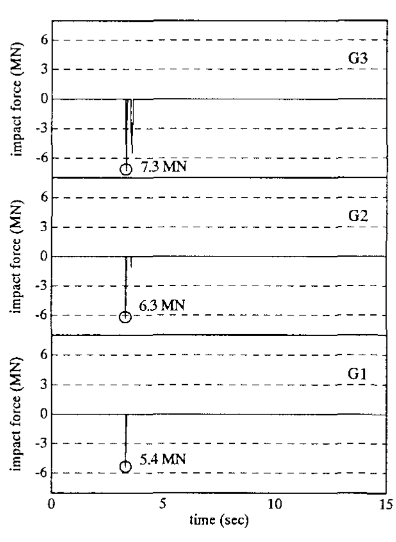


Fig. 8 Pounding time-history at expansion joint (Case I, RIN)

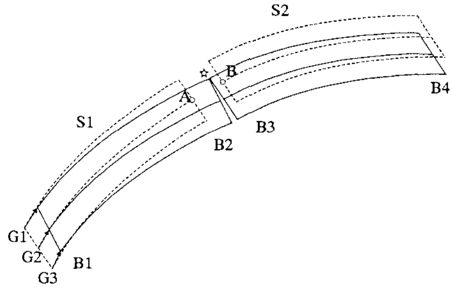


Fig. 9 Illustration of impact behaviour of highway viaduct (Case I)

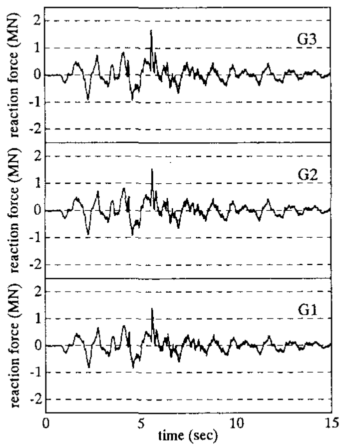


Fig. 10 Out-plane force time-history of roller bearings (Case I, TAK)

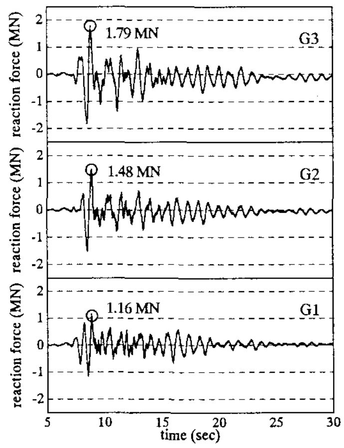
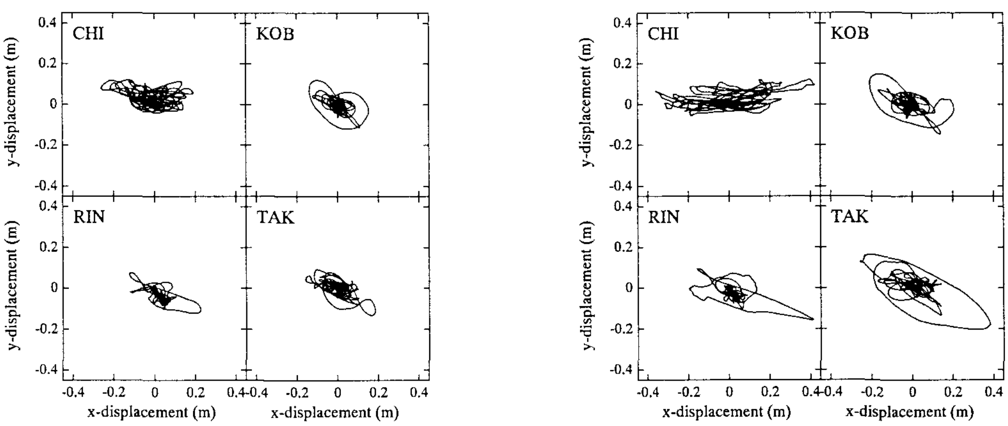


Fig. 11 In-plane force time-history of fixed bearings (Case I, KOB)



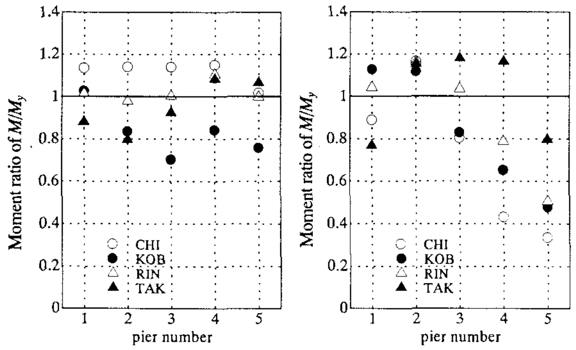
(a) Point A (b) Point B

Fig. 12 Displacement trajectories of deck superstructure (Case I)

Due to pounding phenomenon, a remarkable point to note is that, in addition to the expected local damage at colliding girders, high impact forces are transmitted to the bearing supports of P2. The large spikes in the in-plane component of reaction force observed in Fig. 10 make the steel bearing supports especially vulnerable to failure, which could result into the collapse of the bridge.

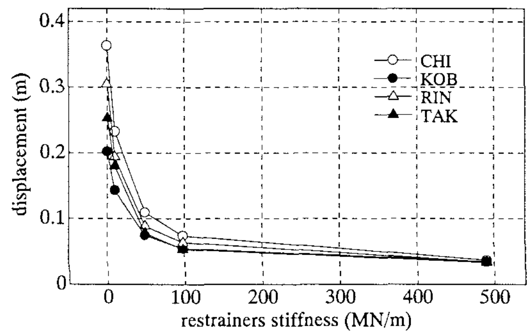
The advantage of using a precise three-dimensional model allows for evaluation of the individual bearing seismic response. Calculated results, shown in Fig. 11, reveal the relevant fact that the distribution of reaction forces for the three fixed bearing supports of P1 is considerably unbalanced. The obviously different appearance of the responses is caused by the viaduct natural tendency to rotate, due to the curved geometry, resulting in significant irregular distribution of bearing reaction forces. Rotation with respect to the vertical axis is also clearly appreciated by observing the trajectories of both adjacent decks at the expansion joint in the horizontal x- and y-directions, shown in Fig. 12. The bearing located at the exterior girder G3 attracts the largest seismically induced force. It is noticeable that the percentage of reaction force carried by this bearing can reach up to 60% of the total bearing line force. This feature is particularly important since the concentration of seismic forces to specific steel bearings may result in failure due to breakage of the anchor bolts that attach the bearing to the deck, leading to the failure of the complete bearing line, and greatly increasing the possibility of deck unseating.

It is well known that bending moments transmitted to the bottom of bridge piers during a seismic event can accurately estimate their induced structural damage. For this reason, the ratios of absolute maximum to the yield bending moment (M/M_y) at the bottom of the five piers have been calculated in both, in-plane and out-plane, directions. Plotted in Fig. 13, the

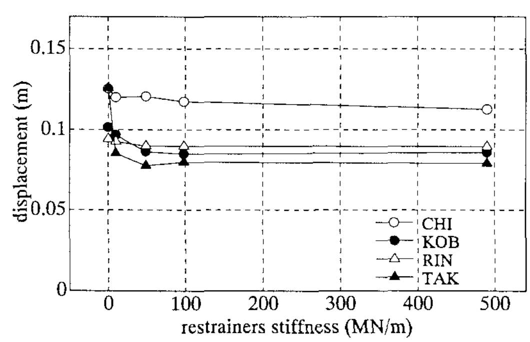


(a) In-plane (b) Out-plane

Fig. 13 Peak bending moment at base of bridge piers (Case I)



(a) In-plane



(b) Out-plane

Fig. 14 Peak relative displacement at expansion joint (Case II)

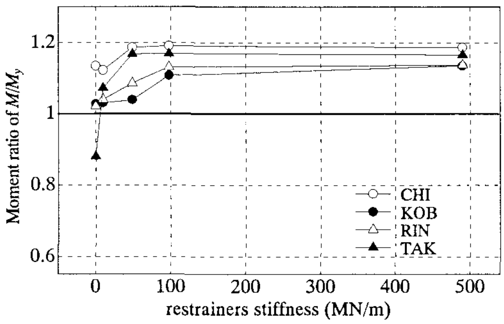
results indicate that piers with steel bearing supports (P1 and P2) exceed their elastic limit for most of the earthquake loadings considered in this study. On the other hand, essentially elastic response of piers equipped with LRB bearings (P3, P4 and P5) is expected. However, it should be indicated that isolated piers also suffer damage because the large pulses from near-fault ground motions are able to impart a great amount of seismic energy to flexible isolated structures, causing large displacements and forces at the LRB isolation bearing supports, thus inducing significant seismic damage to the piers. It is noticeable that maximum out-plane bending moment of P2 is generally the largest, since it receives large reaction forces of its bearings, which need to accommodate important out-plane deformations from both superstructures at the expansion joint. The maximum M/M_y ratio of this pier is about 1.15, a value in the range where moderate damage would be expected.

(2) Case II

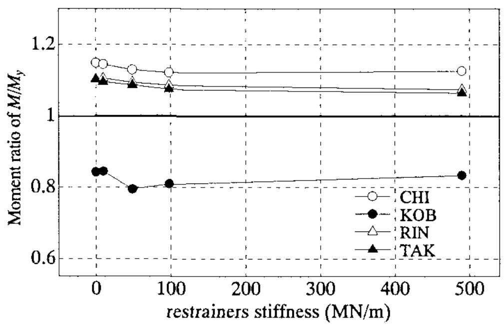
On the basis of previous calculated results, the capacity of cable restrainers, connecting longitudinally the three girders of adjacent viaduct sections, to improve the seismic response of the original bridge configuration is evaluated.

As it can be seen in Fig. 14-a, installation of seismic restrainers appears to be very effective in achieving significant reductions in the peak relative longitudinal displacement between adjacent spans. The gap opening reduction is more sensitive to initial changes in restrainer stiffness than to later ones. Therefore, a relatively moderate level of stiffness (K_r = 49.0 MN/m) is required to assure acceptable peak openings for CHI, KOB, RIN and TAK ground motions of 0.110, 0.075, 0.089, and 0.078 m, respectively, which approximately correspond to 70% reduction compared to maximum relative displacements previously calculated for Case I (K_r = 0.0 MN/m). Accordingly, the possibility of deck unseating of S1 at P2 is minimized, reducing the maximum relative displacement of roller bearings to values that do not exceed the typical seating length of bridge piers. Furthermore, the high level of force associated with stiff restrainers may result in failure of the unseating prevention device. Moreover, it is noteworthy in Fig. 14-b that longitudinal restrainers are able to achieve slight reductions in the maximum out-plane opening as a result of the coupled response of motions caused by the curved geometry of the highway viaduct.

A compromise using restrainers is that the overall bridge structure may behave in a different manner due to the transference process of seismic forces between adjacent superstructures when the unseating prevention system is activated. The detailed global response examination of the viaduct has revealed that the presence of restrainers substantially alters the seismic response of bridge piers. As shown in Fig. 15, a direct comparison with Case I indicates that, as the restrainer stiffness increases, in-plane maximum bending moments are observed to simultaneously increase for piers supporting the non-isolated deck, decreasing in case of piers of the isolated bridge section.

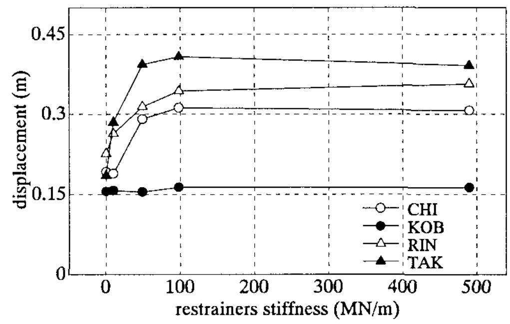


(a) Pier P1

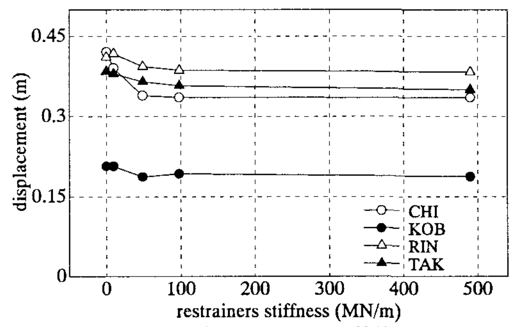


(b) Pier P4

Fig. 15 Peak in-plane bending moment at pier base (Case II)



(a) Non-isolated deck (S1)



(b) Isolated deck (S2)

Fig. 16 Peak in-plane displacement of deck (Case II)

The increased pier demands are due primarily to the uniform of displacements taken place when decks are connected by restrainers. Displacement of the non-isolated superstructure is similar to the deformation of the fixed steel bearings. This implies that the increased displacements imposed on the non-isolated deck by the restrainers (Fig. 16-a) could have put unacceptably high in-plane reaction loads to the fixed bearings, causing increased damage to P1. On the other hand, the plot shown in Fig. 16-b indicates a marginal decrease in maximum displacement in the isolated deck with increased stiffness of restrainers. It should be noted that similar trend of reduction occurs for out-plane damage of isolated piers due to the effect of curved configuration of the viaduct.

Fig. 17 shows comparison of maximum total impact forces between adjacent spans for different stiffness of restrainers. It can be seen that restrainers are not completely effective to mitigate pounding at the expansion joint. Pounding response follows an unpredictable behaviour, increasing in case of KOB and TAK ground motions and decreasing for CHI and RIN earthquakes, as the stiffness of restrainers is increased. A detailed examination of pounding phenomenon has revealed that for all earthquakes, decks collide each other when both superstructures are moving in the same direction. This type of impact is relatively benign to the bridge because extremely large forces would be expected

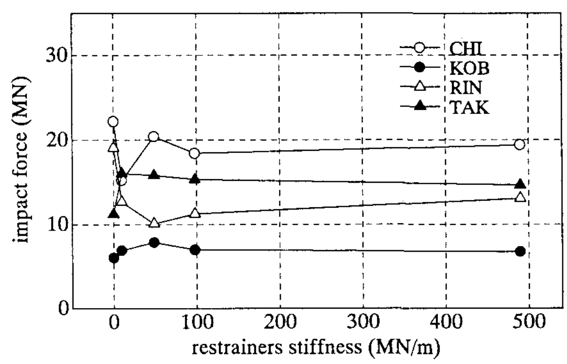


Fig. 17 Peak total impact force at expansion joint (Case II)

in case of pounding with decks moving in opposite directions. As appreciated in Fig. 18, it is found that peak total impact forces are related to the relative velocities between the adjacent spans at the time of the main impact. Longitudinal restrainers tend to uniform the relative displacements between adjacent spans, however this type of unseating prevention system is not able to uniform velocities at the moment of the impact in such way that maximum pounding forces can be reduced. Activation of restrainers during the earthquake ground motions arbitrary modifies velocities of adjacent decks. Therefore, impact forces may have both possibilities of increasing or reducing by restrainers depending on variation of relative velocity between decks at the moment of the impact.

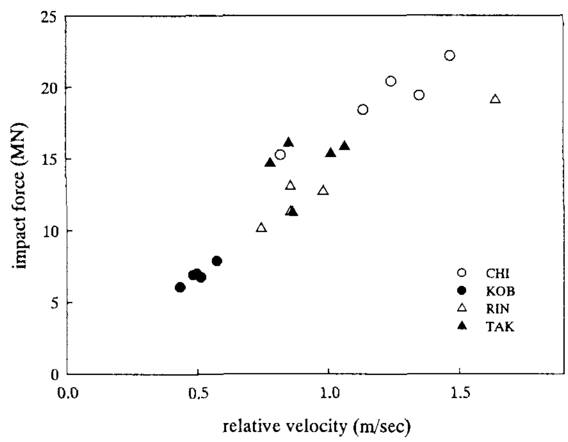


Fig. 18 Peak total impact force-relative deck velocity relationship (Cases I and II)

It is finally indicated that restrainers do not have a significant effect on equilibrating the irregular pounding at the expansion joint as well as the unbalanced distribution of forces on fixed bearings of P1 caused by the curved viaduct alignment.

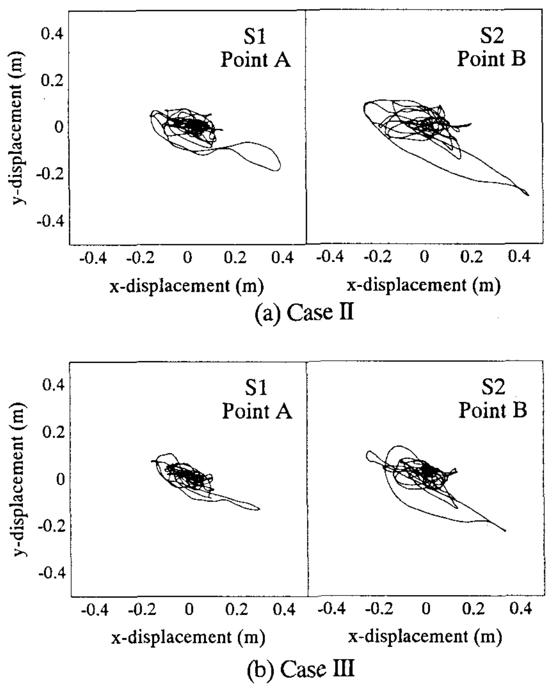


Fig. 19 Displacement trajectories of deck (TAK)

(3) Case III

In order to secure the expansion joint for all movements in the horizontal plane, a pair of cable restrainers is set in both longitudinal and transverse configurations at each girder end connection. In the present study, only tension behaviour is considered for longitudinal restrainers, while transverse restrainers, provided with adequate lateral bracing, can act in tension and compression, resisting positive and negative relative displacements.

The performance of the proposed configuration has been investigated by comparing the calculated results with those obtained in Case II for K_r = 49.0 MN/m. This value of longitudinal restrainers stiffness is selected since restrainers with moderate stiffness level displays effective gap opening reduction and adequate level of restrainers force. It should be indicated that the same restrainers stiffness (K_r = 49.0 MN/m) is selected for restrainers in transverse configuration since small variations of seismic response have been observed for lower or higher levels of stiffness and restrainers do not yield during the earthquakes.

It is appeared that installation of transverse restrainers in combination with those in longitudinal configuration (Case III) has a significant effect on the viaduct seismic response. Examination of the x- and y-trajectories of adjacent spans reveals the especial nature of pulse-type ground accelerations, typical of near-fault earthquakes, which induce severe peak displacements coincident with the dominant pulse at the beginning of the earthquake. As it can be observed in Fig. 19, transverse restrainers are very effective in reducing not only the out-plane but also the in-plane displacements of both superstructures compared to Case II.

The seismic response quantities of this study are found predominantly influenced by maximum displacements of isolated and non-isolated deck superstructures. Consequently, the substantial reduction in deck displacements results in considerably diminution of seismic demands on substructure elements, bearing supports and piers. Therefore, ductility demands of all pier units, plotted in Fig. 20, can be positively limited and minimized in both, in-plane and out-plane, bridge directions.

Moreover, Fig. 21 indicates that Case III is also observed to work more effectively than Case II in decreasing the relative velocities between adjacent decks at the instant of impact. As impact forces show similar trend of variation, this implies a pronounced reduction in the impact forces at the expansion joint. The improvement of several critical demands of bridge components may be attributed to the combined action of the seismic restrainers that allow controlling the coupled behaviour derived from the curved alignment of the highway viaduct.

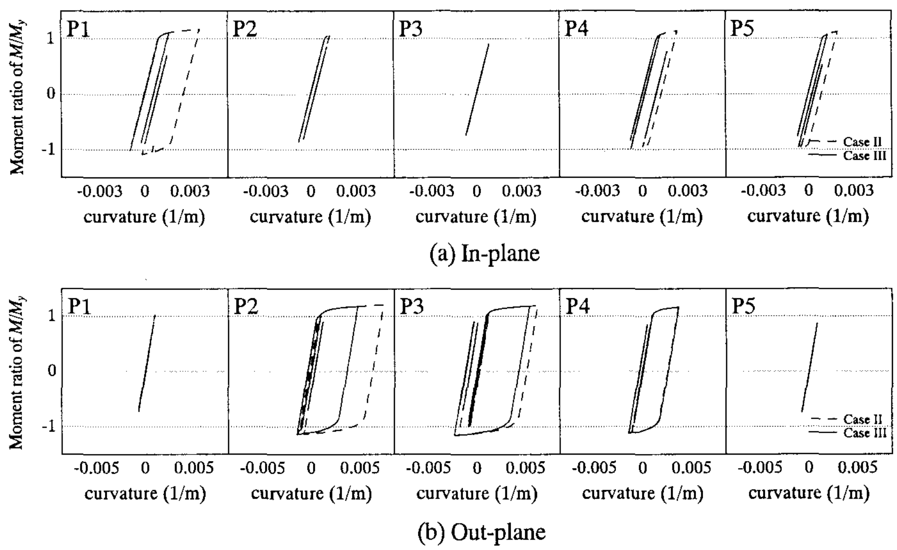


Fig. 20 Bending moment-curvature relationship at base of piers (TAK)

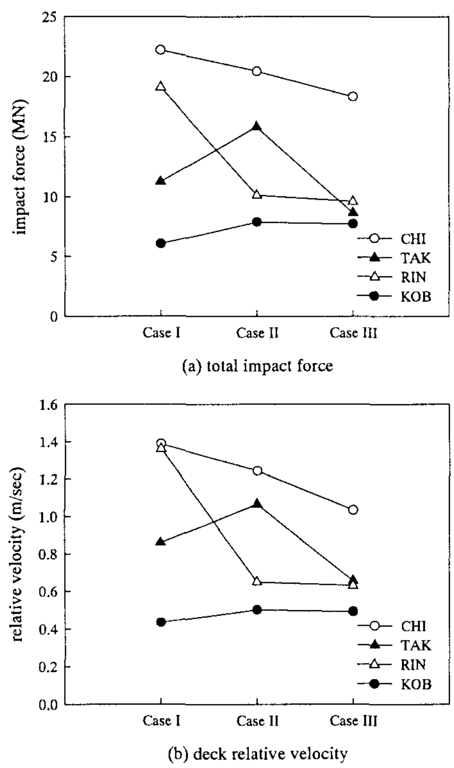


Fig. 21 Effect of restrainers on total pounding force and deck relative velocity

6. Conclusions

The effectiveness of seismic cable restrainers, installed in two different configurations to mitigate earthquake damage of connection between isolated and non-isolated sections of curved highway viaducts is evaluated in this study through three-dimensional nonlinear finite element response analysis. For this purpose, every important bridge element as well as the global structural response has been examined in detail under the action of four near-fault ground motions. The investigation results provide sufficient evidence for the following conclusions:

(1) The use of longitudinal restrainers is very effective in preventing the expected collapse due to deck unseating of the original viaduct configuration by excessive gap opening at the expansion joint. However, it is indicated that excessively stiff restrainers substantially increase ductility demands on piers of the non-isolated section of the viaduct. A moderate value of restrainer stiffness is required to limit joint movements to acceptable levels, avoiding in addition the high forces associated with stiff restrainers that may result in failure of the unseating prevention device.

(2) As a result of the coupled response of motions in the horizontal plane caused by the curved geometry of the viaduct, longitudinal restrainers are able to slightly reduce the peak out-plane opening of the expansion joint. However, in order to obtain significant reductions it is necessary the adoption of additional transverse seismic restrainers, which limit the joint movements to levels that allow for protection the expansion joint even under the action of strong near-fault earthquake ground motions.

(3) It is found that longitudinal restrainers are not completely effective in reducing damage to colliding girders and bearing supports due to pounding at the expansion joint.

However, the combined action of longitudinal and transverse restrainers achieves significant reductions in the relative velocity between adjacent spans at the time of the impact, thus decreasing the total impact forces.

(4) Near-fault earthquake ground motions induce extremely large deformations to LRB bearings that support the isolated section of the viaduct. This fact results in considerable seismic damage transmitted to the isolated piers. This damage can be only marginally reduced by the action of longitudinal restrainers, which tend to alleviate the seismic response of the isolated section. But it has been observed that the beneficial reduction of deck horizontal displacements with the adoption of longitudinal and transverse restrainers connecting the adjacent spans results in significantly smaller bending moments at the base of isolated piers, with the consequent protection of the piers against the effect of the seismic ground motion.

(5) The precise three-dimensional viaduct model proposed in this study allows for evaluation of the individual bearing seismic response. The calculated results reveal that the viaduct natural tendency to rotate with respect to the vertical axis induces an irregular distribution of reaction forces to fixed bearings. The considerable unbalanced distribution of forces is especially critical in the in-plane direction for fixed bearings. The concentration of seismic forces to the bearing located at the exterior girder makes it vulnerable to failure, which could result into the failure of the complete bearing line, and a great increase of the possibility of collapse of the bridge due to deck unseating. Restrainers are observed to be ineffective to reduce this phenomenon since it is intrinsically attributed to the geometric effect of the curved alignment of the viaduct.

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(Received: September 17, 2004)