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Study on Seismic Response of Curved Viaduct Systems with Different Isolation Conditions under Great Earthquake Ground Motion

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Curved highway viaducts may sustain severe damage during an earthquake due to their complex behavior. The use of isolation systems can substantially reduce their induced seismic forces. The main objective of this study is to evaluate the effectiveness of installation position, configuration and type of isolation. Two alternatives of isolation positions are examined, applying isolators underneath viaduct piers or as bearing support between deck and piers. For a typical curved viaduct, the isolators are configured either in radial or tangential arrangement. The validity of applying side restrainers to prevent the transverse movement of isolators is examined through the comparison between partial and full isolations. The nonlinear performance of a curved viaduct model seismically isolated by lead-rubber bearings over a wide range of isolation design parameters is evaluated and compared to the behavior of the originally nonisolated one under a three-dimensional great earthquake excitation. A finite element procedure is adopted to carry out the time history analysis. Nonlinearities due to material and geometry changes are considered. It is found that all isolation systems are effective in highly improving the in-plane performance of the nonisolated viaduct over a wide range of bearing stiffness while for the tangential configuration full isolation is essential to improve their out-of-plane response. The position of isolation displayed relatively better performance for systems of bottom isolators. Simple reciprocal moment and displacement relationship is presented to obtain the design parameters of isolators which yield the optimum performance of the isolation systems.

Key Words: curved viaduct, seismic isolation, bottom bearing, side restrainer, steel structure.

1. Introduction

The collapse and damage of numerous viaducts of elevated highways and railways during the earthquake of the Hyogoken-Nanbu 1995 was an unexpected shock, for the first time in world history, steel piers suffered damage due to earthquake. Severe local buckling, brittle crack failure and the falling of girders off their piers were typical damage to steel hollow section members. Bearings were the second element failed following major substructure damage, mainly, conventional roller and fixed steel supports were vulnerable and suffered severe damage [1,2]. Since then it has become a great challenge for earthquake engineers to adopt strategies that can ensure the safety of such arterial routes.

The effort on protecting a bridge against earthquake should be focused on minimizing the forces carried by piers. Piers should be designed to remain elastic for at least the design earthquake so that the structure can perform satisfactory during a greater magnitude earthquake [3,4]. To achieve such aim, seismic isolation is considered one of the most promising alternatives of hazard mitigation. The concept of base isolation is simple. It was recognized that it is usually the horizontal ground movement of an earthquake which causes damage to structures. So, if it is possible at one and the same time to uncouple the structure by providing a flexible interface between it and the ground, the lateral load will be minimized and the damage will be greatly reduced [5]. Among others, lead-rubber bearings (LRB) have been adopted extensively as the isolation bearings for seismically isolated structures. The base isolation bearings mainly change the stiffness of the structure, generally resulting in an increase in the natural period, increase the amount of damping and provide rigidity under service load levels [6,7].

Isolating bottom of columns from foundation is commonly adopted in base isolated buildings all over the world. This method offers the potential for significant damage reduction and also possible cost savings. Many base isolated buildings experienced great earthquake shaking during the last years. Investigating their dynamic response during earthquake proved that they behaved well and according to the expectations. Typical examples are the two isolated building in the northern area of Kobe- city during the Kobe earthquake [8] and the base isolated USC hospital building during the 1994 Northbridge earthquake [9]. The concept of bridge seismic isolation is different. Commonly, seismic isolation is adopted as bearing supports between the substructure and superstructure. Meanwhile, after its severe damage during the Hyogoken–Nanbu earthquake, the Benten viaduct was the first bridge in Japan to be reconstructed using isolators at the foot of piers. The main reason of adopting such innovative strategy was to rigidly tie the bridge pier and beams of superstructure for the longest possible length to avoid the falling down of the superstructure. For the transverse direction, side–brick type restrainers were installed on the isolators, 5 mm side away [10], so that the bridge is considered partially isolated (unidirectional isolation through the longitudinal axis of the bearings).

Partial isolation has been popular for bridges in several countries for many years to restrain the lateral movements and due to restriction of expansion joints. Such restraining may lead to high values of both overturning moment and shear force and furthermore requires very large pier size. Large pier size is not economic and will cause significant disturbance to violate the imposed environmental constraints [11]. Visual observation of the partially base isolated Onneto bridge in Japan, which utilizes steel side stoppers installed at both sides of all LRB bearings was carried out with the help of acceleration data recorded during four moderate to small earthquakes. The results revealed that the gap between the transverse side stoppers and the bottom flange of steel plate girders was clogged, which made the two to act as a single element. This bad behavior prevented the bridge to act as a typical base isolated one [12]. Such observations have increased the need to accurately reconsider the validity of applying partial isolation to highway viaducts.

Curved viaducts play inevitable role in any highway network. The behavior of such structures is complex and must be accurately determined [13]. Bridges with smaller curvature radius may sustain severe damage owing to rotation of the superstructure toward the outside of the curve [14]. The performance of curved viaduct depends on the configuration of the bearings. Tow different configurations of bearings and piers are adopted, radial and tangential configurations.

This paper presents an investigation of the peak performances of seismically isolated curved viaduct systems due to changing isolation conditions as position, configuration and type over a wide range of isolator design parameters. The installation of isolators as bearing supports at top of piers or underneath them, while the other side is rigidly connected, is studied. Both isolators and piers are arranged either in radial or tangential configurations. This is along with studying the effectiveness of isolators side restrainers through the response of partial and full isolation (through all horizontal directions) systems. The performances of these systems are compared with the behavior of the original case of nonisoalted viaduct with rigid connections. A simple method of selecting the design parameters of isolators as function of its stiffness ratio is presented depending on peak pier overturning moment and deck displacement response. This is followed by an integrated response analysis for the studied systems with the selected isolators stiffness ratio.

2. Description and Modeling of Viaduct

2.1 Deck and pier

The curved viaduct is four spans continuous hollow box section steel deck of total length equal 160 m (40 m each span) and radius of curvature equal 100 m. The height of all hollow box section steel piers are 20 m. Weight of the superstructure is about 11.8 MN. Radial and tangential arrangements for both piers and bearings are adopted in this study. General view, plan, elevation and cross sections of both deck and pier for the studied systems are shown in Fig. 1 thorough Fig. 4.

For the three dimensional dynamic analysis of a real structure, flexural fiber element is adopted to represent the behavior of both girder and piers. The cross section of these elements has been divided into twelve in–plane slice shaped fibers with an integration point at the centroid of each fiber element along the cross section. The integration point along the member length has been made at the middle of the member [15]. Material nonlinearity is modeled through bilinear stress–strain relationship with kinematic hardening law for the structural steel with a second inclination of $E/100$ to take into account the effect of strain hardening as shown in Fig. 5. The yield stress and elastic modulus are equal to 240 MPa and 200 GPa, respectively.

2.2 Isolation bearing

The studied systems are isolated using Lead– rubber bearings (LRB) that have found wide application in the field of highway bridges. Lead– rubber bearings are considered as elastomeric bearings with a lead core at its center, which extends over the full depth of the bearing. Under low (service) load the lead core provides elastic stiffness K_1 up to the shear yield force F_y. Beyond this point the lead core provides energy dissipation during the cycling of a seismic event. The rubber/steel laminated bearing surrounding the lead plug carries the weight of the structure and provides a restoring force for the device. Energy dissipation and damping are provided through material hysteresis. The shear force– displacement relationship for lead rubber bearings is idealized as bilinear hysteretic loops using spring elements in the three dimensions as shown in Fig. 6. K_1 is a property of the material of bearing; K_2 is proportional to the size of the bearing and inversely proportional to the rubber height. F_y is proportional to both bearing size and lead radius, F and d respectively, represent the shear force and displacement in the local axes of bearings.

To integrally evaluate the response of the studied systems, wide range of LRB design parameters, characteristic strength and stiffness, should be studied. This range is determined by initially studying the behavior of the nonisolated viaduct with both radial and tangential configurations assuming complete rigid connection between deck and piers as an original case. Then, lead rubber bearings are applied either between deck and piers or at bottom of piers. These bearings are idealized with linear behavior of stiffness K_0. Finally, the system is studied to get the value of K_0 which gives the same response as the

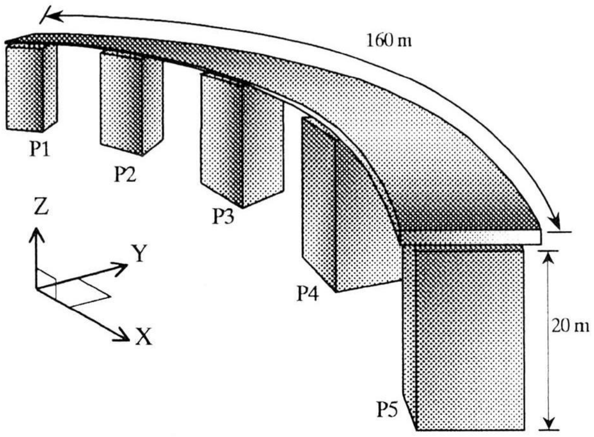

Fig. 1 General view of viaduct

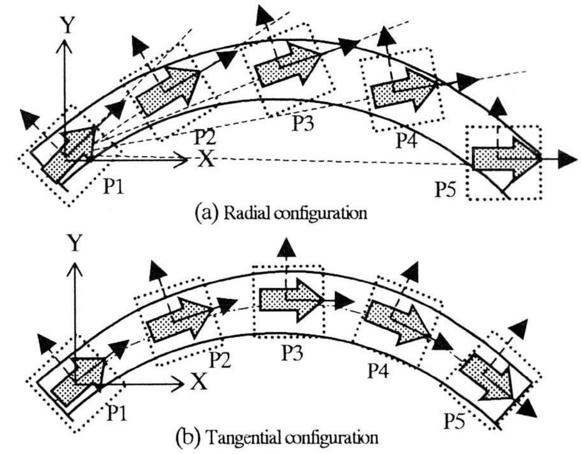

Fig. 2 Bearings and piers configuration

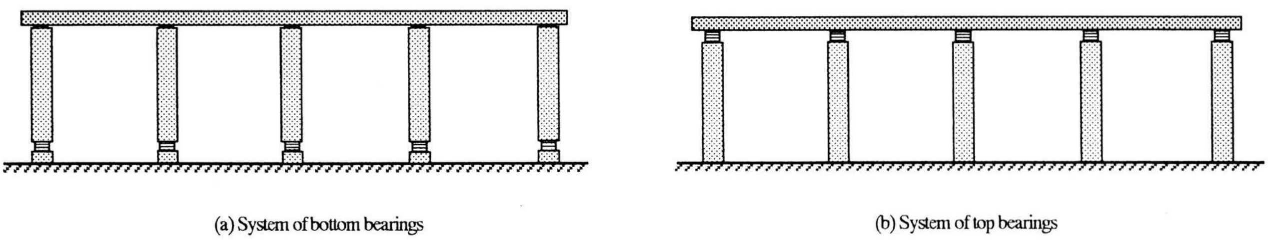

Fig. 3 Elevation of viaduct

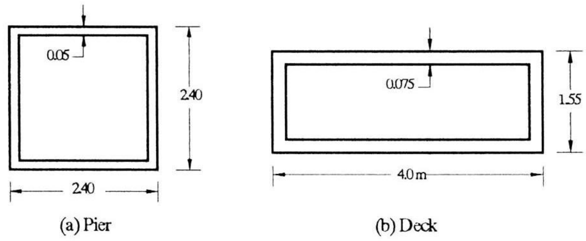

Fig. 4 Cross sections

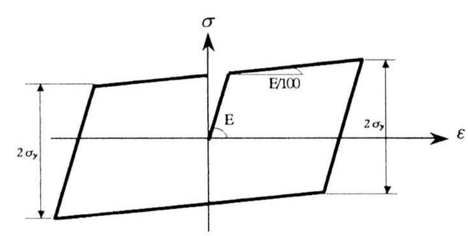

Fig. 5 Stress-strain relationship for steel

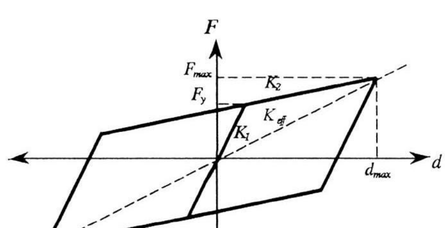

Fig. 6 Bilinear model of LRB

original case. The value of axial stiffness K_0 in the directions of the local axes of bearings is found to be equal to 35 GN/m, and that of the rotational stiffness around the same axes is equal to 35 TNm /rad. So, the range of LRB stiffness ratio is decided to be from stiffness ratio $K_1/K_0 = 5\text{x}\ 10^{-4}$ to stiffness ratio $K_1/K_0 = 1\text{x}\ 10^{-2}$. The strain hardening ratio is nominally equal to 0.15 [14]. F_y is chosen as a ratio of K_1, out of many studied values of this ratio, it is chosen to be equal to 0.02 to achieve balanced deformation and seismic forces behavior.

3. Nonlinear Equation of Motion

The motion of the viaduct system is described by the second-order differential equation

$$[M]\{\ddot{u}\}^{t+\Delta t}+[C]\{\dot{u}\}^{t+\Delta t}+[K]^{t+\Delta t}\{\Delta u\}^{t+\Delta t}$$

$$=\{F_{ext}\}^{t+\Delta t}-\{F_{int}\}^{t} \qquad (1)$$

where $[M]$ is the structural mass matrix, $[C]$ is the structural damping matrix and $[K]^{t+\Delta t}$ is the structural tangent stiffness matrix at time ($t+\Delta t$). Accelerations, velocities and incremental displacements are represented by $\ddot{u}$, $\dot{u}$ and Δu, $\{F_{ext}\}$ and $\{F_{int}\}$ represent the vectors of external and internal forces, respectively.

Numerically, this equation is solved directly in the time domain using the well–known Newmark –beta method which is considered as generalization of the linear acceleration method. Using Newmark integration method, the equation of motion is expressed as:

$$(a_1[M]+b_1[C]+[K]^{t+\Delta t})\{\Delta u\}^{t+\Delta t}=\{F_{ext}\}^{t+\Delta t}-\{F_{int}\}^t$$

$$-(a_2[M]+b_2[C])\{\dot{u}\}^t-(a_3[M]+b_3[C])\{\ddot{u}\}^t \qquad (2)$$

or

$$[K^{eff}]\{\Delta u\}^{t+\Delta t}=\{F^{eff}\} \qquad (3)$$

where

$$a_1=\frac{1}{\beta(\Delta t)^2}\quad a_2=\frac{1}{\beta(\Delta t)^2},\quad a_3=1-\frac{1}{2\beta},$$

$$b_1=\frac{\gamma}{\beta(\Delta t)},\ b_2=1-\frac{\gamma}{\beta},\ b_3=\Delta t(1-\frac{\gamma}{2\beta}) \qquad (4)$$

γ and β are the integration parameters of Newmark which are taken 0.5 and 0.25 respectively. This equation is solved to get the incremental displacement $\{\Delta u\}^{t+\Delta t}$ and hence the incremental velocities and accelerations can be obtained as follows:

$$\{\Delta\ddot{u}\}=a_1\{\Delta\bar{u}\}+a_2\{\Delta\dot{u}\}^t+(a_3-1)\{\ddot{u}\}^t$$

$$\{\Delta\dot{u}\}=\Delta t\{\ddot{u}\}^t+0.5\Delta t\{\Delta\ddot{u}\}^t \qquad (5)$$

($\Delta\bar{u}$) is the displacement measured from the previous convergence. The response quantities at time $t+\Delta t$ can now attained as[15]:

$$\{u\}^{t+\Delta t}=\{u\}^t+\{\Delta\bar{u}\}$$

$$\{\dot{u}\}^{t+\Delta t}=\{\dot{u}\}^t+\{\Delta\dot{u}\}$$

$$\{\ddot{u}\}^{t+\Delta t}=\{\ddot{u}\}^t+\{\Delta\ddot{u}\} \qquad (6)$$

Newton–Raphson iteration is used for the nonlinear analysis using equation (2). Tangent stiffness matrix $[K]^{t+\Delta t}$ is updated at each time step to speed up the convergence rate considering both geometric changes and material nonlinearities.

Damping of the structure is taken as Rayleigh damping which effectively capture the damping of the structure. The Rayleigh damping matrix $[C]$ is a combination of mass and stiffness matrices as follows:

$$[C]=A[M]+B[K] \qquad (7)$$

The coefficients A and B can be calculated as follows:

$$A=\frac{2\omega_i\omega_j(\xi_i\omega_j-\xi_j\omega_i)}{\omega_j^2-\omega_i^2}$$

$$B=\frac{2(\xi_i\omega_j-\xi_j\omega_i)}{\omega_j^2-\omega_i^2} \qquad (8)$$

The stiffness and mass proportional damping constants (A, B) are given in terms of the first two natural frequencies and corresponding damping ratios ξ_i and ξ_j. The values of damping ratios are taken 2%.

4. Ground Motion

The three components (E–W, N–S, and U–D) of the strong motion recorded at the JR West Japan Takatori Station during the Hyogoken–Nanbu 1995 earthquake, ground type II is used for the nonlinear dynamic response. The E–W component represents the ground motion in the longitudinal direction X of the viaduct while the N–S and U–D components represent the ground motions in the transverse Y and vertical Z directions of viaduct, respectively. The three components are shown in Fig. 7.

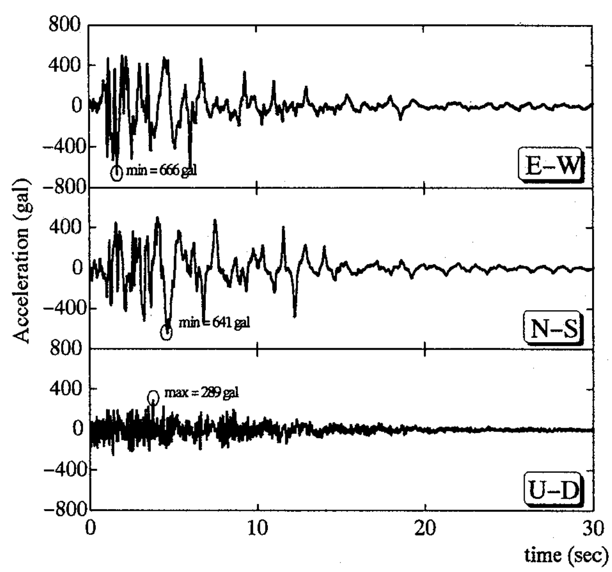

Fig. 7 Earthquake ground motions

5. Natural Vibration Analysis

Natural vibration analysis is carried out using consistent mass method to determine the values of the natural period for the studied cases. For a freely undamped system the equation of motion can be represented by the familiar characteristic equation:

$$[[K]-\omega^2[M]]\{a\}=\{0\} \qquad (9)$$

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

For an isolated viaduct the natural period should be large enough to shift the fundamental period of the structure to a range outside the energy containing periods of earthquake ground motion and hence avoid resonance with the excitation. Also it should be short enough to resist the ambient vibration of wind or traffic.

The natural periods of the first two modes of vibration for the nonisolated viaduct, radial configuration, are determined to be 0.58 and 0.55 seconds, respectively. Using isolation systems, remarkable shift in the period is observed depending on the system of isolation and the stiffness of bearings. The ratios of the natural periods of the first two modes of vibrations for the isolated systems (T) to those of the nonisolated one (T_0), radial configurations, are plotted in Fig. 8. The notations BP and BF represent the partially and fully isolated systems with bottom bearings, while the notations TP and TF represent the partially and fully isolated systems with top bearings respectively. It can be clearly observed that the ratios of the natural period are inversely proportional to the stiffness of bearing and as the stiffness decreases the difference of these ratios between the different systems increases except for partially isolated systems which display almost constant values of second mode close to that of the nonisolated system. This behavior can be attributed to that the first mode of vibration is related to the motion in the longitudinal direction while the second mode is related to the transverse motion. Applying full isolation highly increased the values of period for the second mode of vibration as the bearing stiffness decreases. The position of bearing results in relatively better response for systems with bottom bearing than those of top bearing while the configuration of bearing almost does not affect the response of natural vibration.

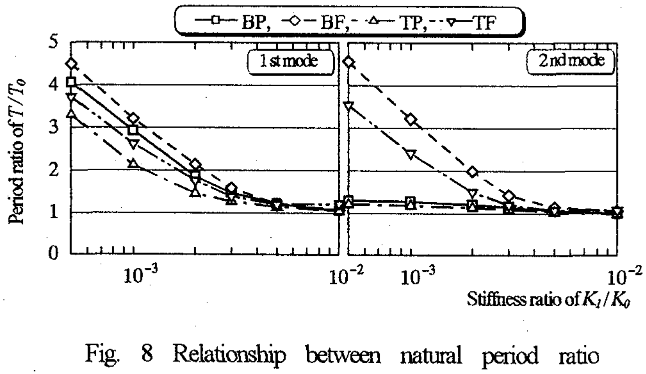

Fig. 8 Relationship between natural period ratio (T/T_0) and stiffness ratio (K_I/K_0)

6. Response to Earthquake Excitation

The performances of the studied isolated and nonisolated systems under the seismic excitation are investigated. The performance is presented through the behavior of deck, bearings and piers. The behavior of deck displacements and accelerations is studied. To show the effectiveness of isolation, peak response of overturning moment and curvature relationships calculated at all piers along with bearing displacement is examined. The following designations are used.

F: Nonisolated viaduct.

R, T: Radial and tangential configurations of bearings, respectively.

6.1 Deck displacement

The peak responses for in- plane and out- of- plane deck displacement of the studied systems are shown in Figs. 9 and 10. It can be observed that beyond stiffness ratio equal 2 x10 $^{-3}$ the isolation systems reveal small deck displacement close to the nonisolated one while systems isolated with smaller isolator stiffness behave differently. Partially isolated systems with radial arrangement display in- plane and out- of- plane response less than that of tangential one. The side restrainers are effective in controlling the out-of-plane displacement demonstrated by fully isolated systems due to the increase of flexibility, regardless of the position and arrangement of isolators, they also reduced the in-plane displacement of radial arrangement systems. The installation position of isolators slightly affects the displacement response yielding relatively higher displacement for bottom-isolated systems in some cases.

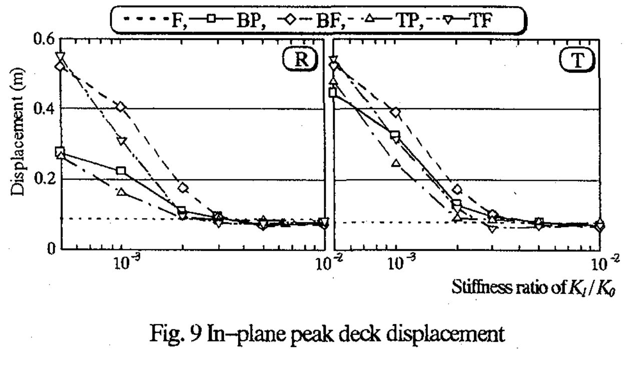

Fig. 9 In-plane peak deck displacement

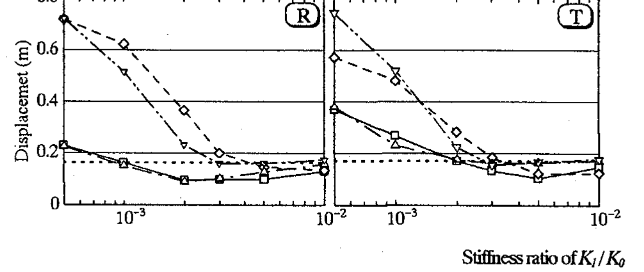

Fig. 10 Out-of-plane peak deck displacement

6.2 Bearing displacement

The calculated results of in-plane and out-of-plane bearing displacement are shown in Figs 11 and 12. Fully isolated bearings with stiffness ratios less than 2 x10 $^{-3}$ have suffered high in-plane and out-of-plane deformations up to 0.73 m for systems of top bearing and 0.59 m for systems of bottom bearings, respectively. Such high deformations are difficult to be accommodated by this type of bearings. For such small stiffness rations, the lateral restrainer could reduce the bearing deformations more than 20%. As the bearing stiffness increases the corresponding bearing deformations highly decreases, the maximum bearing deformations associated with stiffness ratio equal to 2 x10^{-3} do not exceed 0.22 m for all isolation systems. Top isolators exhibit higher deformations than the bottom isolators. This behavior may be attributed to the effect of axial load which increases for bottom bearing systems due to the increase of pier weight.

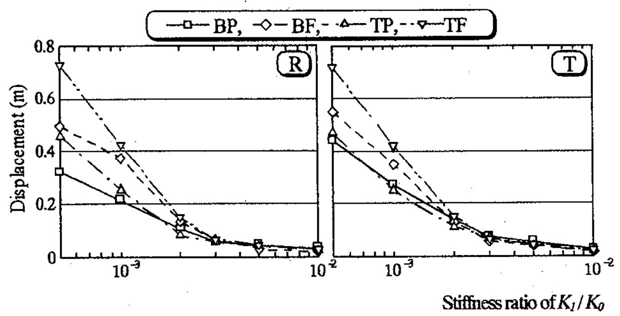

Fig. 11 In-plane peak bearing displacement

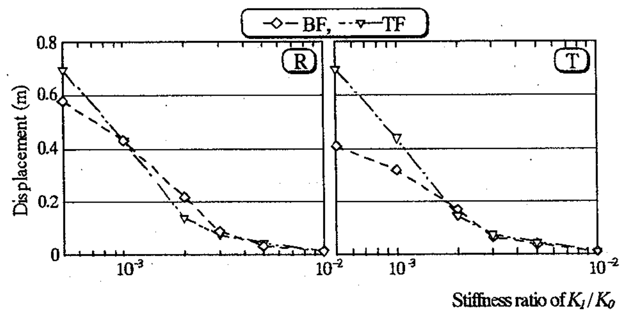

Fig. 12 Out-of-plane peak bearing displacement

6.3 Deck acceleration

The amplification factor (ratio of deck acceleration to ground acceleration) is calculated for the different systems to identify the amount of transmitted earthquake motion. It is observed that the nonisolated viaduct exhibits high and similar acceleration response for both cases of radial and tangential configurations with an amplification factor more than 3.33 for the in-plane and 3.42 for the out-of-plane accelerations, respectively. The isolation systems are effective in controlling such high acceleration of deck depending on both system and stiffness ratio as shown in Figs. 13 and 14. It is clear that for all systems the response highly increases beyond stiffness ratio equal 3 x10^{-3}. The in-plane response of acceleration is close foe all systems with relatively better performance for the fully isolated systems. Partially isolated systems suffer high out-of-plane deck acceleration. Applying full isolation improves this response, the most sound improvement is observed for stiffness ratios less than 3 x10^{-3}.

6.4 Seismic forces

The performance of seismic forces is evaluated though the determination of peak values of overturning moment and curvature over all piers of each studied system of curved viaduct. The in-plane peak response of overturning moment is shown in Fig. 15, maximum and minimum ratios of reduction in the values of in- plane

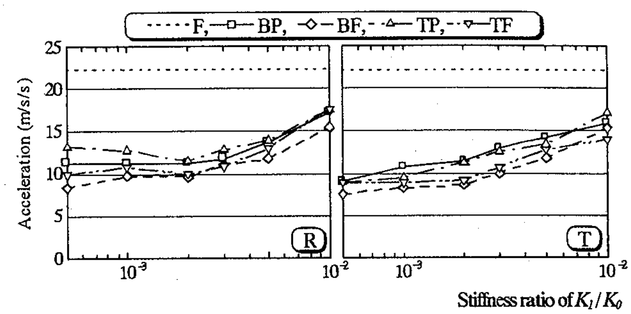

Fig. 13 In-plane peak deck acceleration

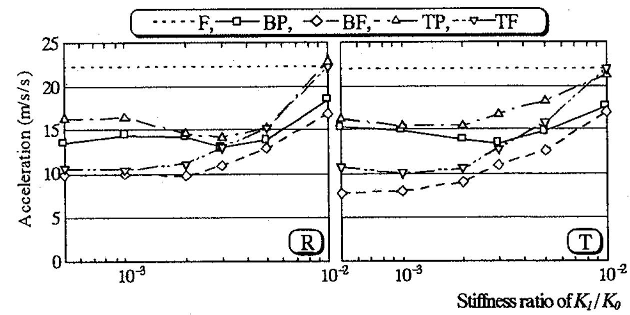

Fig. 14 Out-of-plane peak deck acceleration

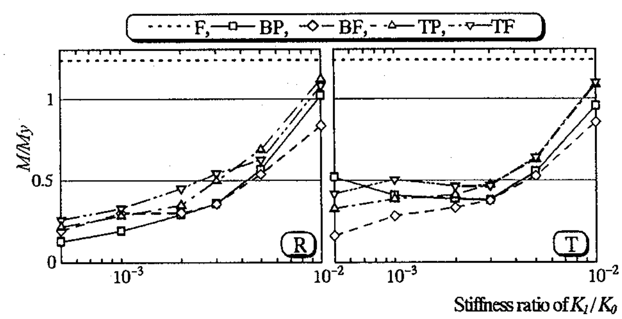

Fig. 15 In-plane peak pier overturning moment

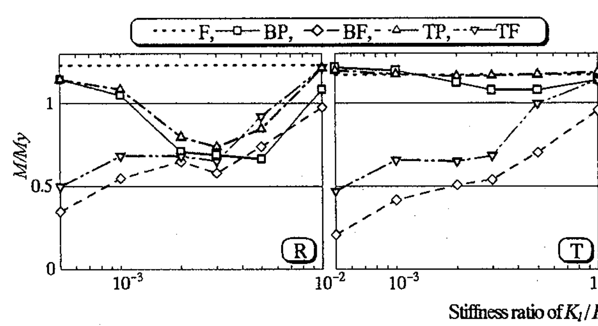

Fig. 16 Out-of-plane peak pier overturning moment

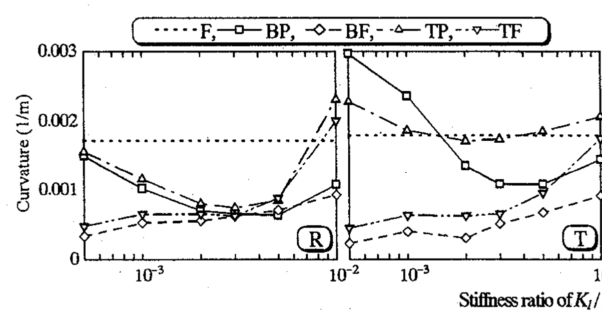

Fig. 17 Out-of-plane peak pier curvature

Table 1 In-plane bending moment reduction ratios (%)

Case	Radial		Tangential	
	Max	Min	Max	Min
BP	90	18	58	23
BF	84	32	87	30
TP	82	10	69	12
TF	79	13	66	11

Table 2 Out-of-plane bending moment reduction ratios (%)

Case	Radial		Tangential	
	Max	Min	Max	Min
BP	46	1	8	-5
BF	72	21	82	18
TP	40	1	1	-2
TF	60	1	60	3

overturning moment related to the nonisolated viaduct are shown in Table 1. It can be clearly seen that for almost all cases of isolation systems the overturning moment is highly affected by the stiffness of isolators, as the stiffness increases the corresponding values of moment highly increases especially beyond stiffness ratio equal 3 x10 $^{-3}$. All isolation systems with radial arrangement are effective in completely eliminating the high plasticity displayed by the nonisolated viaduct while systems of tangential configuration demonstrate plasticity only for high isolator stiffness equal 1 x10 $^{-2}$. The response of the isolation systems is relatively close with slight improvement for the systems of bottom isolation than systems of top isolation. Systems with radial configuration reveal better performance for isolator stiffness ratio less than 3 x10 $^{-3}$.

The nonisolated original viaduct suffers very high out-of-plane plasticity. Partially isolated systems with tangential configuration completely fail to overcome such hazardous behavior. This response may be attributed to the high restriction of such configuration to the out-of-plane motion of isolation bearings, which leads to response independent of the stiffness of bearings and close to the nonisolated one. Partially isolated systems with radial configuration exhibit different response; systems with bearing stiffness ratios ranging from 2 x10 $^{-3}$ to 5 x10 $^{-3}$ could prevent the propagation of such high plasticity. Applying full isolation is effective in improving the response for all ranges of bearing stiffness in case of tangential configuration and up to stiffness ratio equal 2 x10 $^{-3}$ for radial one with better response for systems with bottom isolators than that of the top bearings as shown in Fig. 16 and Table 2.

Such behavior can also be observed from the out-of-plane peak response of curvature. Partially isolated systems with tangential configuration have undergone very high curvature exceeded that of the nonisolated viaduct in many cases. Applying full isolation is effective in highly improving such behavior for all range of stiffness. Systems with bottom bearings exhibit performance relatively better than that obtained for systems with top isolators shown in Fig. 17.

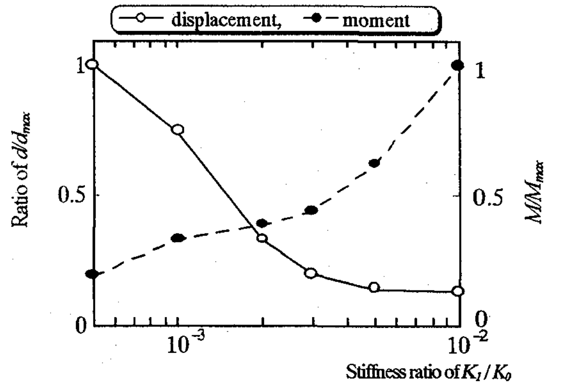

Fig. 18 Method of determining the optimum isolator stiffness ratio

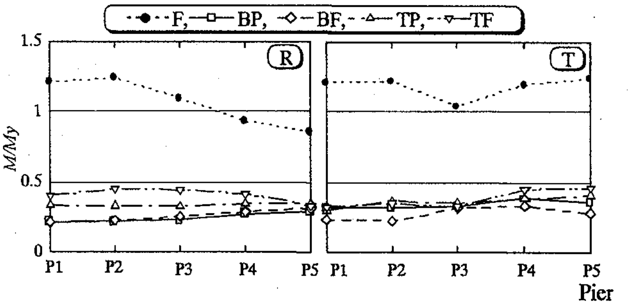

Fig. 19 In-plane peak overturning moment for all piers

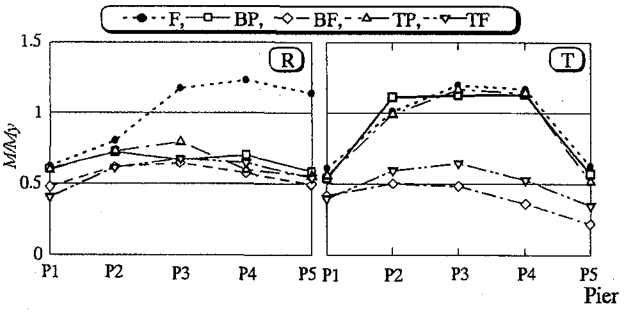

Fig. 20 Out-of-plane peak overturning moment for all piers

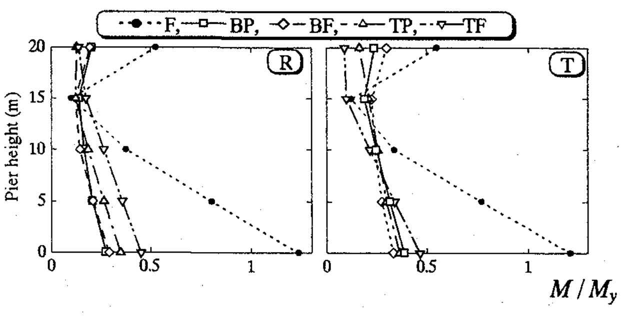

Fig. 21 In-plane peak overturning moment over pier height

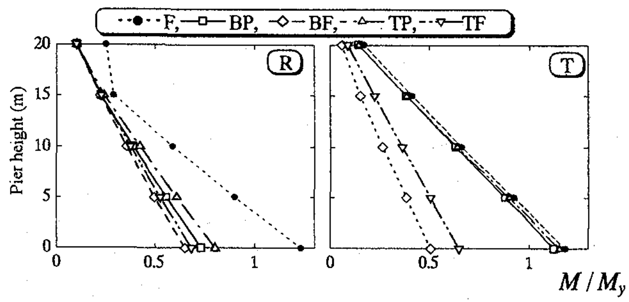

Fig. 22 Out-of-plane peak overturning moment over pier height

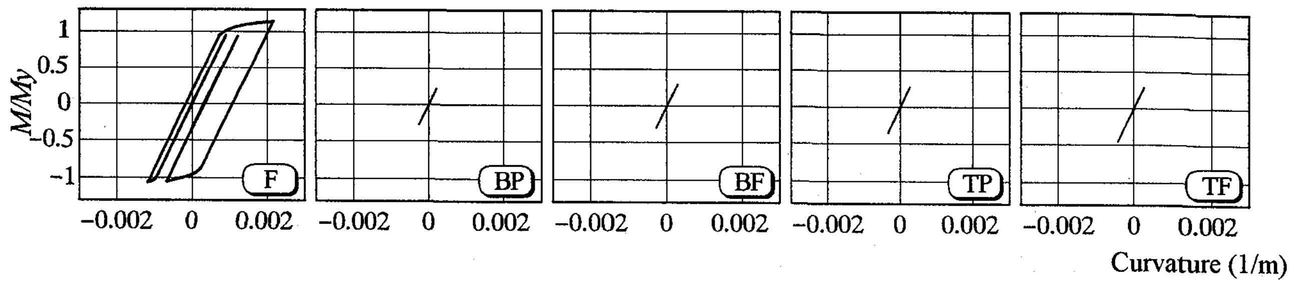

Fig. 23 In-plane moment and curvature relationship, radial configuration

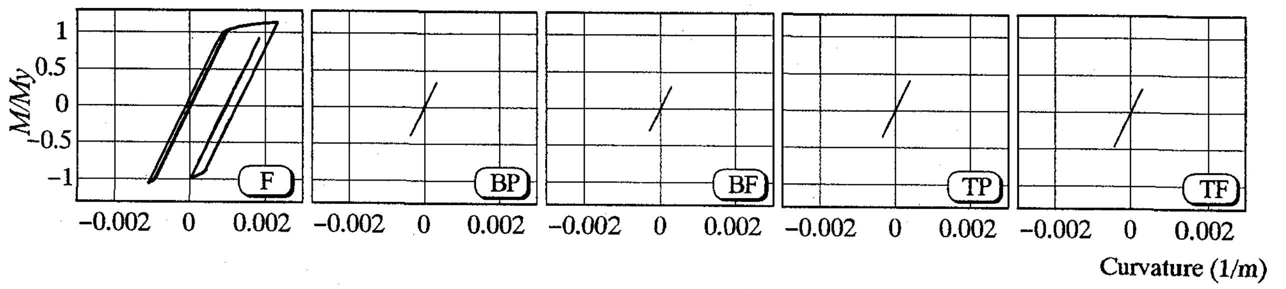

Fig. 24 In-plane moment and curvature relationship, tangential configuration

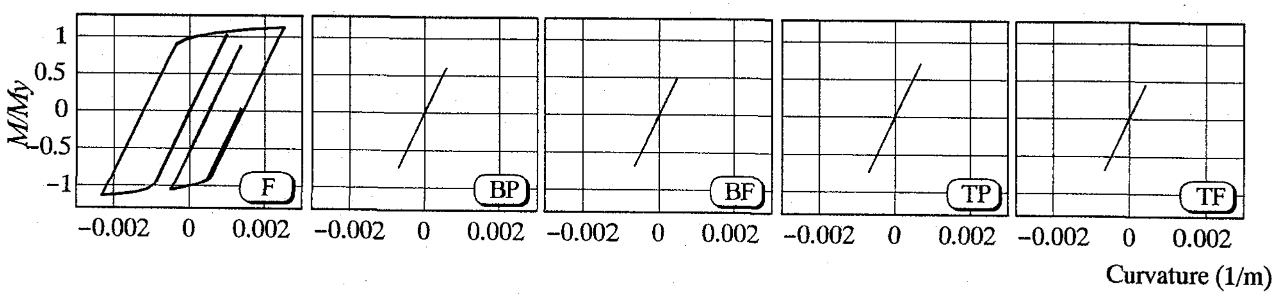

Fig. 25 Out-of-plane moment and curvature relationship, radial configuration

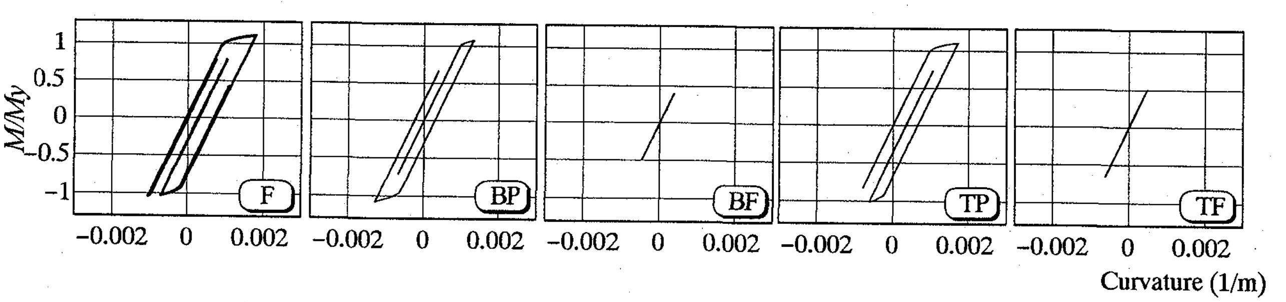

Fig. 26 Out-of-plane moment and curvature relationship, tangential l configuration

Considering displacement and overturning moment to have the same importance in determining the design parameters of isolators, characteristic stiffness, it can be seen that systems isolated with bearings which have stiffness ratios equal to 2 x10^{-3} and 3 x10^{-3} reveal the optimum response. Fig. 18 shows the method of selecting such design parameters for system of bottom bearing with tangential configuration through the reciprocal moment and displacement relationship.

Depending on the proposed reciprocal moment and displacement relationship for determining the optimum stiffness ratio of isolators, isolated systems with isolator stiffness ratio equal to 2 x10^{-3} are chosen to integrally discuss their nonlinear dynamic response at the base of all piers and over the height of the piers, their maximum hysteretic moment and curvature relationship at the base of the piers are also explained and compared to the response of the nonisolated case.

In-plane and out-of-plane overturning moment for all piers is shown in Figs. 19 and 20. Applying isolation is very effective in highly improving the plastic behavior of nonisolated case to ratios more than 75% and 68% for systems of bottom bearings and 62% for systems of top bearings with radial and tangential configurations, respectively. Partially isolated viaduct with tangential configuration behaves almost as the nonisolated one in the out-of-plane. Full isolation is necessary to improve this behavior with reduction up to 57% and 44% for bottom and top isolated systems, respectively. Systems with radial configuration behave better, the response of all piers is in the elastic range. For these cases the difference between partial and full isolations is not as large as the case tangential configuration, the response improved from 40% to 47% and from 35% to 45% for bottom and top isolation, respectively.

The peak response of overturning moment over the height of piers is studied and shown in Figs 21 and 22. It can be observed that the response of the nonisolated viaduct is not uniformly distributed over the height of the pier but it sharply increases near the foot of pier, this behavior can lead to local buckling or rupture at the bottom of pier. Isolation systems are effective in introducing almost in-plane uniform response over the height. For the out-of-plane moment, while the isolation systems with radial configuration behave almost similarly, the partial isolation systems with tangential configuration behave close to the nonisolated viaduct. This behavior can be highly improved by using full isolation system.

The maximum in-plane and out-of-plane moment and curvature relationship at for the same cases at foot of piers in comparison to the nonisolated viaduct are shown in Figs 23 through 26. It is clearly seen that the nonisolated viaduct has suffered high in-plane and out-of-plane severe moment and curvature hysteressis loops. Such dangerous behavior may lead to the occurrence of buckling phenomenon or rupture of these piers and hence the collapse of the whole structure. All the isolated systems can successfully eliminate such in plane hysteressis loops achieving a reduction ratios in curvature more than 80%.

Regarding the out-of-plane moment curvature performance, it can be clearly seen that partially isolated systems with tangential configuration suffer hysteressis loops close to that observed for the nonisolated viaduct with rigid connections achieving slight improvement in the response equal to 25% and 5% for systems of bottom and top bearing, respectively. Radial isolation systems display better response, the corresponding ratios of improvement are 72% and 67%. Applying full isolation is effective in improving the response of tangential configuration achieving reduction ratios more than 73% and 65% for systems of bottom and top bearing respectively, while slight improvement is observed for radial configuration systems.

7. Conclusions

A comparative study is carried out to evaluate the effect of isolation conditions such as installation position, configuration and type on the nonlinear response of curved viaduct systems. The performances of these systems over a wide range of isolator design parameters are obtained and compared to the response of the originally nonisolated one. A simple method is presented to determine the optimum behavior of isolation systems relying on the behavior of displacement and overturning moment. Accordingly, the behavior of these systems with the optimum value of isolator stiffness ratio is exclusively discussed. The following conclusions can be drawn out.

(1) The nonisolated viaduct suffers high in-plane and out-of-plane plasticity at the base of the piers. All the isolation systems are effective in eliminating the in-plane plasticity up to stiffness ratio equal to 5 $x10^{-3}$. Partially isolated systems fail to improve the out-of-plane performance of the nonisolated viaduct for the whole range of stiffness in case of tangential configuration and for some range of isolation stiffness ratio for systems with radial configuration.

(2) The position of isolators relatively affects the response yielding better performance for systems isolated with bottom isolators. The arrangement of bearings results in better out-of-plane performance for systems with radial configuration than those with tangential one due to the restriction of this configuration to the lateral movement of isolators. While side restrainers highly control the out-of-plane displacement of deck for small isolator stiffness ratios they yield high out-of-plane forces response. Applying full isolation highly improves the out-of-plane response of systems with tangential configuration up to isolator stiffness ratio equal to 5 $x10^{-3}$ and for stiffness ratios less than 2 $x10^{-3}$ for systems with radial configuration.

(3) Systems utilizing isolators with small stiffness ratio display high deck and bearing displacement, which may lead to the fall of deck in cases of systems with top bearings or require an expensive bearing to accommodate such high deformations. On the contrarily, the response of in-plane overturning moment is proportional to the stiffness of isolators, systems with high isolator stiffness exhibit high moment response and plasticity in some cases. It is inevitable to obtain a balanced response between displacement and moment to ensure a safe behavior of viaduct. Using a simple reciprocal moment and displacement relationship, it is observed that for this structure and the applied excitation, isolators with stiffness ratios 2 $x10^{-3}$ and 3 $x10^{-3}$ display the optimum behavior.

(4) Investigating the behavior of systems with isolator stiffness ratio equal to 2 $x10^{-3}$ shows that all isolation systems reveal in-plane uniform response for all piers and over its height and can highly improve the response of the nonisolated one. Partially isolated systems with tangential arrangement display high out-of-plane moment and curvature response close to that of the nonisolated one.

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