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Viscoelastic Dampers in Bridges

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ABSTRACT

This paper discusses the effectiveness of viscoelastic (VE-) dampers in seismic damage mitigation of bridges. A nonlinear hysteretic VE-element is used to study the effectiveness of viscoelastic dampers in several bridge models. The nonlinearity introduced due to impact (pounding) of adjacent spans at the expansion joint is also considered. The linear response simulation of the VE-damped bridge is carried out using modal approach and some recommendations regarding the correct global and local response simulation made.

INTRODUCTION

Viscoelastic (VE-) dampers are currently being investigated and proposed for their effectiveness in seismic damage mitigation of buildings (e.g., Kasai & Munshi, 1994). These dampers add both stiffness and damping to the structure resulting in significant reduction of its seismic response. Thus far, little effort has been made towards the application of viscoelastic dampers in bridges, where they could prove to be effective. Grenier (1992) studied the effectiveness of compression type viscoelastic fluid piston device for seismic damage mitigation of bridges. This device used in Italy and France has been found to be effective in bridge seismic response mitigation. Viscoelastic dampers could be placed on the roller end of the bridge girders so as to deform in shear due to longitudinal translation of the girders. A cylindrical VE-device which would allow shear deformation of the VE-layers between the sliding internal cylinder and the external cylinder (Fig. 1) could be used for this purpose. The viscoelastic material will allow free thermal movement of the girders due to its creep deformation. However, it will significantly stiffen the structure and dampen its response during a high frequency and random earthquake shaking. This is the first paper using an accurate VE-hysteretic element (Kasai et al, 1993) to study the bridge application of viscoelastic dampers. This paper also discusses the response simulation of a VE-damped bridge through simplified state-of-practice linear analysis methods.

The study is conducted on several bridge systems with viscoelastic dampers placed at different locations so as to study their effectiveness. Through this study, it will be shown that viscoelastic dampers are effective in seismic damage mitigation of bridges due to the following; (1) high damping in excess of 50% can be attained with practicable sizes of VE-damper layers; (2) substantial horizontal stiffness is added to the bridge; (3) the added damper devices cause significant reduction of overall response of the bridge; (4) reduction and redistribution of forces relieves the strength demand on any particular pier; and, (5) the in-phase motion is promoted between adjacent spans having different dynamic characteristics leading to small opening and closing of the deck joints and reducing the possibilities of collision or falling off of the deck.

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VISCOELASTIC DAMPERS

Analytical Model. Viscoelastic material is stiff at high frequencies (or low temperatures) and soft at low frequencies (or high temperature). The typical response of a viscoelastic damper under cyclic loading is in the form of an inclined ellipse as shown in Fig. 2. The stiffness and damping properties of a VE-damper are discussed in detail elsewhere [Kasai et. al, 1993]. VE-hysteretic inclined elliptic loop can be expressed as a combination of two components; the elastic component with stiffness K_D' and viscous damping component having elliptic hysteretic loop with no inclination. When incorporated into a structure, VE-dampers add the stiffness K_D' and damping as shown in Fig. 2. Since the damper stiffness and damping are frequency and temperature dependent, the overall response of VE-damped bridge depends upon its vibration frequency and ambient temperature. Further, the temperature rise of VE-damper due to energy dissipation causes a gradual degradation of its stiffness and damping with increasing loading cycles.

Authors have developed an efficient and accurate analytical scheme to simulate the complex behavior of viscoelastic damper for structural application [Kasai et al, 1993]. The nonlinearity due to temperature rise of VE-damper is taken care of by using the principle of thermo-mechanics. The temperature-frequency equivalence principle of classical viscoelasticity is used to account for the change stiffness and damping of the VE-damper due to change in ambient temperature and damper temperature rise. The model performance is verified for several harmonic, random, and pounding (impact) type earthquake motions and good correlation with experiments obtained [Kasai et al, 1995]. The model is used in this paper to simulate the VE-damper behavior in the time history nonlinear analyses of VE-damped bridges.

SEISMIC ANALYSIS OF DIFFERENT BRIDGE MODELS

Three different bridge models, Model-1, 2, and 3 shown in Fig. 3 are analyzed for the undamped (without VE-dampers) and damped (with VE-dampers) situations. The mass and stiffness properties of superstructure and substructure are selected in a manner to represent the existing bridge systems primarily designed for the dead and vehicular loading and not necessarily for the seismic loading. It would be relevant to simulate the seismic vulnerability of such bridges and study the damper effectiveness for their retrofit. Analysis of the bridges is carried out for the 1.5 times El Centro ground motion (peak ground acceleration = 0.52g) assuming 5% damping for the original undamped structure. For simplicity, all the bridge models are assumed to be contact free against the abutments.

Model-1 Bridge without VE-Dampers. The Model-1 bridge (Fig. 3(a)) is a 2 lane, 2 span (2@120 feet) continuous girder bridge with 40 feet high intermediate R.C column of 5 feet equivalent diameter and 1% steel reinforcing. The compression capacity of column is 8600 kips and its moment capacity is 38000 kip-in. The masses are lumped at 5 equidistant nodes for each span and 4 nodes for column with each node having 3 degrees of freedom. The weight of the bridge is about 1200 kips. The far ends of the deck rest on rollers supported on rigid abutments. The period of this bridge without VE-dampers is 2.17 seconds. This bridge is seismically weak and develops a maximum elastic displacement of about 12.8 in. (Fig. 4), under 1.5 times the El Centro earthquake. Analysis of the inelastic bridge model under the same earthquake indicates a plastic hinge rotation of about 0.02 rad. at the column bottom (Table 1). The vulnerability of the bridge is obvious through the large displacement and the resulting strength demand on the pier (Table 1).

Model-1 Bridge with VE-Dampers. A VE-layer with area 200 in^2 and thickness 1 in. is provided between either abutment and deck as shown in Fig. 3(a). This damper size gives 51% damping ratio determined at the fundamental period 1.2 seconds, and at ambient temperature 24°C using a damping ratio estimation method [Munshi & Kasai, 1994]. Nonlinear time history analysis for 1.5 times El Centro earthquake indicates that bridge responds elastically. The maximum displacement reduces to only 2.2 in. (Fig. 4), and column shear and moment become 0.2 times those of undamped elastic bridge (Table 1), and are well within capacities. Thus, VE-dampers are very effective in reducing the strength demand on the pier bents of weak bridges as well as magnitudes of opening and closing of the expansion joints.

Model-2 Bridge without VE-Dampers. This model has the same superstructure and substructure properties as Model-1 except that span discontinuity exists at the intermediate pier which seats roller bearing from span A and pinned bearing from span B as shown in Fig. 3(b). This model is selected to study the vulnerability of deck expansion joint over the intermediate pier, for situations such as pounding (impact) damage as well as excessive joint opening due to dissimilar dynamic characteristics of the two adjacent spans, A and B. The vibration periods of the bridge are 1.6 seconds (span B longitudinal displacement mode), and 0.32 (span A vertical deformation mode). Analysis of this bridge is carried out for two types of expansion gaps of 1 in., and 3 in. assuming 5% damping ratio. The nonlinearity introduced due to pounding of the adjacent spans at this joint is simulated by using the gap element [Maison 1992]. The spring stiffness of the gap element is set to 13000 kip/in, equal to the in-plane deck stiffness of the spans.

For the 3 in. gap, elastic bridge model (span B) develops a maximum displacement of 6.5 in. as shown in Fig. 5(a), (column forces in Table 1). The maximum relative movement at the joint (Table 1) are 6.5 in. (opening), and 3.2 in. (closing). A closing larger than 3 in. in this case indicates contact at the joint which results in an impact force of about 2600 kip (Table 1.). The responses of the inelastic bridge model for same 3 in. gap indicates a column base plastic hinge rotation of 0.013 rad., and maximum displacement 7.7 in (Fig. 5(a)). The maximum impact force in this case is 791 kip.

The situation is more serious for 1 in. gap at the joint. The joint closing of 1.31 in. and impact of 4014 kip, increase the responses of the elastic bridge model (Fig. 5(b), Table 1.). For the inelastic model case, the plastic hinge rotation of the column reaches about 0.016 rad. with an impact force of 1657 kips at the expansion joint. This bridge could suffer serious damage due to large impact force at the expansion joint.

Model-2 Bridge with VE-Dampers. A damper with area 200 in^2, and thickness 1 in. is provided at two locations, A and B shown in Fig. 3(c). This damper size results in a global damping of about 55% for 0.8 seconds (span B) vibration period of the bridge. Seismic analysis of the VE-damped bridge for 3 in. gap at the joint indicates no contact due to significant reduction of displacement (Fig. 5(b)). The maximum joint opening and closing reduce to 1.38 in and 1.75 in. (Table 1). For 1 in. gap the maximum impact force of 1430 kip (1.11 in. closing), is about 0.3 times that for elastic bridge model without VE-dampers (Table 1). Fig. 6 compares the hysteresis of the VE-damper at the joint for 1 in. and 3 in. gaps, and shows that for 1 in. gap the VE-damper unloads due to contact as expected. This analysis indicates that VE-dampers could reduce the pounding damage by reducing both the incidence as well as magnitude of impacts if any, at the expansion joints of the bridges.

Model-3 Bridge Without VE-Dampers. The superstructure properties of this bridge are similar to that of Model-2, but span A abutment is replaced by a column of the same section as column B with half as much height (Fig. 3(c)). This model is used to simulate the case where span A also vibrates longitudinally as span B. The out-of-phase motion of the adjacent structures results due to their dissimilar dynamic characteristics. VE-damper, however, will help bring the two structures into a phase as will be explained. This model is also used to study the effect of damper location on bridge response. The vibration periods of this bridge are 1.6 seconds (span B), and 0.53 (span A). Fig. 7(a) shows that displacement response of the two adjacent spans over pier B are quite out-of-phase which could result in falling of deck due to excessive joint opening (9.7 in.) or pounding damage due to excessive closing (3.3 in. against 3 in. gap). The inelastic bridge model also indicates a similar trend (Table 1).

Model-3 Bridge with Two VE-Dampers. Two dampers with 200 in^2 x 1 in. size are placed at locations A and B as shown in Fig. 3(c). Fig. 7(b) shows that VE-dampers promote in-phase motion of the two dissimilar structures, significantly reducing the joint opening (1.1 in), and closing (0.91 in.). This eliminates the pounding for a gap of 1 in. and more, and also reduces the girder seat requirement for the expansion end (span A).

Model-3 Bridge with One VE-Damper. This study considers the effectiveness of damper placement in a bridge. The two locations considered are; (1) span A at intermediate pier, and (2) span B at abutment. A damper of 400 in^2 x 1 in size is placed first at location A only, and then at location B only. Fig. 8 and Table 1 shows that a damper at location B is more effective in controlling the seismic response as compared to the damper at location A (Fig. 8(b)), due to relatively rigid abutment as compared to the pier. However, Fig. 8(b) shows that a damper at location A is more effective in promoting in-phase response between the two adjacent structures (see also Table 1. for joint opening/closing). The impact force at the joint with damper at location A is only about 0.3 times (90 kip) that with damper at location B (280 kip).

LINEAR RESPONSE SIMULATION

The vibration frequency and damping ratio of the VE-damped structure are obtained through the so-called modal strain energy method (Munshi & Kasai, 1994), for the bridge having original stiffness K_o and damper equivalent stiffness K_D'. Model-1 (Fig. 3(a)) is considered for seismic response simulation through this linear approach (period 1.2 sec., and damping ratio 0.51). Fig. 9 compares the global displacement history of the VE-damped bridge (under 1.5 El Centro) predicted through linear method and that predicted by accurate analysis using nonlinear response of dampers. Although linear approach closely predicts the displacement history (Fig. 9), it significantly underestimates the damper force as 84 kip against the actual 170 kip determined through nonlinear approach. A correction proposed by the writers' [Munshi & Kasai, 1994] is important for design of damper and its connecting members.

CONCLUSIONS

The viscoelastic dampers of even relatively small size are very effective for seismic damage mitigation of bridges, since these; (1) reduce the overall response; (2) promote in-phase motion of the adjacent spans; and (3) reduce the magnitude of impact damage if any. A damper placed at abutment is more effective in controlling the response, while as a damper placed between two adjacent spans is more effective in promoting the in-phase motion between them. The effect of damper temperature rise not reported here is seen to alter the damper effectiveness and the bridge response. For this damper strains would have to be well limited so as to minimize the damper softening.

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Table 1. Seismic Response Quantities of 3 Bridge Models.

Model	Damping Ratio	Vibration Period (Sec)	Deck Joint Gap (in.)	Col. Peak Dispt. (in.)	Col. Peak Shear (kips)	Col. Peak Moment (kip-in.)	Column Plastic Rotation (rad.)	Joint Opening/ Closing (in.)	Impact Force (kips)
1 (Elastic)	0.05	2.17	-	12.8	338	161,800	-	-	-
1 (Inelst.)	0.05	2.17	-	6.8	103	46,600	0.02	-	-
1 with VE	0.51	1.20	-	2.2	64	29,500	0.0	-	-
2 (Elastic)	0.05	1.60	3.0	6.5	169	81,000	-	6.5/3.20	2600
2 (Inelst.)	0.05	1.60	3.0	7.7	104	49,000	0.013	7.8/3.06	790
2 with VE	0.55	0.80	3.0	1.8	51	27,700	0.0	1.4/1.75	0.0
2 (Elastic)	0.05	1.60	1.0	7.6	208	96,500	-	7.6/1.31	4010
2 (Inelst.)	0.05	1.60	1.0	8.3	106	49,100	0.016	8.3/1.13	1660
2 with VE	0.55	0.80	1.0	1.7	69	21,200	0.0	1.7/1.11	1430
3 (Elastic)	0.05	1.60	3.0	7.0	172	86,900	-	9.7/3.30	3400
3 (Inelst.)	0.05	1.60	3.0	7.2	101	47,300	0.01	7.7/3.10	940
3 with VE (1)	0.54	0.83	3.0	2.3	69	28,900	0.0	1.1/0.90	0.0
3 with VE (2)	0.49	0.93	1.0	4.0	86	40,300	0.0	1.2/1.0	90
3 with VE (3)	0.55	0.80	1.0	2.6	47	20,900	0.0	1.9/1.0	280

(1) Damper Placed at Supports A and B.
(2) Damper Placed at Support A only.
(3) Damper Placed at Support B only.

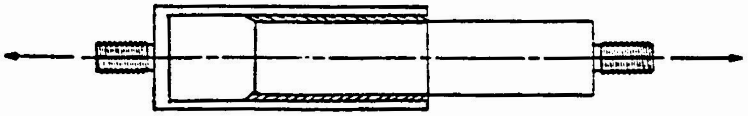
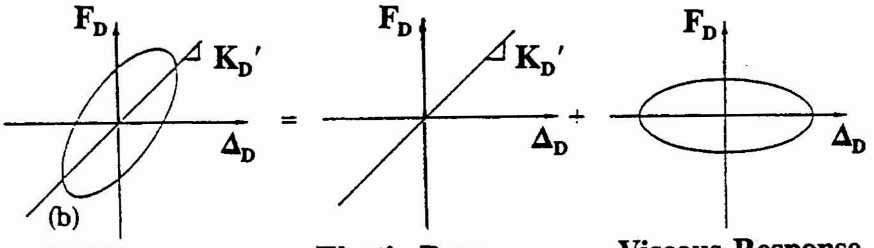


Fig. 1 Viscoelastic Device for Bridges.



VE-Damper Elastic Response Viscous Response

Fig. 2 Stiffness and Damping of VE-Damper.

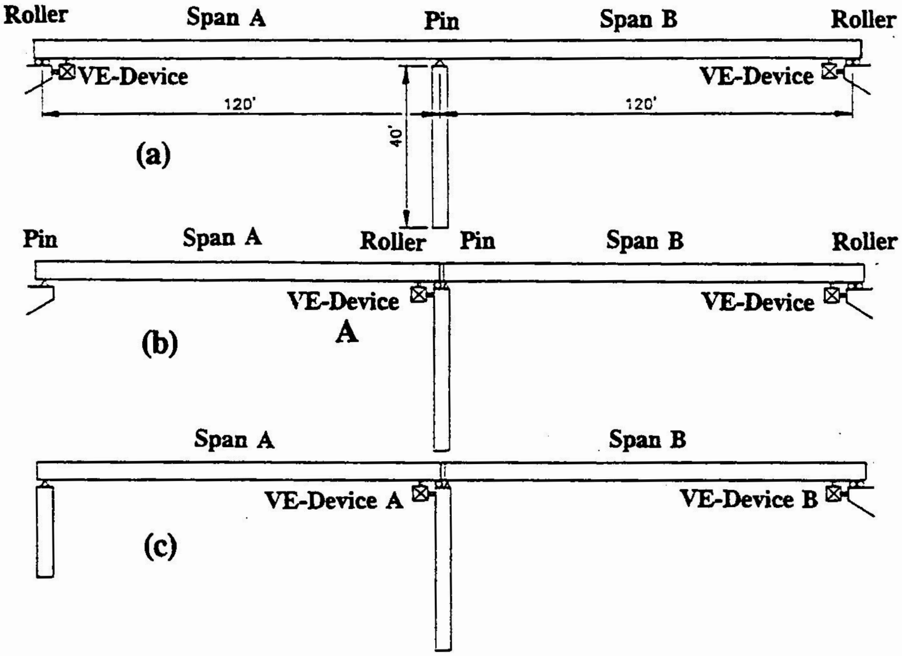


Fig. 3 Bridge Models used for Seismic Analysis.

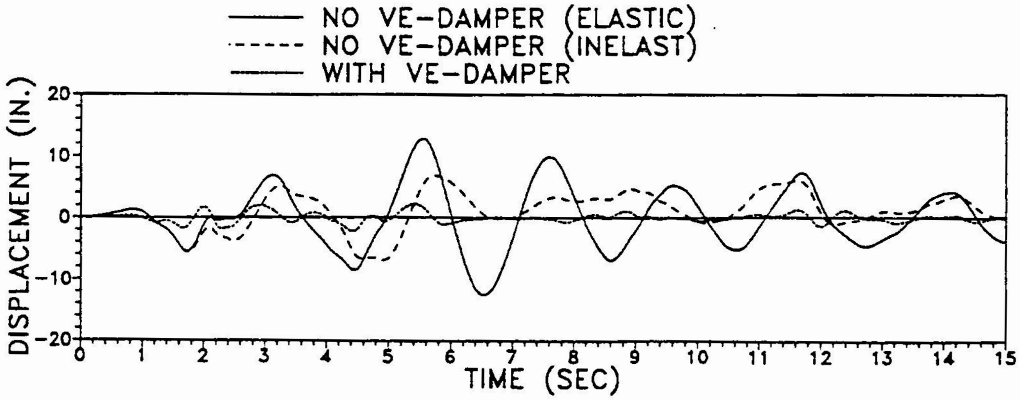


Fig. 4 Displacement Response of Model-1 Bridge.

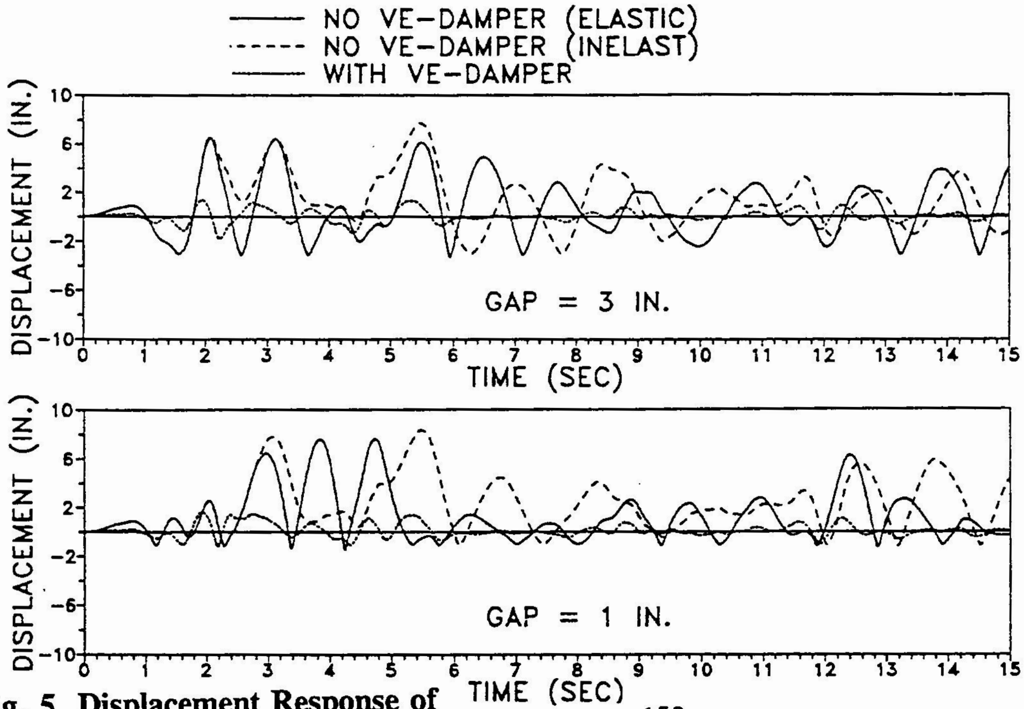


Fig. 5 Displacement Response of Model-2 Bridge.

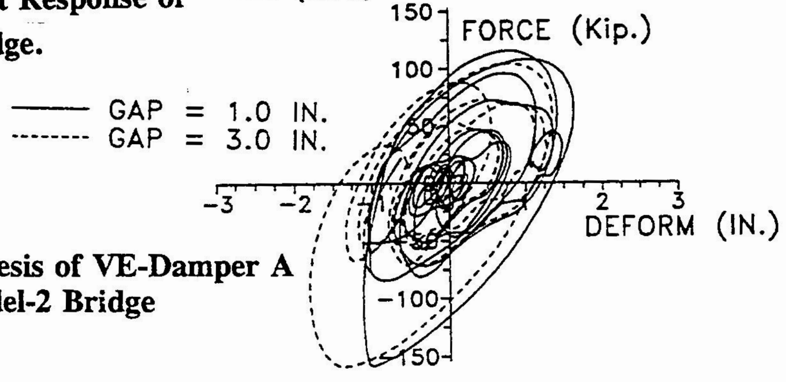


Fig. 6 Hysteresis of VE-Damper A in Model-2 Bridge

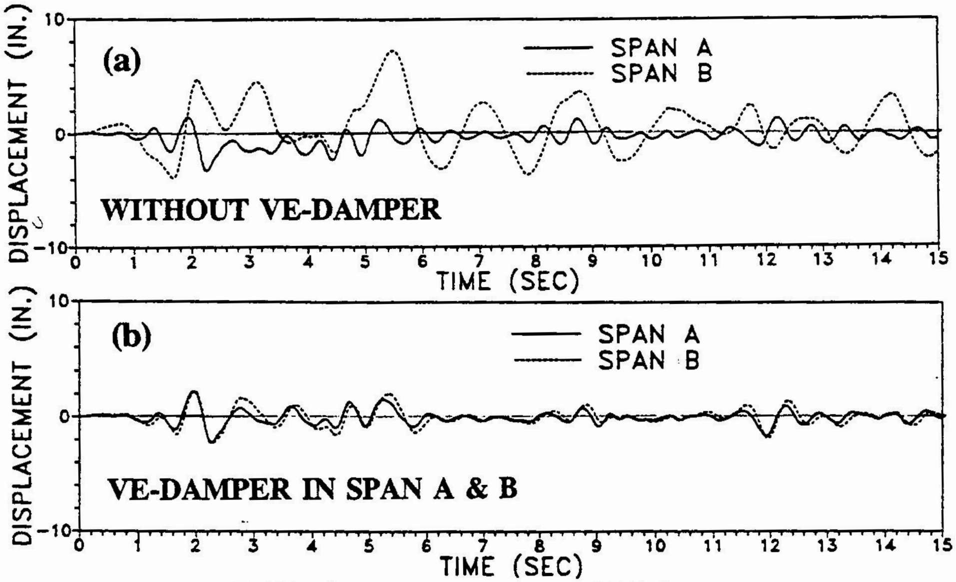


Fig. 7 Displacement Response of Model-3 Bridge.

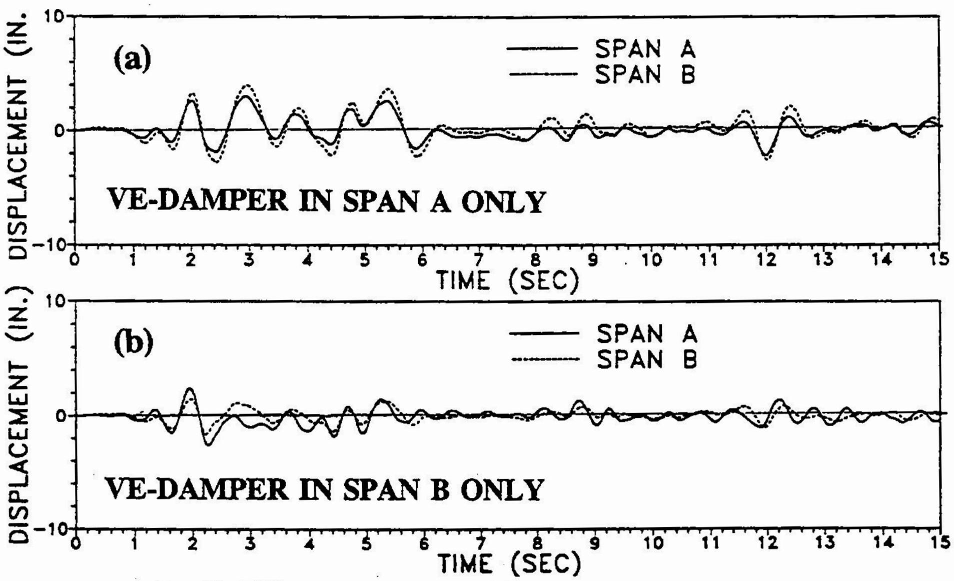


Fig. 8 Effectiveness of VE-Damper Placement.

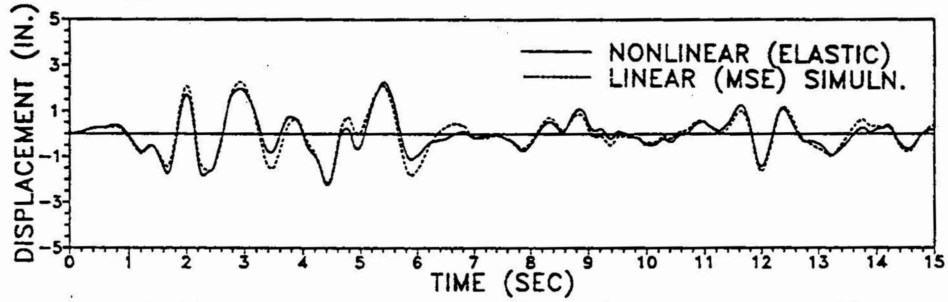


Fig. 9 Linear Seismic Response Simulation of Model-1 Bridge.