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Vertical ground motion influence on seismically isolated & unisolated bridges

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ABSTRACT

In this study, influences of vertical ground motion on seismically isolated bridges were investigated for seven different earthquake data. One assessment of bearing effect involves the calculation of vertical earthquake load on the seismically isolated bridges. This paper investigates the influence of vertical earthquake excitation on the response of briefly steel girder composite bridges (SCB) with and without seismic isolation through specifically selected earthquakes. In detail, the bridge is composed of 30m long three spans, concrete double piers at each axis supported by mat foundations with pile systems. At both end of the spans there exists concrete abutments to support superstructure of the bridge. SCBs which were seismically isolated with ten commonly preferred different lead-rubber bearings (LRB) under each steel girder were analyzed. Then, the comparisons were made with a SCB without seismic isolation. Initially, a preliminary design was made and reasonable sections for the bridge have been obtained. As a result of this, the steel girder bridge sections were checked with AASHTO provisions and analytical model was updated accordingly. Earthquake records were thought as the main loading sources. Hence both cases were exposed to tri-axial earthquake loads in order to understand the effects under such circumstances. Seven near fault earthquake data were selected by considering possession of directivity. Several runs using the chosen earthquakes were performed in order to be able to derive satisfactory comparisons between different types of isolators. Analytical calculations were conducted using well known structural analysis software (SAS) SAP2000. Nonlinear time history analysis was performed using the analytical model of the bridge with and without seismic isolation. Response data collected from SAS was used to determine the vertical load on the piers and middle span midspan moment on the steel girders due to the vertical and horizontal component of excitation. Comparisons dealing with the effects of horizontal only and horizontal plus vertical earthquake loads were introduced.

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1. Introduction

A structure should remain elastic during seismic excitation to avoid damage and should also retain the ability to undergo large deformations to facilitate energy dissipation. These demands seem paradoxical with the application of regular structural elements. Therefore, special structural elements should be considered to accomplish these crucial two extremes. The seismic isolation systems can be accounted for the special structural elements which provide structural elements stress levels in elastic range and energy dissipation by large deformation capability.

Vertical earthquake effect on bridges is a serious issue since it has considerable contribution to design stresses of structural members. This is why several studies regarding the vertical earthquake loads on bridges have been performed so far. Most of the studies focused on vertical ground motion influence on bridges without seismic base isolation.

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McRae and Tagawa (2002) undertook a dynamic inelastic time-history analyses of single-degree-of-freedom (SDOF) bilinear oscillators in order to determine the ability of the Coefficient Method (FEMA273/FEMA356) and the Capacity Spectrum Method (ATC-40) to predict the total displacement demands of simple structures. Both the Coefficient Method (CM) and the Capacity Spectrum Method (CSM) were calibrated to obtain the exact inelastic response displacements for near-fault (NF) and far-fault (FF) shaking. As a result of dynamic inelastic analyses of single degree of freedom bilinear oscillators McRae and Tagawa (2002) indicates that, oscillators with demands estimated by the CM, and with fundamental periods less than about 0.8 s, were not affected significantly by near-fault shaking effects. For longer fundamental period oscillators, oscillator strengths may need to be increased by more than 60% to account for inelastic shaking effects from NF sites in the region of positive directivity compared to that for shaking from Far Fault (FF) or NF near-epicenter sites for the same target displacement ductility. NF shaking did not cause significant trends in the displacement demands of oscillators evaluated by the CSM method.

A study by Kunnath et al. (2008) indicates that, studies in the past have clearly identified several potential issues that deserve additional attention. The study is undertaken with the objective of assessing the current provisions in The Caltrans Seismic Design Criteria (SDC) 2006 for incorporating vertical effects of ground motions in seismic evaluation and design of ordinary highway bridges. In the code, it is required to take into account the vertical ground motion for ordinary highway bridges where the site peak rock acceleration is 0.6 g or greater. In order to consider the vertical ground motion an equivalent static vertical load required to be applied to the superstructure to estimate the effects of vertical acceleration. The loading procedure aims to perform a separate analysis to check nominal capacity of superstructure against loading stated in SDC 006. As a result of the study, two major conclusions were found. First, vertical ground motions have significant effects on the axial force demand in columns. Second, vertical ground motions have significant effects on moment demands at the middle of the span. Particularly, for the case of the shear demand and shear capacity fluctuations. It should also be noted that axial forces vary at much higher frequencies than lateral forces. Hence, the sudden shifts in shear capacity as the column goes from compression to tension may require further investigation. On the other hand, the study concludes the amplification of negative moments in the midspan section as the primary issue that needs to be involved in the SDC-2006. In particular, the current requirement that vertical ground motions be considered only for sites where the expected peak rock acceleration is at least 0.6 g is considered not to be an adequate basis to assess the significance of vertical effects. According to Kunnath et al. (2008) a more detailed SDC criteria shall be created about the design specification for the consideration of vertical effects by means of a static load equivalent to 25% of the dead load applied in the upward direction.

Warn et al. (2008) studied vertical earthquake loads on seismic isolation systems in bridges. The study summarizes and presents sample results from earthquake simulation testing performed on a bridge model isolated with low damping rubber bearing and lead rubber bearings. Results from the testing program were used to investigate the influence of vertical excitation on the vertical load carried by the isolation system and the axial load of individual bearings. As a result of simulations, significant amplifications in the vertical response for both the low damping rubber bearing and lead rubber bearing bridge configurations were experimentally observed. However from a comparison of amplification factors for both the isolated and fixed-base configurations estimated using spectral analysis suggests the isolation system itself results in only a marginal increase in amplification over the fixed-base bridge for the model and systems considered in the study. Hence, those results suggest that the vertical flexibility of the bridge-isolation system should not be ignored for design. Use of the peak ground acceleration of the vertical component would underestimate the vertical earthquake load on the isolation system. They concluded that, the spectral analysis procedure considering the full vertical stiffness of the isolator lead to more reasonable and accurate estimates of the vertical earthquake load on the isolation system for the bridge model and isolation systems.

In this study, bridges with seismic isolation and without seismic isolation are considered. They can be classified as lightweight structures. If seismic isolation is applied, weight of the structure gains importance against vertical component of ground motions. Therefore it is considered seismically isolated bridges are needed to be investigated deeply. From this point of view, the study intends to investigate the influence of vertical earthquake excitation on the response of a briefly two types of bridges through seven earthquakes. First type of bridge is seismically isolated steel composite bridge. Second type is a regular steel composite bridge. Lead rubber bearings are used for seismic isolation. The intention of studying same bridge with seismic isolation and without seismic isolation is to be able to reach a comparison regarding influence of vertical component of earthquakes. Additionally, in order to reach more realistic and satisfactory comparisons between isolators on same bridge, a set composed of nine isolators which have different characteristic properties is used. To sum up considering many parameters stated herein several time history analyses are conducted using the structural analysis software SAP2000. As a result response data collected from the analyses is used to determine the vertical load on the piers and isolators and middle span moment on the steel girders due to the earthquake excitations. A comparison between all configurations is introduced and discussed.

2. Bridge Model and Isolators

In this study, a steel girder composite bridge is considered because it is in the group of beam type bridges. As a result, influences on bending of beam can be observed.

Also, composite steel girder bridge may be classified as relatively more lightweight than other type bridges.

The made up bridge which was investigated in an earlier study done by Eröz and DesRoches (2007), has been chosen. This is a regular steel girder bridge with RC piers and piles. The fundamental structural elements are derived from the previous study and developed through this study. After developing the bridge an initial design is performed in order to see whether the bridge is complying with the AASHTO provisions or not. For this reason, main girder and RC piers are checked against major loads applied to the bridge. For the sake of safety of initial design, AASHTO Strength I load combination limit state is used. Strength I load combination involves dead load and live load with load factors 1.25 and 1.75, respectively. After adequate number of iterations on sections, the structural elements of the bridge are finalized.

The number of span is 3 and the span length is 30.3 m. Fig. 1 and Fig. 2 show elevations of the bridge in the long and in the short axis, respectively. From top to bottom structural members and sectional properties are as follows; RC deck is 0.25 m thick. One of the most important members of the bridge is certainly steel girders. The steel girders are selected from I shaped sections with the geometric properties shown on Fig. 3. Diaphragm beam is composed of a rectangular section with a width and a height of 1.2 m. Circular RC pier has a diameter of 0.9 m and a height of 4.6 m. RC Mat foundation has plan dimensions of 3.6 m to 3.6 m with a thickness of 1.1 m. Finally, circular RC pile has a diameter of 0.45 m and a height of 4.5 m. Moreover, soil underneath the foundation is considered to be a common soil type of clayey medium dense sand with a subgrade reaction modulus of 35GPa/m.

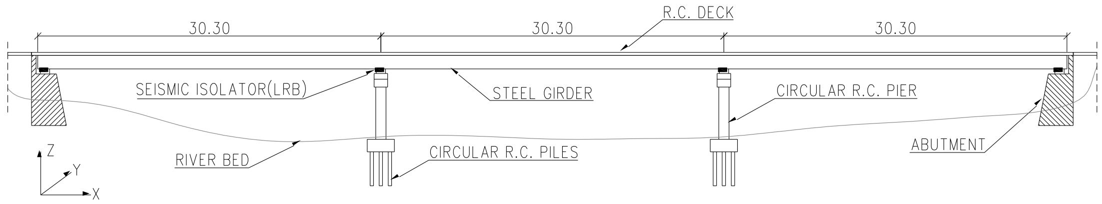

Fig. 1. Bridge elevation in long axis.

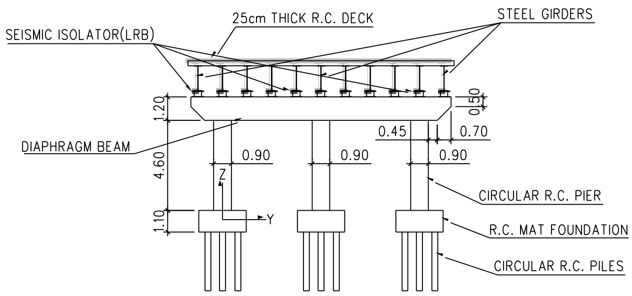

Fig. 2. Bridge elevation in short axis.

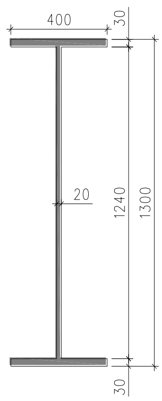

Fig. 3. I shaped steel girder's geometric properties.

In this study, isolators are modeled as nonlinear link elements. Hence, effective stiffness, damping, yield strength and post yield stiffness ratio are defined. By definition, yield strength is the force at yield of isolator. Post yield stiffness ratio stands for ratio of plastic stiffness to elastic stiffness. This ratio is usually taken as 0.1 for design purposes. The basic characteristic properties are chosen from a manufacturers catalog cut-sheet. Furthermore, other mentioned properties for nonlinear analyses are calculated to be used on the analyses program. 9 different commercially available isolators are investigated in the study. All isolators are chosen from LRB in order to investigate the response with damping. The overall diameters, rubber thicknesses and layers, lead core diameters and all mounting plate properties differ from one isolator to other. Hence, a wide range of isolators are used in the study. Table 1 shows the characteristic properties and calculated additional properties of the isolators which are used for modeling. On the table bold letters and numbers show the calculated properties and the rest of parameters are taken from the manufacturer's cut-sheet. Additionally, the labels of the commercially available isolators are show as ISO-XX where XX is suffix starting from 01 to 09. Therefore the difference that comes out in results may be distinguished easily.

3. Ground Motion Sets

Earthquakes (EQ) are generally grouped in accordance with three properties; peak ground accelerations (PGA) and peak ground velocities (PGV), soil classifications and possession of directivity. In this study, soil classifications and possession of directivity are considered as constant parameters and PGA and PGV vary to investigate response of the bridge under different conditions.

Soil classification is described as an important parameter for an EQ since soil properties directly have influences on propagation waves generated by ground motions. Hence, it is decided to choose EQs whose site classes are same, for a reasonable comparison different EQs.

In the study, the soil classification criteria of USGS is preferred since the EQs record source Peer Strong Motion Database directly gives soil classification as per USGS and it is chosen as EQs record source. Moreover, a soil class of "C" as per USGS classification is selected for all chosen EQs and stations.

Table 1. Characteristic properties of the selected isolators.

Isolator ID	Isolator Diameter D_I (mm)	Lead Diameter D_L (mm)	Post Yield Stiffness K_d (kN/mm)	Elastic Stiffness K_e (kN/mm)	Compression Stiffness K_v (kN/mm)	Effective Stiffness K_{eff} (kN/mm)	Characteristic Strength Q_d (kN)	Yield Strength F_y (kN)	Maximum Displacement u_{max} (mm)
ISO-01	355	0-100	0.20	2.00	100.00	0.85	65.00	72.22	150
ISO-02	455	0-125	0.30	3.00	100.00	0.96	110.00	122.22	250
ISO-03	570	0-180	0.50	5.00	500.00	1.25	180.00	200.00	360
ISO-04	800	0-230	0.70	7.00	1000.00	1.48	265.00	294.44	510
ISO-05	900	0-255	0.70	7.00	1400.00	1.65	355.00	394.44	560
ISO-06	1050	0-305	0.90	9.00	2100.00	2.13	580.00	644.44	710
ISO-07	1260	0-355	1.20	12.00	3700.00	2.60	755.00	838.89	810
ISO-08	1360	0-380	1.40	14.00	5100.00	2.95	890.00	988.89	860
ISO-09	1550	0-405	1.80	18.00	6500.00	3.49	1025.00	1138.89	910

EQs are classified as being near fault ground motions involving directivity. A dependable source is used to see the EQs classified as near fault ground motions involving directivity. The source is a previous study done by MacRae and Tagawa (2002) which investigates methods to estimate some specific structures using far-fault and near-fault directivity record sets. The near fault directivity record sets in MacRae and Tagawa (2002) are considered to be a dependable list of near fault ground motions involving directivity.

PGAs and PGVs are very important for grouping EQs. Three components (two horizontal and one vertical) of the EQs are used in the study. The selected EQs' lateral components and vertical components are in the range of 0.268 g to 0.897 g, and 0.242 g to 0.586 g, respectively, regarding PGA. On the other hand, their horizontal components and vertical components are in the range of 46,9 cm/s to 109,3 cm/s and 18,4 cm/s to 38,5 cm/s, respectively, regarding PGV. Seven EQs are considered in this study in order to provide adequate range of ground acceleration and ground velocity. They are sorted by the ratio of SRSS of horizontal peak accelerations to vertical peak acceleration. As a result, a normalized EQs property ratio is formed to be used while constituting comparisons of results. Table 2 presents the properties of the selected ground motions within the specified selection criterions.

4. Analysis Procedure

Three types of analysis cases are defined in the program. These are linear static, linear modal and nonlinear modal history (FNA) cases. Linear static type is used to define dead load and live load cases. Linear modal type is used to define Ritz-vector mode shape analysis case. Finally, the FNA type is used to define EQ load cases. Since FNA is a faster sort of time history analysis (THA), in order to be able to perform several runs this method is used. All cases started with a zero condition to discard effect of each analysis case to each other. For Ritz-vector case a tri-axial acceleration load is defined, since it is strictly required to define particular loads for this type of analysis. In addition to load definition, maximum number of modes is set as 17 to make sure 90% mass participation in each orthogonal direction. Two different FNA cases are created for each EQ. One of them is created to observe the influence of only horizontal components of EQs and named with suffix "…+H". Additionally, other one is created to observe the influence of both horizontal and vertical components of EQs and named with suffix "…+HV".

Modal damping is chosen as 5% constant damping for all of the FNA cases. Also, number of output time steps and output step sizes are chosen considering the EQ data time step and length. A total history time of 15 seconds is specified for each EQ.

In the analysis, combinations are defined in accordance with strength criteria stated in AASHTO Section 3. Each analysis case involves only effects due to its defined load or EQ. This is why combinations are defined. They are aimed to be used for combining effects due to different case. The combinations are grouped into two: Strength-I and Extreme Event-I. Strength-I has one sub-item including dead load (DL) and live load (LL). On the other hand, Extreme Event-I has 15 sub-items. 14 of them are generated from DL, LL and EQs and one is from DL and LL. Extreme Event-I is abbreviated to read Comb1 to Comb7. Also, type of combination for all of them is selected as linear add.

Table 2. Selected ground motion sets.

Earthquake	Location	Station	USGS Soil Class	Direction	A_p (g)	V_p (cm/s)	A_p / V_p (1/s)	$A_{p,HOR}$ (g)	$A_{p,HOR,max}$ / $A_{p,VERT}$	$A_{p,HOR}$ / $A_{p,VERT}$
Imperial (15/10/79)	California/ USA	El Centro Array #7	C	140 (UX)	0.338	47.617	6.963	0.57	0.851	1.054
				230 (UY)	0.463	109.261	4.157			
				Up (UZ)	0.544	26.310	20.284			
Imperial (15/10/79)	California/ USA	El Centro Array #5	C	140 (UX)	0.519	46.857	10.866	0.64	0.966	1.197
				230 (UY)	0.379	90.549	4.106			
				Up (UZ)	0.537	38.522	13.675			
Northridge (17/01/94)	California/ USA	Newhall	C	90 (UX)	0.583	74.878	7.638	0.83	1.077	1.514
				360 (UY)	0.590	96.879	5.974			
				Up (UZ)	0.548	31.532	17.049			
Düzce (12/11/99)	Turkey	Düzce	C	180 (UX)	0.348	59.990	5.691	0.64	1.499	1.788
				270 (UY)	0.535	83.506	6.285			
				Up (UZ)	0.357	22.605	15.493			
Kocaeli (17/08/99)	Turkey	Yarımca	C	60 (UX)	0.268	65.740	3.999	0.44	1.442	1.818
				330 (UY)	0.349	62.177	5.506			
				Up (UZ)	0.242	30.814	7.704			
Northridge (17/01/94)	California/ USA	Sylmar	C	52 (UX)	0.612	117.432	5.113	1.09	1.531	1.853
				142 (UY)	0.897	102.208	8.609			
				Up (UZ)	0.586	34.587	16.621			
Erzincan (13/03/92)	Turkey	Erzincan	C	EW (UX)	0.496	64.282	7.569	0.72	2.077	2.883
				NS (UY)	0.515	83.959	6.017			
				Up (UZ)	0.248	18.373	13.242			

5. Comparison of Results

5.1. Modal analysis

Initially, modal analysis outputs are given such as fundamental periods of each orthogonal direction for all mathematical models. Table 3 presents the fundamental periods of the models with predefined isolator and connection property. All modal outputs are calculated using Ritz-Vector analysis.

Table 3. Fundamental periods of the bridge models.

Model	Fundamental Period (s) UX	UY	UZ
SCB1	1.046	0.937	0.285
SCB2	1.000	0.889	0.284
SCB3	0.911	0.794	0.276
SCB4	0.860	0.740	0.275
SCB5	0.829	0.708	0.275
SCB6	0.762	0.642	0.275
SCB7	0.715	0.597	0.274
SCB8	0.687	0.572	0.274
SCB9	0.651	0.541	0.274
SCB-Fix	0.080	0.210	0.253

5.2. Steel girder midspan moment

Nine steel girder bridge models are generated using nine different commercially available lead rubber bearings. Outputs from these models are normalized with each other to provide a complete investigation about influences of LRBs on girder midspan moment.

Fig. 4 illustrates the H/HV ratios of girder midspan moments with respect to K_{eff}/K_v for seven different EQ data and average of them. Fig. 5 shows that girder midspan moment of the bridge systems does not change with K_{eff}/K_v ratios for the same EQ data. If one use the average result of the seven EQ data for design purposes, the value of H/HV will be 0.72. This means including vertical ground motion in the analysis increases girder midspan moment by 40%.

Fig. 5 illustrates the H/HV ratios of girder midspan moments with respect to $A_{p,HOR}/A_{p,VERT}$ for nine different rubber isolators. It is obvious in this figure that all isolators H/HV curves' fitting on each other which reveals isolator property doesn't have a significant effect on the girder midspan moments in steel girder bridges.

5.3. Bridge systems with and without seismic isolation

The model of the composite steel girder bridge with seismic isolation is reconstructed as a steel girder bridge without seismic isolation. Basically, the reconstruction includes replacement of the isolators at the connections of diaphragm beams and steel girders with rigid links. On the issue of replacement, main objective is enabling axial force transfer from girders to piers or vice versa.

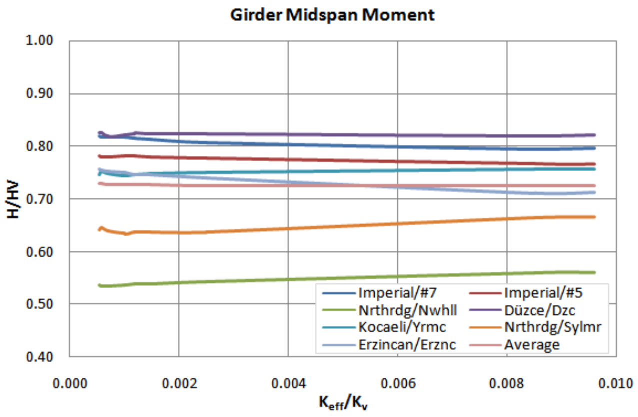

Fig. 4. Girder midspan moment ratios of the bridges with respect to K_{eff}/K_v.

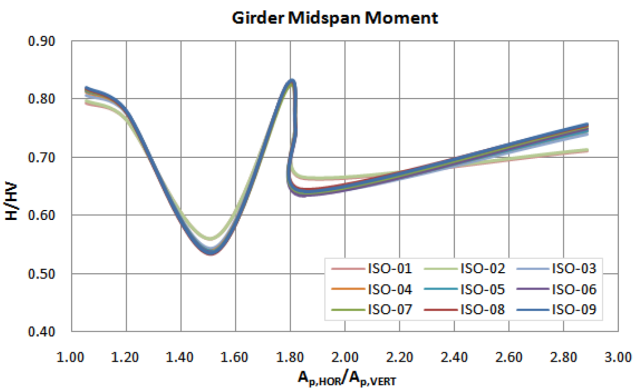

Fig. 5. Girder midspan moment ratios of the bridges with respect to $A_{p,HOR}/A_{p,VERT}$.

Fig. 6 and Fig. 7 present the *H*/*HV* ratios of girder midspan moments and pier axial loads with respect to $A_{p,HOR}/A_{p,VERT}$, respectively. On both figures, *H*/*HV* ratios of the bridge with seismic isolation show the mean member force ratios from the bridge models with nine different isolators. On the other hand *H*/*HV* ratios of the bridge without seismic isolation show output from only one bridge.

Fig. 6 shows that the *H*/*HV* ratios of the bridge with seismic isolation are slightly higher than the H/HV ratios of the bridge without seismic isolation. Since, the results calculated by horizontal EQ components (H) in the bridge with seismic isolation is close to the same sort of results in the bridge with seismic isolation, the difference between the *H*/*HV* ratios depends on the change of HV value (results calculated with the inclusion of vertical EQ components). Therefore, use of isolation system on a bridge causes significant increase in girder midspan moment in most of the load cases. The rate of increase is approximately 0.9%.

Fig. 7 shows that *H*/*HV* ratios of the bridge with seismic isolation are smaller than the *H*/*HV* ratios of the bridge without seismic isolation. The discussion in the paragraph above regarding the effects of horizontal and vertical EQ components on H and HV is also acceptable for the pier axial force. Hence, use of isolation system on a bridge causes significant decrease in pier axial force in most of the load cases, although the girder midspan moment increases. The decrease in the pier axial force can be as high as 11%.

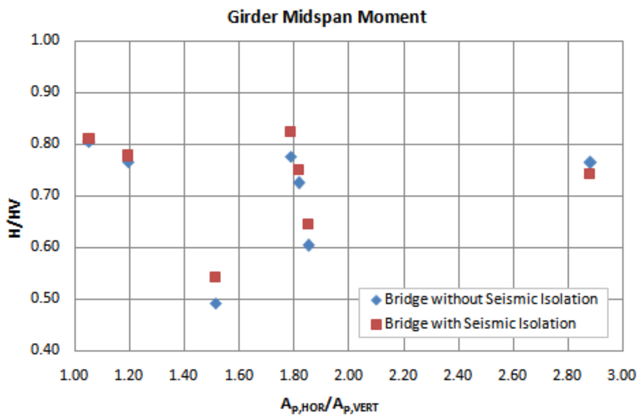

Fig. 6. Girder midspan moment ratios of the bridges with and without seismic isolation.

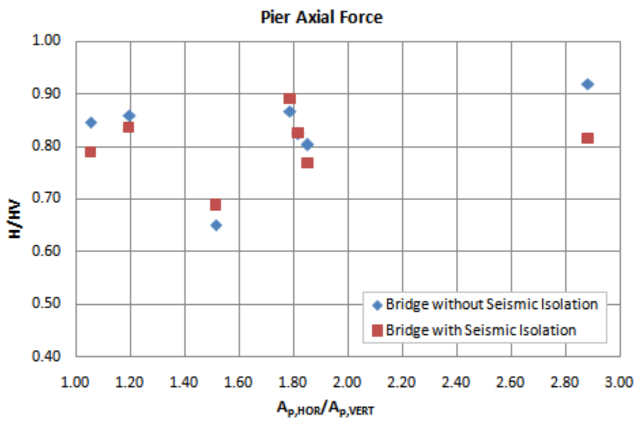

Fig. 7. Pier axial force ratios of the bridges with and without seismic isolation.

6. Conclusions

The main objective of this study was to investigate the influences of vertical ground motion on the bridges with and without seismic isolation. After a brief introduction on seismic isolation and devices, more specific information on seismic isolation on bridges was presented. Besides this, all required information on performing this numerical study was introduced one by one. The study was carried out using a steel girder composite bridge and nine LRBs with different characteristic properties. Nine mathematical bridge models with and without seismic isolation were built. Moreover, seven different EQ data in same site class were selected. Two horizontal and one vertical components of the EQs were used in the analysis of bridges using FNA in the SAP2000 software. Finally, numerical results were obtained and introduced. The member forces presented for comparison are girder midspan moment, pier axial force and isolator axial force.

This study reveals that;

- The bridges with seismic isolation are not as vulnerable as the bridges without seismic isolation according to the fundamental periods on the response spectrums regarding the vertical component of the EQs.
- The vertical components of the EQs are not producing resonance in seismically isolated bridges for the ground motion data chosen in this study.
- Vertical ground motion affects the girder midspan moment significantly.
- Disregarding the vertical ground motion from the analysis leads to mean underestimation of 27% of girder midspan moment.
- The bridge systems with different isolators have the same H/HV ratio of the girder midspan moment for same combination. Also, varying K_{eff}/K_v ratio doesn't have a significant effect on the girder midspan moments. Hence, isolator property doesn't have a significant effect in steel girder bridge analyzed in this study.
- Use of isolation system on a bridge causes significant increase in girder midspan moment in most of the load cases. The rate of increase fluctuates with the $A_{p,HOR}/A_{p,VERT}$ ratio in a range of 0.8% to 9.5%.

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