Istanbul Bridge Conference
August 11-13, 2014
Istanbul, Turkey

EFFECT OF VERTICAL MOTIONS ON SEGMENTAL BRIDGES UNDER CONSTRUCTION

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ABSTRACT

Results of non-linear analyses conducted on different segmental bridges under construction showed that the vertical ground motion significantly affects: (a) the axial force demand in piers; (b) the moment and shear demands at the face of the bent cap; (c) the moment demands at the mid-span. It is remarkable than the records with the minor v/h ratios of the set of earthquakes used in the non linear analyses caused the maximum damage indexes and the highest curvature ductility demands in the piers. Then, it can be concluded that the v/h ratio is not the only parameter for determining the potential destructivity of the vertical seismic component and the proximity between the vertical period of the structure and the dominant frequency of the vertical excitation could be more important. Results obtained for the completed bridges suggest that the Caltrans recommendation of accounting for the vertical effect by means of an equivalent vertical load with a magnitude of 25% of the dead load of the structure applied separately in the upward and downward directions should be revised. Including vertical accelerations in the analyses is recommended for reliable seismic assessment of bridges in the vicinity of active faults, where the vertical excitation is likely to be high.

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Jara M, Alvarez O, Jara JM, Olmos BA.Effect of vertical motions on segmental bridges under construction. *Proceedings of the Istanbul Bridge Conference*, 2014.

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ABSTRACT

Results of non-linear analyses conducted on different segmental bridges under construction showed that the vertical ground motion significantly affects: (a) the axial force demand in piers; (b) the moment and shear demands at the face of the bent cap; (c) the moment demands at the mid-span. It is remarkable than the records with the minor V/H ratios of the set of earthquakes used in the non linear analyses caused the maximum damage indexes and the highest curvature ductility demands in the piers. Then, it can be concluded that the V/H ratio is not the only parameter for determining the potential destructivity of the vertical seismic component and the proximity between the vertical period of the structure and the dominant frequency of the vertical excitation could be more important. Results obtained for the completed bridges suggest that the Caltrans recommendation of accounting for the vertical effect by means of an equivalent vertical load with a magnitude of 25% of the dead load of the structure applied separately in the upward and downward directions should be revised. Including vertical accelerations in the analyses is recommended for reliable seismic assessment of bridges in the vicinity of active faults, where the vertical excitation is likely to be high.

Introduction

Previous studies have revealed that vertical component of ground motions is especially significant for high frequency content and sites located close to the epicenter, where the vertical acceleration can be even greater than the horizontal components. Based on the vertical to horizontal peak ground acceleration ratio of 33 time histories recorded in the United States, Newmark et al. [1], suggested an engineering rule of thumb of assuming the vertical to horizontal ratio (V/H) intensity of 2/3 for design purposes. Latter, Collier and Elnashai [2], among others, observed that this rule is conservative for epicenter distances greater than 60 km and un-conservative for near fault records. Moreover, the 2/3 scaling lead to the same frequency content for all components; however, Rosenblueth [3], since 1975, had shown that vertical effect is period dependent. Studies related to the response of highway bridges to near fault vertical accelerations have been carried out by several researchers (Kunnath [4], and others). Results of these analyses lead to the conclusion that vertical component effects can have a significant variation in axial force demands in bridge piers. These variations produce changes to moment and shear capacity of the

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elements, and fluctuations in moments at mid-span of the superstructure and at the face of the bent cap. Other authors have also attributed some of the observed failures in bridges to the vertical component.

In spite of that, currently bridge design specifications do not include any provisions to construct vertical design spectrum. Seismic Design Criteria-2006 of the California Department of Transportation (CALTRANS) [5], requires consideration of the vertical effects but does not require analysis of the structure under combined horizontal and vertical components of ground motion. Instead, it stipulates the check of the structure under an equivalent vertical load with a magnitude of 25% of the dead load of the structure applied separately in the upward and downward directions to account for vertical accelerations. Commentary of the American Association of Standards and Highway Transportation Officials (AASHTO) Bridge Specifications [6], specify a vertical design spectrum having ordinates of 2/3 of the horizontal design spectrum.

Most of the studies of the vertical component effect on bridges have concentrated on the fluctuation of the axial load on piers of completed bridges. Nevertheless, during the staged process of construction of medium span bridges, the resistance, stiffness and stability of the structure against vertical accelerations could be seriously reduced. This study was undertaken with the goal of assessing the response of bridges during their construction process, as well as for completed bridges.

Vertical component characteristics for subduction earthquakes

One hundred records from the Mexican Strong Motion Database, were selected for the study. The acceleration time histories were recorded at distances less than 60 km and for magnitudes larger than 5.0. Seventy five of the records correspond to shallow inter-plate earthquakes generated at the Mexican Pacific coast, and the remainder twenty five records had their epicenters at intermediate depths, and correspond to normal faulting, deeper in-slab earthquakes. The V/H ratios of the ensemble of records are greater for shallow earthquakes than for intermediate depth earthquakes. The V/H ratio increases gradually with increasing earthquake magnitude and decreases with increasing fault distance and focus depth. It is remarkable the high V/H ratios obtained for distances greater than 40 km; most of the V/H ratios are greater than 2/3 for distances between 40 and 60 km. As it was obtained previously by other researchers, the vertical component has much higher frequency content than that of the horizontal component. This can be attributed to the fact than the wavelength of P-waves is shorter than that of S-waves.

Construction methods and description of bridges

Incrementally launched bridges

The method consists of constructing the superstructure of a bridge by segments in a prefabrication area behind one of the abutments; each new unit is concreted directly against the preceding one and after it has hardened the resultant structure is moved longitudinally into its final position (Fig. 1 left).

Figure 1. Bridges constructed by the incremental launching method (left) and the balanced cantilever method (right).

Balanced cantilever method

Construction begins from the top of each bridge pier, with the segment normally fixed to the pier either permanently or temporarily (Fig. 1 right). Subsequent segments are post-tensioned to the previous sections on alternate sides of the pier so that the out-of-balance moment is kept to a minimum. Segment construction is continued until a joining midpoint is reached where a balanced pair is closed.

Bridge description

Two bridges constructed by the incremental launching method were selected for assessing the bridge response during different construction phases and for identifying the structural elements that are more affected by the vertical acceleration. The first bridge (Fig. 2) is a 203 m x 10 m, five span structure with end spans of 34 m and three intermediate spans of 45 m. The superstructure is supported on two end abutments, two exterior piers and two central piers. The heights of the two central piers (piers 2 and 3) are 45 m and the two exterior piers (piers 1 and 4) are 20 m high. The piers have a hollow circular shape section with diameter of 5 m and thickness of 0.75 m and diameter of 3 m and thickness of 0.5 m for the tall and short piers respectively. The superstructure consists of a 3 m deep single cell concrete box girder. The launching nose is 27 m long that covers the 60% of the main spans length.

Figure 2. Five span bridge constructed by the incremental launching method.

The second bridge (Fig. 3) is a 3 span continuous superstructure with high horizontal curvature and a steel composite and orthotropic box girder. The superstructure is supported on two intermediate piers and two end abutments. The interior span is 180 m long and the end spans 71.5 m for a total length of 323 m. The superstructure was launched by the two sides, in 26 segments 12 m long and two additional final segments at midspan of 9 m. The piers are hollow rectangular shape sections 8 x 3 m, with thickness of 1 and 0.8 m in the long and short dimension of the pier respectively. The piers height is 86 m and 95 m. The bridge has a longitudinal slope of 5% and transversal slope of 10%.

A third bridge, constructed by the balanced cantilever method, was selected. This is a three span bridge 349 m long (Fig. 4), with end spans of 94.5 m and a main span of 160m. The superstructure is a single cell reinforced concrete box girder with a variable depth that ranges from 3 m at mid-span to 9 m at the pier. The girder was formed with 56 segments, 4.6 m long, 4 segments, 6 m long and a closure segment of 4 m. The piers are hollow rectangular shape sections 6 x 6 m with thickness of 0.8 m. The columns height is 58 m.

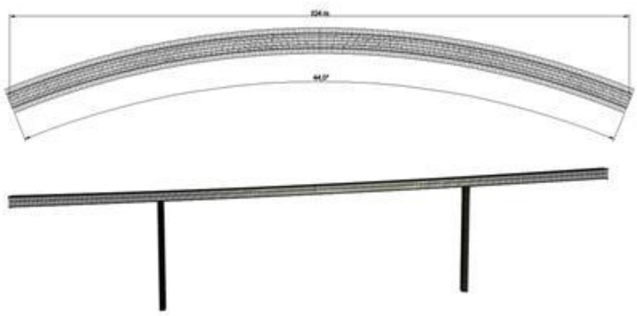

Figure 3. Three span curved bridge constructed by the incremental launching method.

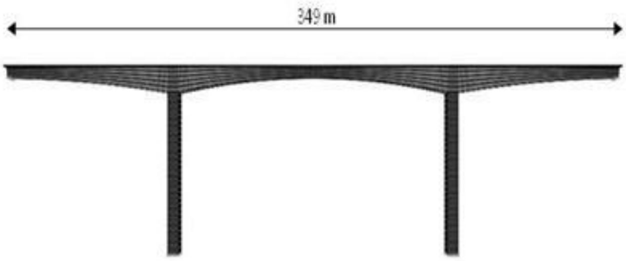

Figure 4. Three span balanced cantilever bridge.

Response of the structures during the construction process

Five different construction stages of each bridge were selected for evaluating the critical conditions of the demands placed on key elements under the action of the vertical component. The five structures of each bridge were subjected to the 1996 Victoria's record that has a V/H ratio of 1.1. Three different load cases were considered in the analyses: dead load alone (DL); two horizontal components acting simultaneously (2C); and the three components acting together. Initially, the (3C – 2C) / DL ratio was used for assessing the impact of the vertical excitation, however, in some cases the ratios were very large, and gave a distorted view of the impact of the vertical component because the absolute dead load and two component horizontal response values were very small. Then, the ratio (DL + 3C) / (DL + 2C) was evaluated for determining the impact of the vertical excitation.

Five span incrementally launched bridge

Fig. 5 shows two different construction stages considered for the first bridge, and Table 1 shows the greatest (DL + 3C) / (DL + 2C) ratios for moment and shear demands on the superstructure. The first row represents the maximum ratio for the flexural moment at mid-span, the second row corresponds to the maximum moment over the bent cap and the third row is for the shear at the end of the span. The maximum impact of the vertical acceleration resulted 39% greater than the mid-span moment produced by the load combination DL + 2C. The shear at the end of the superstructure experienced an increment of 38% of the effect caused by the DL + 2C load combination. The maximum vertical impact for moment at mid-span and shear force on the superstructure occurred when the bridge was completed (stage 5). Results presented in Table 1 suggest that the CALTRANS recommendation of accounting for the vertical impact by means of an equivalent vertical load with a magnitude of 25% of the dead load of the structure applied separately in the upward and downward directions should be revised. The ratio for the axial force demand on piers is shown in Table 2. The vertical excitation effect for the central piers was about 40% of the total force (dead load + two horizontal components), while the ratio for the exterior piers were 13% and 4% of the total force demand. The ratio obtained for other forces or moments was very close to 1.0.

Figure 5. Construction stages 2 and 3 of the first bridge.

Table 1. Moment and shear demands on the superstructure of the three bridges.

Moment or shear	5-span bridge		Curved bridge		Balanced cantiliver	
	Stage	Ratio	Stage	Ratio	Stage	Ratio
$M_{máx}$ (+)	5	1.39	4	1.15	4	1.40
$M_{máx}$ (-)	2	1.12	2	1.22	2	1.34
$V_{máx}$	5	1.38	2	1.28	2	1.34

Three span curved incrementally launched bridge

Table 1 presents the greatest (DL + 3C) / (DL + 2C) ratios for the superstructure. The maximum impact of the vertical acceleration resulted 22% greater than the end moment produced by the DL + 2C load combination. The shear at the end of the superstructure experienced an increment of 28% of the effect caused by the DL + 2C load combination. The ratio for the axial force demand on piers is shown in Table 2. The highest ratio for the 86 m pier was about 22% greater than of the DL + 2C load combination. As in the first bridge, the vertical component influence for other forces or moments in piers was negligible. The moment demand at the base of the piers resulted 39% and 54% greater than the dead load moment.

Table 2. Axial force ratio demands for piers of the three bridges.

Pier	5-span bridge		Curved bridge		Balanced cantiliver	
	Stage	**Ratio**	**Stage**	**Ratio**	Stage	**Ratio**
1	4	1.13	2	1.22	5	1.54
2	3	1.41	5	1.07		
3	3	1.38				
4	4	1.04				

Three span balanced cantilever bridge

Fig. 6 illustrates two different construction stages considered for the analyses of the third bridge. Table 1 presents the greatest (DL + 3C) / (DL + 2C) ratios for the superstructure. The highest impact of the vertical acceleration was an increment of 40% of the mid-span moment produced by the DL + 2C load combination. The shear at the end of the superstructure experienced an increment of 34% of the effect caused by the DL + 2C load combination. These values are slightly greater than the ratios obtained for the other two bridges. The influence index for the axial force on piers is shown in Table 2. The highest ratio for the piers was 54% of the DL + 2C combination load and was obtained for the completed bridge. The value was higher than the ratios obtained for the incrementally launched bridges. The index obtained for other force or moment demands in piers was very close to 1.0. The moment demand at the base of the piers was significantly greater than the dead load moment.

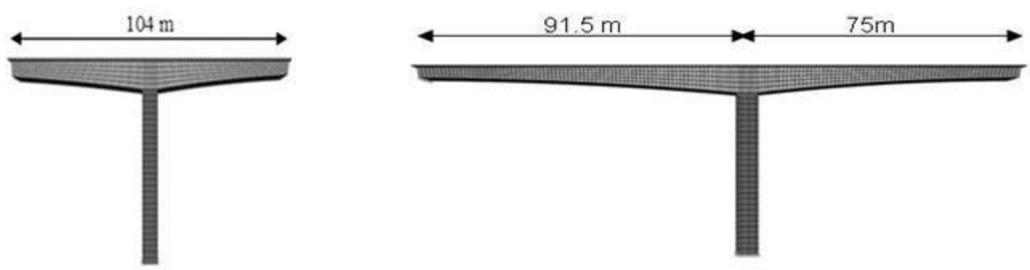

Figure 6. Construction stages 2 and 4 of the third bridge.

Nonlinear analyses of the critical construction stages

After the analyses of the different structures generated during the bridge construction, the critical stages of each bridge were selected for the nonlinear analyses. The response of the completed structure of the three bridges was also assessed. Ten sets of three component acceleration time histories, representing rock and soil conditions, at distances up to 60 km and for magnitudes greater than 5.0, were selected from the one hundred records of the data base. The ensemble of records was selected on the basis of the maximum vertical to horizontal peak acceleration ratio. Table 3 presents the seismic records characteristics and its V/H ratio. It can be observed that V/H ranges from 0.84 to 2.53. All the records were scaled to 0.48 of the spectral acceleration corresponding to the seismic design spectrum coefficient for the vertical period of the bridge. The factor scale was obtained with the expressions given by the Euro-code, Part 2 [7], for an exceedance rate p= 0.05 and a return period T_R = 60 years. The acceleration time histories applied to the three completed bridges were matched to the design spectrum for the site, that is, the spectrum for a return period of 475 years. The nonlinear analyses were carried out with the Perform 3D software. The material and geometrical nonlinearities were considered in all analyses.

Table 3. Earthquake records for the nonlinear analyses of the critical structures.

Num.	Station	M	R (km)	Soil (*)	Depth (m)	PGAV/PGAH
1	IAGS	6.6	2.8	F	10	2.53
2	VICS	6.1	9.4	F	12	1.04
3	ARTG	5.1	28.5	R	39	0.89
4	BALC	5	57.6	R	19	1.13
5	ACAC	6.5	55.2	F	19	0.94
6	XALT	5.1	55.3	R	13	1.69
7	XALT	5	31.9	R	12	1.38
8	PTSU	5.9	46.5	R	82	0.84
9	UNIO	5.1	13.9	R	82	1.08
10	VILD	6	49.9	R	54	0.88

(*) F = flexible; R = rock

Results for the five span incrementally launched bridge

Fig. 7 displays the moment-curvature relationships for the two types of piers of the five-span incrementally launched bridge. Two different curves are shown for each type of column: the curves with higher ductility were obtained considering the axial force produced by the deal load condition only, whereas the curves with lower ductility represent the piers with the axial force including the vertical effect.

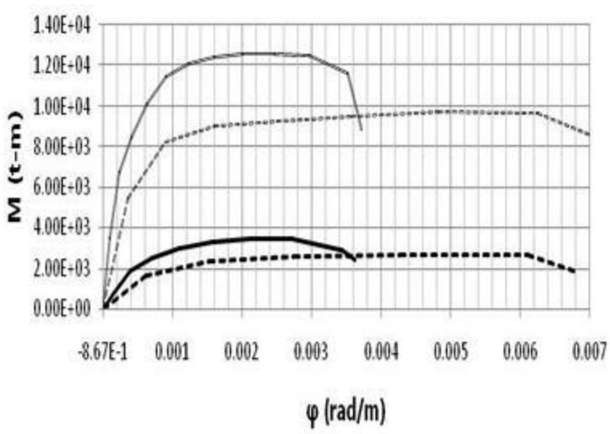

Figure 7. Moment curvature relationships for the two type of piers of the first bridge.

During the construction process, the maximum increment of the axial force in the short piers (33%) occurred when the PTSU record, identified in Table 3 as record number 8, was applied to the structure, whereas the most important increment for the tall piers was obtained with the BALC record (number 4 in Table 3), and was 28% greater than the axial force without the vertical component effect. It is remarkably that the highest impact of the vertical component was produced by the record with the lowest V/H ratio of the group of ten earthquakes.

Fig. 8 illustrates the effect of vertical excitation in the axial force response for the highest piers of the five-span bridge after it was completed. The figure at the right is the axial force variation when the dead load and the two horizontal components were acting on the bridge. The figure at the left is the axial load variation when the vertical component was included in the analysis. It can be observed that the vertical motion induced important increments of the axial load in the piers which lead to reduction of the ductility capacity of the element. The reduction of axial force due to the fluctuating vertical acceleration is close to zero in the time history shown in Fig. 8. If changes of the axial force sign had been taken place the diagonal shear capacity would have reduced significantly. The flexural moment demand at the base of the piers was practically the same if the vertical accelerations were considered. Fig. 9 displays the hysteretic cycles at the base of the short piers for the structure corresponding to the third construction stage. The figure at the left corresponds to the analyses with the two horizontal components and the dead load acting simultaneously. The figure at the right is for the same pier when the vertical component was included. The curvature ductility demand when the vertical acceleration was added is twice the curvature ductility demand if only the horizontal components are considered. In all the structures generated during the construction process, the curvature demands on the short piers were greater than the curvatures demands on the tall piers. Records 5, 8 and 10 of Table 3 caused the maximum curvature demands on the piers, even though these records have the minor V/H ratios of the ensemble (0.94, 0.84 and 0.88 respectively). The proximity between the vertical period of the structure and the dominant frequency of the vertical excitation can explain these results. The results confirmed that the V/H ratio is not the only parameter for determining the potential destructivity of the vertical seismic component.

Curvature ductility demands and damage estimation

The five-span incrementally launched bridge experienced inelastic behavior when it was subjected to three of the ten records reported in Table 3. The maximum curvature ductility demand for the central piers was (μ_φ)máx = 1.46 and for the short piers (μ_φ)máx = 1.44, for the structure generated during the third stage of construction. These results were obtained when only the dead load and the two horizontal components were applied simultaneously. The ductility demands obtained in both type of piers, would produce only minor cracks. If the vertical component was included in the analyses, the maximum curvature ductility is greater ((μ_φ)máx = 2.57) than that of the horizontal component analyses, but minor damage level was also produced. In the case of the completed bridge the ductility demands were of (μ_φ)máx = 3.19 in the short piers and $(\mu\varphi)$máx = 3.28 in the central piers. If only the horizontal components were included in the analyses of the completed bridge, the ductility demands obtained for all piers resulted more than 50% greater than those of the bridge model under construction; whereas, If the vertical acceleration was included, the ductility demand in all piers was more than 100% of the ductility demands in the bridge under construction.

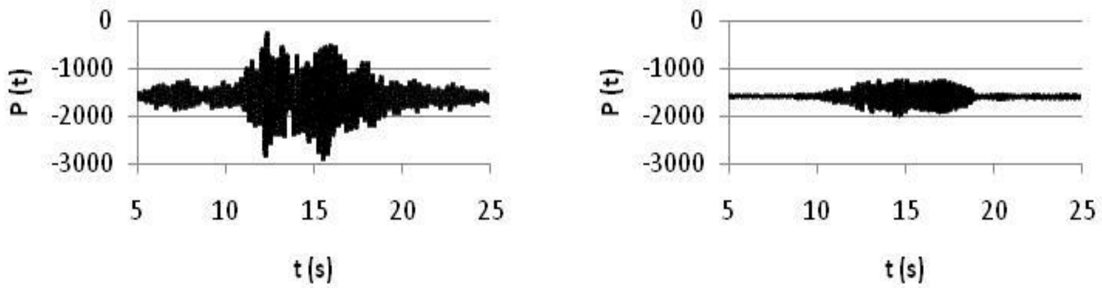

Figure 8. Axial force time histories for the central piers of the five-span bridge, with (left) and without (right) vertical accelerations included.

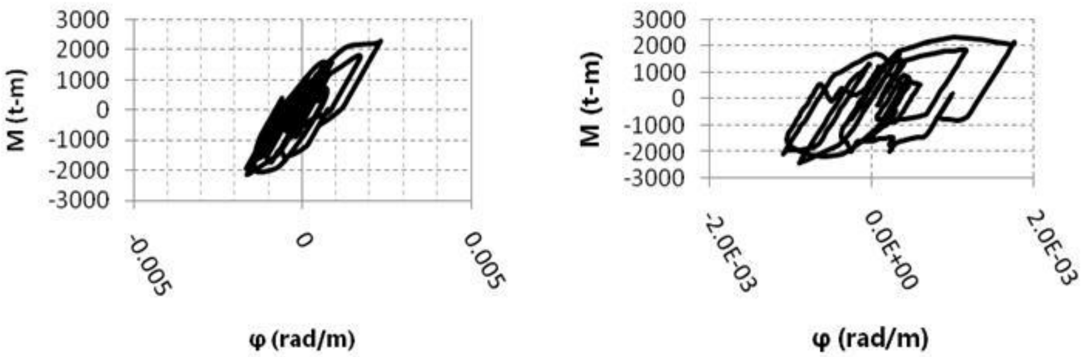

Figure 9. Hysteretic cycles for the central pier of the five-span bridge with (left), and without (right) the vertical accelerations included.

The well known Park and Ang [8] damage index was determined for each pier of the structures under construction as well as for the piers of the completed bridge. The action of the two horizontal components in the first bridge during the construction stages, produced a maximum damage index of 0.11 for the central piers and 0.18 for the exterior piers. These values correspond to minor damage, represented by minor cracks in the element. If the vertical component is included, the damage index for the short piers is now of De = 0.87 and for the central piers De = 0.43. The damage index obtained for the short piers, corresponds to severe damage, including exposure of the steel reinforcement and damage of the concrete core of the element. The damage index for the tall piers was in the limit between moderate and severe damage. When the vertical action is not included the damage index reflects only minor damage in both short and tall piers. A comparison of the ductility demands and the damage indexes obtained confirm that the ductility alone is not an appropriate predictor of damage level.

Conclusions

A comprehensive study of the characteristics of the vertical seismic component for shallow inter-plate and normal faulting in-slab earthquakes generated at the Mexican Pacific Coast was developed. The V/H ratios of the seismic records are greater for shallow earthquakes, less than 40 km in depth, than for intermediate depth earthquakes. It is remarkable the high V/H ratios obtained for distances greater than 40 km; most of the ratios are greater than 2/3 for distances between 40 and 60 km.

Results of the analyses of the bridges under construction showed that the vertical ground motion significantly affects: (a) the axial force demand in piers; (b) the moment and shear demands at the face of the bent cap; (c) the moment demands at the mid-span. Severe damage was estimated for the five-span incrementally launched bridge during construction as well as for the completed bridge. The incrementally launched three-span curved bridge experienced moderate damage during the construction process. The balanced cantilever bridge suffers only minor damage during the construction process, and behaved linearly when the complete structure is considered. Results obtained for the completed bridges suggest that the Caltrans recommendation of accounting for the vertical effect by means of an equivalent vertical load with a magnitude of 25% of the dead load of the structure applied separately in the upward and downward directions should be revised.

The records with the minor V/H ratios of the set of earthquakes used in the non linear analyses caused the maximum damage indexes and the highest curvature ductility demands in the piers. Then, it can be concluded that the V/H ratio is not the only parameter for determining the potential destructivity of the vertical seismic component. The proximity between the vertical period of the structure and the dominant frequency of the vertical excitation could be more important.

Including vertical accelerations in the analyses is recommended for reliable seismic assessment of bridges in the vicinity of active faults, where the vertical excitation is likely to be high. Vertical design spectrum should be included in design codes, especially if there are not particular challenges impeding its inclusion. The use of vertical spectrum equal to 2/3 of the corresponding horizontal spectrum is not recommended.

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