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Parameters influencing seismic response of horizontally curved, steel, I-girder bridges

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Abstract. This study examines the influence of curved, steel, I-girder bridge configuration on girder end reactions and cross frame member forces during seismic events. Simply-supported bridge finite element models were created and examined under seismic events mimicking what could be experienced in AASHTO Seismic Zone 2. Bridges were analyzed using practical ranges of: radius of curvature; girder and cross frame spacings; and lateral bracing configuration. Results from the study indicated that: (1) radius of curvature had the greatest influence on seismic response; (2) interior (lowest radius) girder reactions were heavily influenced by parameter variations and, in certain instances, uplift at their bearings could be a concern; (3) vertical excitation more heavily influenced bearing and cross frame seismic response; and (4) lateral bracing helped reduce seismic effects but using bracing along the entire span did not provide additional benefit over placing bracing only in bays adjacent to the supports.

Keywords: Horizontally; curved, steel; I-girder; bridge; seismic; finite; element.

1. Introduction

Horizontally curved bridges have become increasingly common over the last 40 years and currently occupy an appreciable share of bridge construction throughout the world. Irrespective of the materials used for the superstructure, curved bridges can offer economic advantages when compared to structures composed of a series of straight chords. These include the utilization of continuous spans and the subsequent reduction in the number of substructure units and expansion joints along with the elimination of complicated deck formwork construction at chord intersection points. Curved steel structures can offer additional benefits, including: the ability to handle complicated plan geometries and tight radii; reduced section depths when compared to other materials; and efficient construction times. In contrast to the above advantages, the presence of initial curvature in the geometry of these types of structures causes interaction of flexural and torsional behavior that complicates the analysis and design processes and must be considered for both static and dynamic loads.

From a dynamic and, more specifically, seismic loading perspective, rotations and displacements of the superstructure resulting from flexural-torsional interaction have been reported as causing damage in steel, I-girder bridges that included curved structures. The reported damage included cross frame member

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failures, associated connection failures, and bearing failures (Julian *et al*. 2007, Itani *et al*. 2004). The combination of the increased construction of curved, steel, I-girder bridges throughout the world, the lack of research related to their dynamic response and the reported steel bridge failures provided motivation to study their seismic behavior.

2. Background

As was stated earlier, the amount of research related to curved, steel, I-girder bridge dynamic response is limited. Studies of dynamic behavior largely encompass free vibration examinations and study of response under moving loads. Free vibration studies were performed by Yonezawa (1962), Culver (1967), Ramakrishnan and Kunukkasseril (1976) and recently by Maneetes and Linzell (2003), Yoon and Kang (1998) and Yoon *et al*. (2005). The dynamic response of curved steel bridges under moving loads was examined by Tan and Shore (1968), Christians (1968), Culver (1967), Oestel (1968), Das (1971), Chaudhari (1975), Chaudhari and Shore (1975, 1977), Huang *et al*. (1995) and McElwain and Laman (2000). Prominent differences between these studies consisted of methods used to represent individual bridge components in the computational models, methods used to idealize the vehicle(s) and the level of complexity of the dynamic analyses that were performed.

In addition to the aforementioned dynamic studies, limited research related to curved, I-girder, bridge seismic response has been completed. Chang *et al*. (1985) completed an analytical study of curved steel bridge seismic response using a Rayleigh-Ritz representation of the structure that included flexural, axial and torsional effects. The study calculated natural frequencies and mode shapes and compared results to those obtained from using matrix methods for single, two, three and four span bridges supported on single pier bents. Chen (1995) compared the effects of modeling and analysis methods on the effectiveness of predicting the response of curved bridges under earthquake loadings. The modeling methods were primarily based on 2-dimensional and 3-dimensional representations of structures and included soil-structure interaction. The analysis methods included single mode, multi mode, and time history analyses as defined in the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* (1994). As a result of the study, multi mode analysis using simplified models consisting of 2-dimensional elements representing the superstructure and spring elements representing soil-structure interaction were recommended. Additionally, the inclusion of soil-structure interaction in the models was recommended for longer bridges. Although these studies looked at curved bridge seismic response, they did not include the effects of varying levels of horizontal curvature. The effect of curvature on the out-of-plane and torsional moments was studied by Al-Baijat (1999). Analytical bridge configurations were developed in SAP 90 with varying angles of curvature and cross frame spacing. The study found that the fundamental period increased with an increase in curvature and that torsional moment magnitudes increased with curvature angle. Among the recent seismic curved bridge studies, one focused on developing performance criteria by examining the response of a single structure with different modeling techniques (Mwafy and Elnashai 2007) and another focused on evaluating the effectiveness of cable restrainers for preventing deck unseating of a curved steel viaduct (Julian *et al*. 2007).

This brief summary of past studies related to curved, steel, I-girder bridge dynamics demonstrates that a need exists to investigate the effect of structure configuration on their seismic response continues to exist. The current study attempted to address this need by examining the influence that structural configuration, including radius of curvature, girder and cross frame spacing, and lateral bracing configuration, has on support reactions and cross frame member axial forces generated in curved, steel, I-girder bridges during seismic events.

3. Methodology

Important modeling parameters and their ranges for the current study were established utilizing information from Kim (2004) and Maneetes and Linzell (2003). Kim (2004) examined the influence of various parameters on curved, steel, bridge live load distribution, with parameter ranges being established from applicable design criteria. The study was based on simply supported, single span, curved steel I-girder bridges with typical plan and cross sectional views as shown in Figure 1 and Figure 2. None of the bridges that were examined had skewed supports and they initially had no lateral bracing. Abutments are designated as "left" and right" as shown in the framing plan. Figure 2 indicates that all girders were assumed to be doubly symmetric, deck superelevation was ignored and parapets were not included. Parameter ranges examined by Kim were adopted for the present study and Table 1 presents key parameters for the 27 designs that were examined.

Similar to the free vibration work by Maneetes and Linzell (2003), the present study included lateral bracing as a parameter of interest. Two arrangements of lateral bracing, in addition to the original case

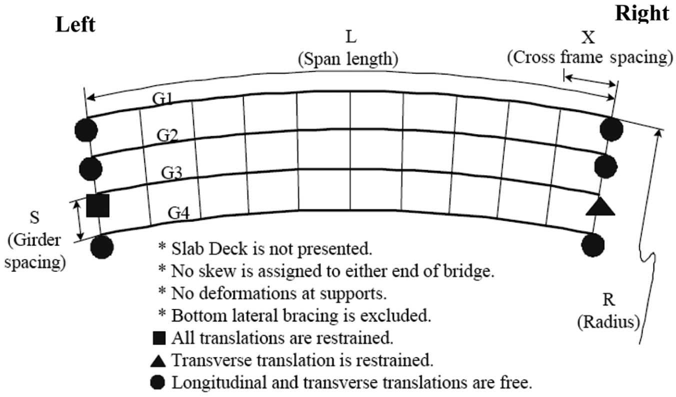

Fig. 1 Typical framing plan (Kim, 2004)

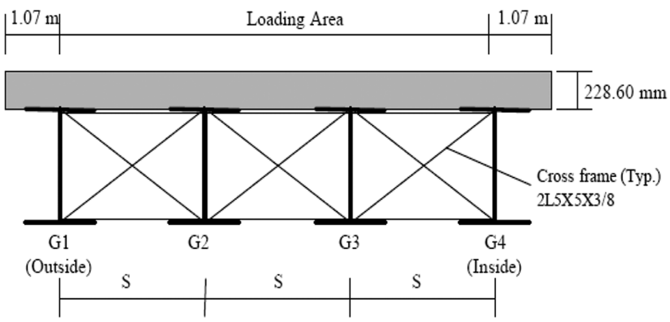

Fig. 2 Typical section (Kim, 2004)

Table 1 Parametric study information (Kim, 2004)

Bridge	Span m (ft)	Radius m (ft)	Girder spacing m (ft)	Cross Frame spacing m (ft)	Web		Flange	
					Depth m (in)	Thickness cm (in)	Width m (in)	Thickness cm (in)
1	44 (144)	61 (200)	3.1 (10)	2.2 (7.2)	1.62 (64)	1.74(0.6875)	0.61 (24)	4.45 (1.75)
2				3.7 (12)	1.67 (66)	1.74(0.6875)	0.61 (24)	5.71 (2.25)
3				5.5 (18)	1.83 (72)	1.91 (0.750)	0.61 (24)	5.71 (2.25)
4			3.3 (11)	2.2 (7.2)	1.71 (67)	1.74(0.6875)	0.61 (24)	5.08 (2)
5				3.7 (12)	1.75 (69)	1.91 (0.750)	0.61 (24)	5.71 (2.25)
6				5.5 (18)	1.87 (74)	1.91 (0.750)	0.61 (24)	5.71 (2.25)
7			3.7 (12)	2.2 (7.2)	1.75 (69)	1.91 (0.750)	0.61 (24)	5.08 (2)
8				3.7 (12)	1.77 (70)	1.91 (0.750)	0.61 (24)	5.71 (2.25)
9				5.5 (18)	1.91 (75)	2.06(0.8125)	0.61 (24)	5.71 (2.25)
10		107 (350)	3.1 (10)	2.2 (7.2)	1.57 (62)	1.58 (0.625)	0.61 (24)	5.08 (2)
11				3.7 (12)	1.57 (62)	1.58 (0.625)	0.61 (24)	5.08 (2)
12				5.5 (18)	1.65 (65)	1.74(0.6875)	0.61 (24)	5.08 (2)
13			3.3 (11)	2.2 (7.2)	1.57 (62)	1.58 (0.625)	0.61 (24)	5.08 (2)
14				3.7 (12)	1.65 (65)	1.74(0.6875)	0.61 (24)	5.08 (2)
15				5.5 (18)	1.71 (67)	1.74(0.6875)	0.61 (24)	5.08 (2)
16			3.7 (12)	2.2 (7.2)	1.52 (60)	1.58 (0.625)	0.61 (24)	5.08 (2)
17				3.7 (12)	1.67 (66)	1.74(0.6875)	0.61 (24)	5.08 (2)
18				5.5 (18)	1.72 (68)	1.91 (0.75)	0.61 (24)	5.08 (2)
19		229 (750)	3.1 (10)	2.2 (7.2)	1.52 (60)	1.58 (0.625)	0.53 (21)	4.44 (1.75)
20				3.7 (12)	1.62 (64)	1.74(0.6875)	0.53 (21)	4.44 (1.75)
21				5.5 (18)	1.67 (66)	1.74(0.6875)	0.53 (21)	4.44 (1.75)
22			3.3 (11)	2.2 (7.2)	1.60 (63)	1.58 (0.625)	0.53 (21)	4.44 (1.75)
23				3.7 (12)	1.71 (67)	1.74(0.6875)	0.53 (21)	4.44 (1.75)
24				5.5 (18)	1.72 (68)	1.74(0.6875)	0.53 (21)	4.44 (1.75)
25			3.7 (12)	2.2 (7.2)	1.67 (66)	1.74(0.6875)	0.53 (21)	4.44 (1.75)
26				3.7 (12)	1.75 (69)	1.74(0.6875)	0.53 (21)	4.44 (1.75)
27				5.5 (18)	1.77 (70)	1.74(0.6875)	0.53 (21)	4.44 (1.75)

having no bracing, were included with bracing members being placed into select cases from the 27 bridges in Table 1. The arrangements were designated as no bracing (original model), bracing in exteriors bays and bracing in every bay as shown in Figure 3. Cases from Table 1 included in the bracing study were selected based on the relative vertical displacements between interior girder (G4) and exterior girder (G1). Those cases having the maximum relative displacement and, subsequently, the maximum tangential rotation of the structure at mid-span were selected. As a result of the criteria, Bridges 1, 10, and 19 from Table 1 were examined with the three bracing arrangements.

For all cases that were examined, parametric studies were completed using finite element models developed in ABAQUS. Shell elements were used for the girders, with four 4-noded, reduced integration, elements (S4R) being used through the web depth and two S4R elements being used along the flange width. S4R elements were also used for the slab with tied-constraints being imposed to maintain

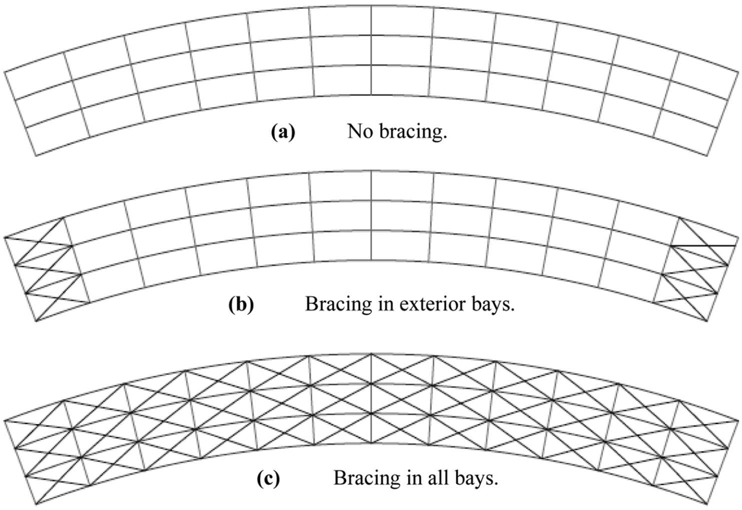

Fig. 3 Lateral bracing cases

composite action. Element aspect ratios were kept as close to one as possible. Cross frame members and lateral bracing were modeled using 3-dimensional, linear, beam elements (B31). Stiffeners were not included in the analyses. At the supports, girders G1, G2 and G4 (see Figure 1) had translations restrained in the vertical direction. G3 had translations in all directions restrained at one end and two translations restrained at the opposite end.

The choice of a seismic analysis method is generally based on the complexity of the structure being examined, the soil that supports the structure, seismic design criteria that may exist, and the presence of any earthquake resisting elements that dissipate energy inelastically. Since this study focused purely on bridge superstructures, involved regions of moderate earthquake intensity with no dissipative elements being present, and since the only anticipated nonlinear response would be via deck cracking, the possibility of extensive inelastic material response was small and elastic time history analyses were selected. It was understood that using a linear elastic approach may underestimate changes in structural stiffness and, while deck cracking would certainly influence the stiffness, again, for the moderate ground movements that were applied it was not anticipated to substantially affect results. The linear elastic seismic analyses involved modal extraction and subsequent modal integration to provide relevant seismic information. Excitation was applied to each structure in two directions, one being perpendicular to a chord joining the supports (termed horizontal) and the other being perpendicular to the plane of the structure (termed vertical). Input acceleration time histories needed to reflect local site conditions and, since adequate seismic records in the area of interest (mid-Atlantic U.S.) were unavailable, the present study utilized El Centro earthquake ground motion data as that to be modified for regional conditions. A procedure developed by Suarez and Montejo (2005, 2007) was used to modify the El Centro data. Nominal material properties were assumed for the concrete and steel.

The time history modification procedure required a predefined local response spectrum and ground motion data. The original response spectrum was based on AASHTO *LRFD Bridge Design Specifications* (2006) criteria for average soil conditions using the minimum allowable acceleration coefficient for record generation. Elastic seismic response coefficients were calculated using AASHTO Equation 3.10.6.1.1, reproduced as Equation 1

$$C_{sm} = S_a = \frac{1.2AS}{T_j^{2/3}} \leq 2.5A; \tag{1}$$

where: T_j = period of vibration of the j^{th} mode (sec),
A = acceleration coefficient from AASHTO Article 3.10.2 = 0.1 for Seismic Zone 2, and
S = site coefficient from AASHTO Table 3.10.5.1-1 = 1.2.

Response coefficients were calculated at set period increments to generate an initial spectrum, denoted as the target spectrum in Figure 4. Corresponding spectra in the horizontal and vertical directions for the original El Centro ground motion are denoted as original spectra in the same figure. The modification procedure outlined by Suarez and Montejo (2005, 2007) involves decomposition of the original record to individual time histories and rescaling of these time histories so that the resulting response spectrum reflects the desired design spectrum (AASHTO Region 2). The resulting response spectra (vertical and horizontal) after the modification procedure are denoted as modified spectra in Figure 4. It should be noted that the relationship between vertical and horizontal response spectra is still not well established, with a ratio of vertical to horizontal spectra of 2/3 being presently used for design. Published results indicate this ratio could be conservative for period of vibration over 0.3 seconds but nonconservative for shorter periods (Imbsen 2006). As a result, this study used the same design spectra for both directions and results are presented separately for those directions to decrease ambiguity. Corresponding modified acceleration data used as input for the ABAQUS models is shown in Figure 5. Since the study was limited to focusing on bridge superstructures, acceleration time-histories shown in the figures were applied directly to the girder supports in the ABAQUS models.

4. Results

As discussed previously, two common locations of damage reported for steel, girder bridges from seismic events were at the bearings and in the cross frames. In the present study, response quantities

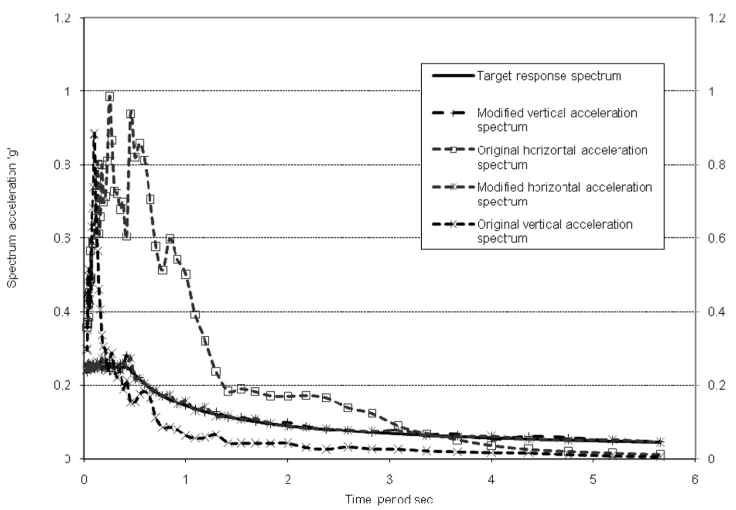

Fig. 4 Original and modified vertical and horizontal ground acceleration response spectra

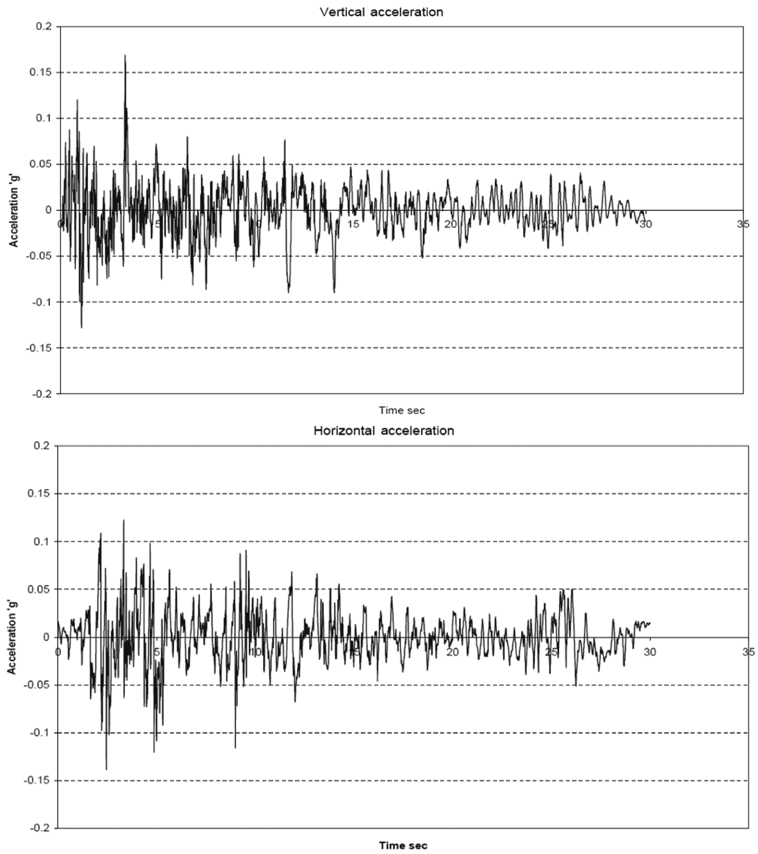

Fig. 5 Modified acceleration time histories

associated with these components were examined, with bearing performance being evaluated by examining reaction magnitudes and cross frame performance being evaluated by examining axial forces. Maximum vertical reactions and cross frame member axial forces were obtained for both modified excitation records shown in Figure 5 for each bridge that was analyzed in ABAQUS. Results were presented as values normalized with respect to dead load magnitudes found from static analyses of the structures using ABAQUS. The normalization approach was selected so that seismic effects could be compared to the substantial and permanent static load effects that all bridges experience and all designers calculate and examine, those due to their own self-weight, and they permitted quick and effective evaluation of the influence of aforementioned curved bridge parameters on those seismic effects.

5. Vertical reactions

The effect of the previously outlined parameters (i.e., radius of curvature, girder spacing, cross frame spacing, inclusion and orientation of lateral bracing) on normalized maximum vertical girder end reactions is presented. Results were normalized by dividing the maximum seismic vertical reactions, with signs corresponding to their direction, by the corresponding dead load reactions at the same support. Representative results are given for the left end supports of each structure in plan (see Figure 1).

5.1 Radius of curvature

To examine the effect of radius of curvature on end reaction and cross frame force levels, results from Bridges 1, 10, and 19 in Table 1 are presented. Bridge 1 was examined for every possible parameter set for convenience and Bridges 10 and 19 had smaller girder and cross frame spacing distances and, subsequently, amplified dynamic effects when compared to other structures listed in Table 1. Results for these structures are shown in Figure 6, which compares the variation in normalized bearing vertical reactions at the left end support as a function of girder location for the bridges that were selected. The plot details how the normalized reaction ratio varied for each structure as you traveled from exterior Girder G1 to interior Girder G4. Negative ratios indicate that the maximum seismic reactions were opposite (upward) to those from the static dead load.

In general, variation of reactions between interior and exterior girders generated from the synthetic seismic event was less pronounced when compared to variations in the reactions caused by the dead load. In addition, results were more adversely affected by vertical ground motions as opposed to horizontal motions. The one exception was for interior (lowest radius) Girder G4, which had the lowest dead load reactions as a result of the horizontal curvature. Subsequently, much higher negative normalized ratios resulted for G4 at the tightest (smallest) radius because seismic variations in the reactions were of the same order of magnitude as the initial dead load reactions and, subsequently, had an increased effect on the ratios. The negative signs indicated that possible uplift of the structure at the G4 bearings at tighter radii could also be a concern. The effects of radius of curvature were also more clearly shown for the G4 reactions when compared to the other girders, with the changes in the normalized values being more pronounced, and even changing sign, with a positive sign indicating no uplift, as opposed to the other girders. Normalized reactions for the other girders in each bridge were considerably smaller due to the high dead load reactions that existed relative to the seismic reaction magnitudes that were observed. In addition, the influence of curvature on reactions for G1, G2 and G3

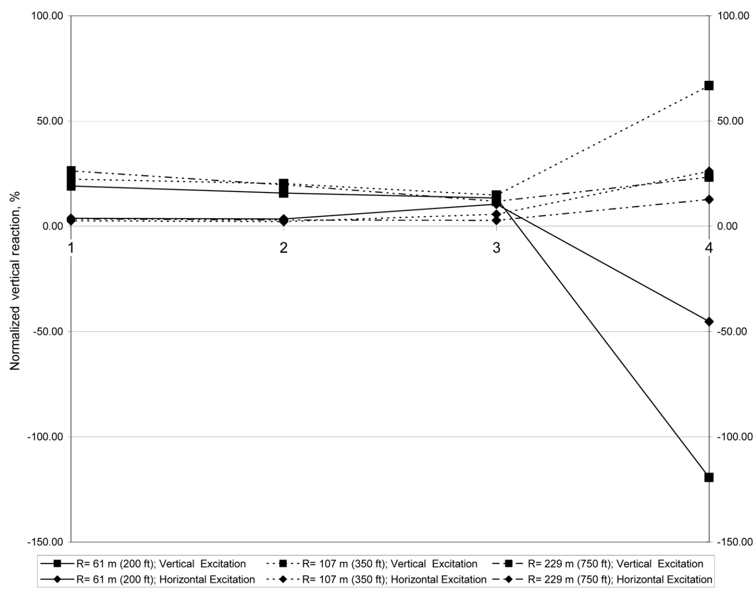

Fig. 6 Effect of radius of curvature (R) on left end normalized reactions, all girders (Bridges 1, 10, and 19)

was small and possible uplift during the seismic events that were studied appeared to not be a concern for these girders.

5.2 Girder spacing

The effect of girder spacing on seismic response is demonstrated via examination of results from Bridges 1, 4, and 7 in Table 1. These structures were selected for similar reasons to those for the radius of curvature studies: either for convenience or because their design geometries and configurations were such the influence of girder spacing on dynamic response could be clearly elicited.

As shown in Figure7, variation in normalized reactions as a function of girder spacing was clearly apparent at G4 and was caused by the relative magnitude of the static dead load reaction when compared to the seismic effects. These findings are similar to those for radius of curvature and, as the bridge radii increased, seismic effects relative to dead load effects decreased, irrespective of girder spacing. Girder spacing variation always had measureable effects on G4 seismic reaction magnitudes and, for all spacing and under vertical and horizontal excitation, uplift at G4 was possible.

5.3 Cross Frame Spacing

The effect of cross frame spacing on seismic response is presented by examining results from Bridges 1, 2, and 3 in Table 1, which were selected because they had the smallest radius of curvature that was studied and the closest girder spacing, resulting in increased mass and higher curvature effects. Figure 8. indicates that the effect of cross frame spacing on normalized vertical reactions was once again more pronounced for G4 due to smaller static dead load reactions when compared to the other girders and uplift was also evident at G4 for all cross frame spacings. However, the effects of variations in the cross frame spacing on the reactions were less pronounced under vertical excitation when compared to

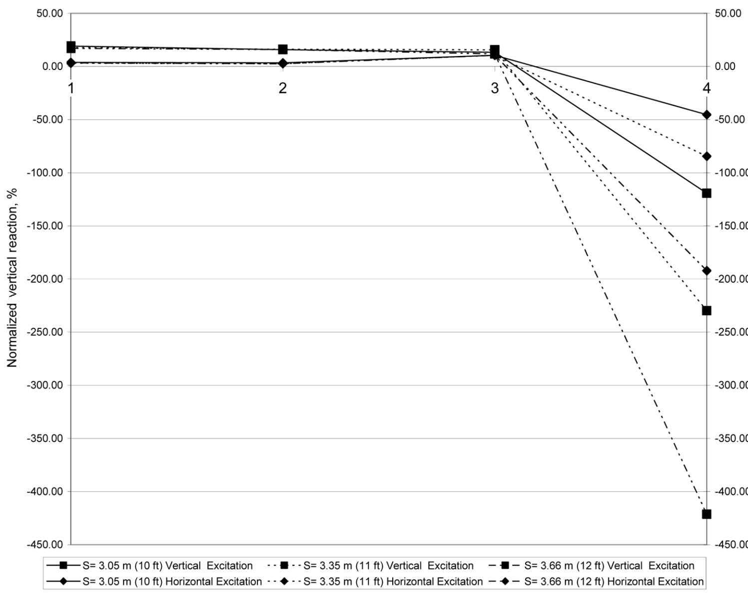

Fig. 7 Effect of girder spacing (S) on left end normalized reactions (Bridges 1, 4, and 7)

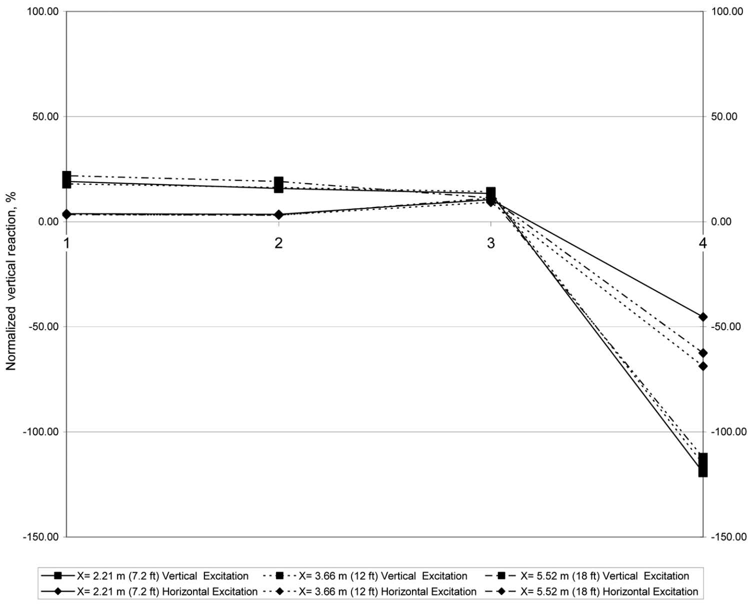

Fig. 8 Effect of cross frame spacing (X) on left end normalized reactions (Bridges 1, 2, and 3)

horizontal excitations and, in general, the overall effects of cross frame spacing on dynamic dead load reactions were smaller than the effects of radius of curvature or girder spacing.

5.4 Lateral bracing

As was stated earlier, the effects of lateral bracing on seismic response were studied for bridges from Table 1 having the largest mid-span differential displacements between the interior and exterior girders and, subsequently, maximum tangential rotations. Representative left end support reaction results from one of the three structures that were examined, Bridge 1, are presented in Figure 9. As shown in the figure, the influence of lateral bracing on response was found to be dependent on the direction of the seismic excitation. For vertical excitation, the effect of lateral bracing was quite small. However, the inclusion of bracing was shown to reduce seismic effects for horizontal excitation. The support at G4 again experienced the greatest seismic effects and, subsequently, the greatest effect from the absence or inclusion of bracing when horizontal ground excitation was considered. It is of interest to note that there was a limit to the bracing effectiveness for horizontal ground excitations, with the additional mass contributed by the use of bracing along the entire span outweighing the torsional stiffness benefit that was provided.

6. Cross frame member axial forces

The effect of the previously outlined parameters (i.e., radius of curvature, girder spacing, cross frame spacing, inclusion and orientation of lateral bracing) on normalized cross frame axial forces is presented in this section. Forces were normalized for a given cross frame member by dividing the maximum seismically induced axial force by the static dead load axial force, with both negative and

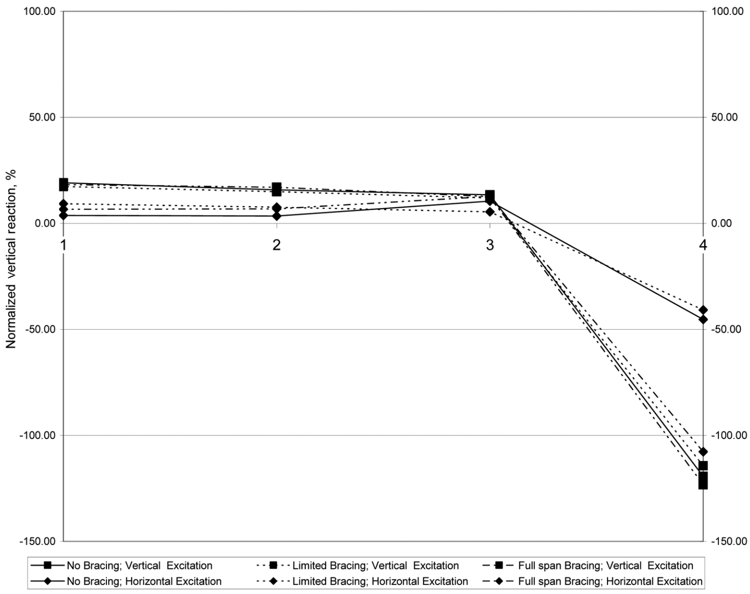

Fig. 9 Effect of lateral bracing on left end normalized reactions (Bridge 1)

positive ratios being plotted due to reversal of the seismic axial forces. Results are shown for a critical cross frame member identified from preliminary examination of the seismic response of Bridges 1, 2 and 3 in Table 1. These preliminary studies identified the critical member as upper chord member U1 between G2 and G3 in the cross frame located at mid-span as shown in Figure 10 (see Figure 1 for structure framing plan).

6.1 Radius of curvature

To examine the effect of radius of curvature on normalized cross frame axial forces, Bridges 3, 12 and 21 from Table 1, structures with higher axial forces due to their larger cross frame spacing, were selected. Results for these structures, which plot the normalized axial forces in U1 for each cross frame with respect to the normalized length of the bridge, are presented in Figure 11 for vertical seismic excitation and in Figure 12 for horizontal excitation.

As shown in Table 1, the radius of curvature increased from 61 m (200’) for Bridge 3 to 229 m (750’) for Bridge 21. As a result of the higher radius for Bridge 21, dead load axial forces in U1 decreased and,

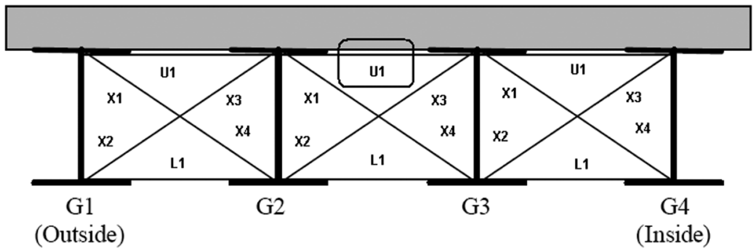

Fig. 10 Cross frame member designations

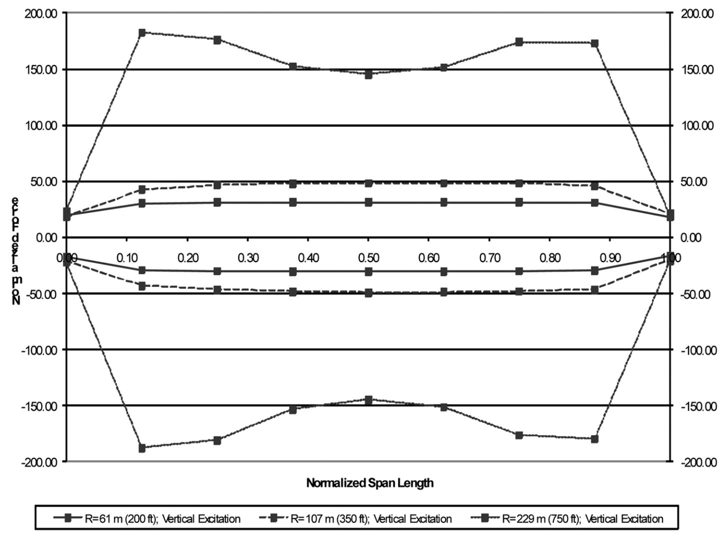

Fig. 11 Effect of radius of curvature (R) on normalized U1 axial forces, vertical seismic excitation (Bridges 3, 12, and 21)

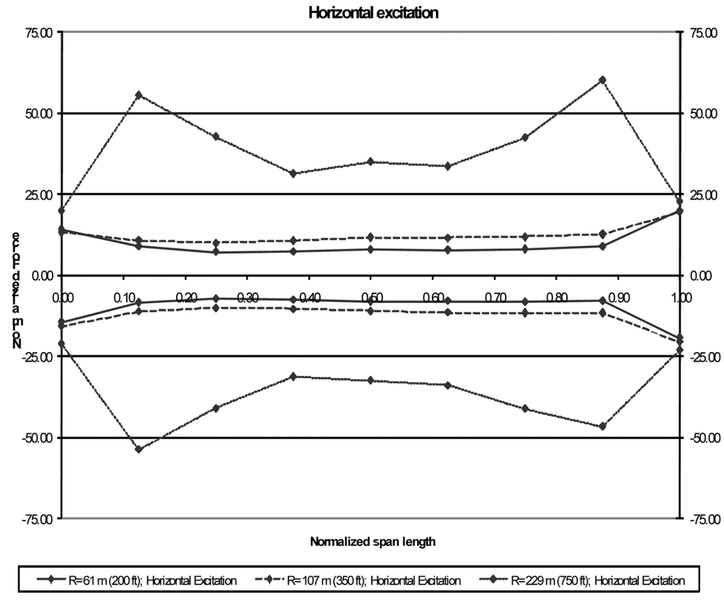

Fig. 12 Effect of radius of curvature (R) on normalized U1 axial forces, horizontal seismic excitation (Bridges 3, 12, and 21)

thus, when axial force induced by the seismic excitations was normalized with respect to this static axial force, large ratios resulted. These ratios were a direct result of the relatively low initial axial forces in the member and not necessarily due to high seismic forces, but they are indicative of the potential change in forces that could result should a seismic event similar to those that were applied occur. The figures also indicate that vertical excitation resulted in more pronounced changes in seismic effects as a function of curvature and, in addition, seismic effects on axial forces were more pronounced away from the supports. Irrespective of radius, none of the resulting total (static + seismic) compressive stress levels in U1 exceeded its critical stress levels.

6.2 Girder spacing

The effect of girder spacing on normalized cross frame axial forces is presented by examining results from Bridges 3, 6, and 9 from Table 1. These structures were selected because of their tight radii and, subsequently, appreciable dead load axial force in the cross frames and results are presented in Figure 13 and Figure 14. The effect of girder spacing on the normalized axial force in U1 was found to be quite small, irrespective of excitation direction. However, horizontal excitation did produce more pronounced seismic effects in U1. Again, no load reversal was evident, seismic effects were more pronounced away from the supports and none of the U1 total stresses exceeded critical stress levels.

6.3 Cross frame spacing

The effect of cross frame spacing on member axial forces is presented by examining the results of the Bridges 1, 2, and 3 from Table 1, which were, again, structures having critical radii of curvature and extreme cross frame axial forces. Results are presented in Figure 15 and Figure 16. The increase in cross frame spacing from 2.2 m (7.2') to 5.5 m (18') between Bridge 1 and Bridge 3 did result in an increase in the maximum axial force developed in the mid-span member. However, results were similar to those for the variation of girder spacing: cross frame spacing effects were small with horizontal excitation producing more pronounced differences as a function of spacing; load reversal was not evident; seismic effects were more evident away from the supports; and no U1 total stresses exceeded critical values.

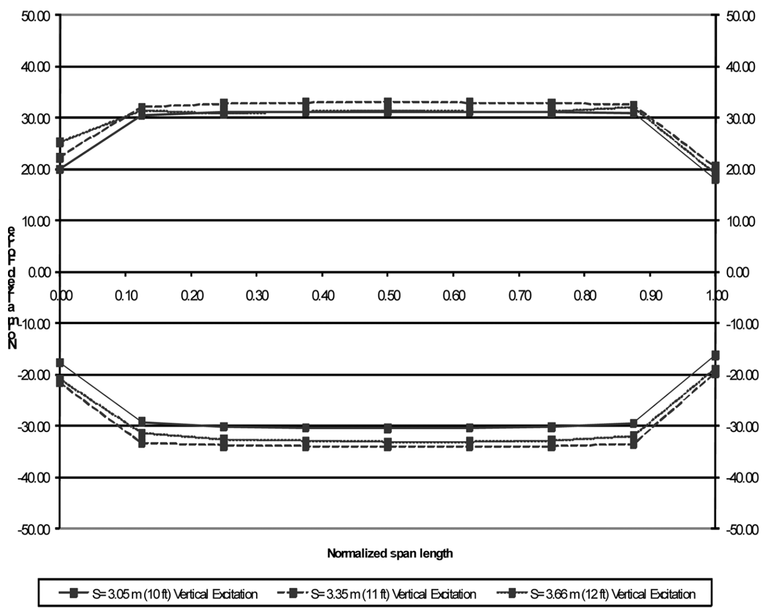

Fig. 13 Effect of girder spacing (S) on normalized U1 axial forces, vertical seismic excitation (Bridges 3, 6, and 9)

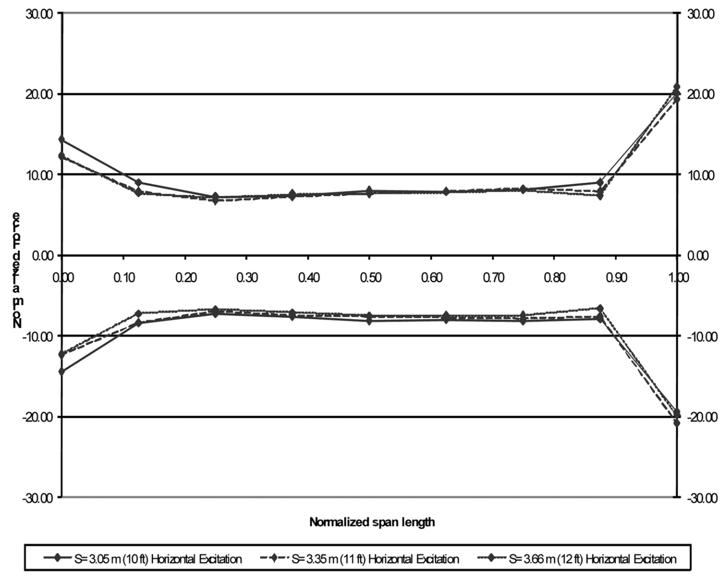

Fig. 14 Effect of girder spacing (S) on normalized U1 axial forces, horizontal seismic excitation (Bridges 3, 6, and 9)

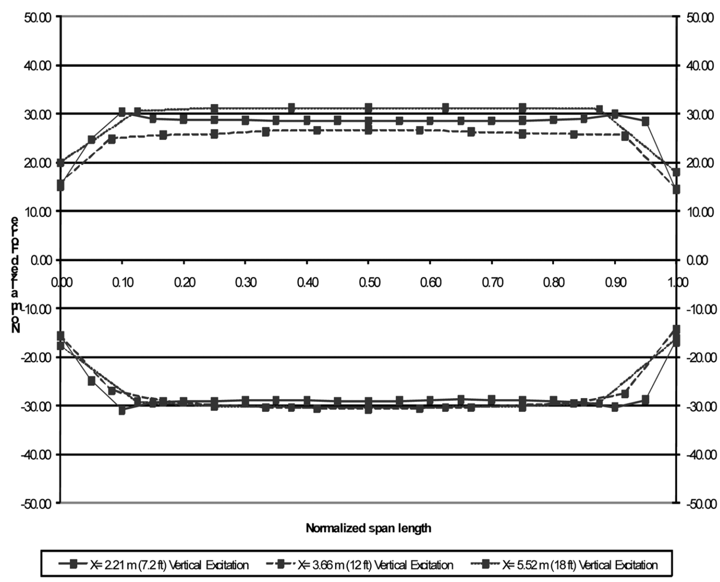

Fig. 15 Effect of cross frame spacing (X) on normalized U1 axial forces, vertical seismic excitation (Bridges 1, 2, and 3)

6.4 Lateral bracing

Similar to what was presented for support reactions, representative results from one of three structures examined with the three different lateral bracing configurations (no bracing, limited bracing, full bracing), Bridge 1, are presented. Results are shown in Figure 17 and Figure 18 and the effect of

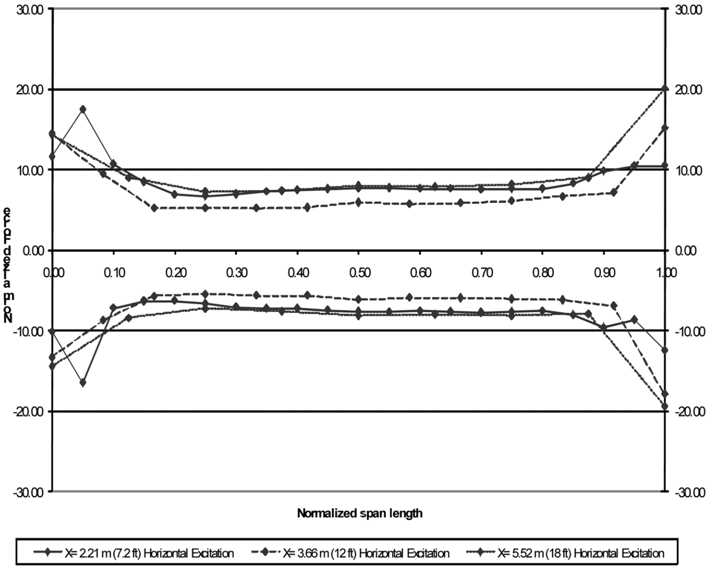

Fig. 16 Effect of cross frame spacing (X) on normalized U1 axial forces, horizontal seismic excitation (Bridges 1, 2, and 3)

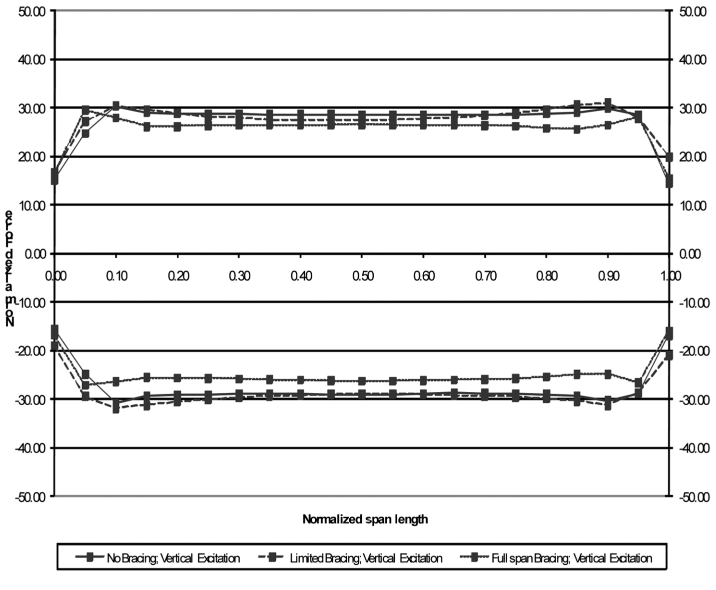

Fig. 17 Effect of lateral bracing on normalized U1 axial forces, vertical seismic excitation (Bridge 1)

direction of excitation on normalized ratios is again clearly demonstrated. Seismic effects are more evident here for vertical excitation and no clear evidence of the effects of bracing configuration on seismic response is evident. This is attributed to the bracing members influencing both static and seismic load levels in curved, steel I-girder bridges in similar fashion. Results indicate that lateral bracing provides seismic benefits with regards to dynamic load sharing between girders that are of

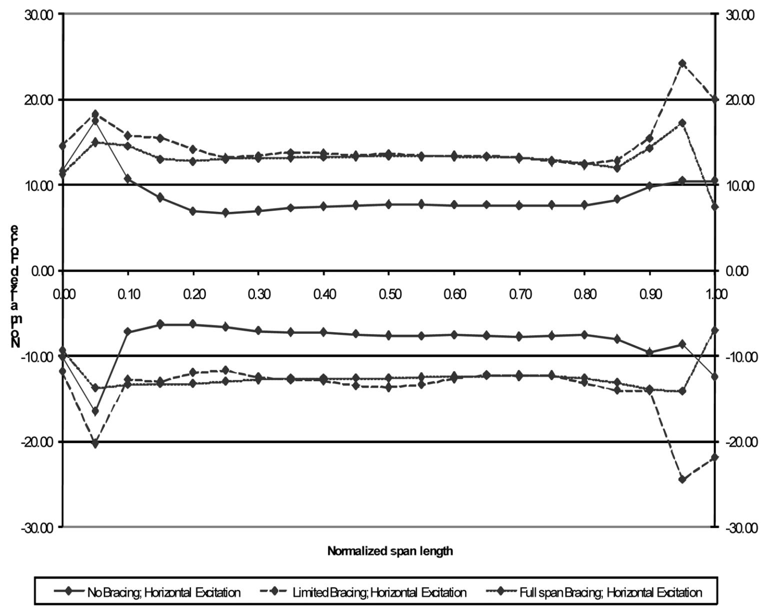

Fig. 18 Effect of lateral bracing on normalized U1 axial forces, horizontal seismic excitation (Bridge 1)

similar order to benefits that are provided with respect to sharing dead load between the girders.

7. Conclusions

Limited examinations of curved, steel bridge seismic response have occurred in the literature. Therefore, a parametric study focusing on the effects of variations in radius of curvature, girder spacing, cross frame spacing and lateral bracing placement on the seismic response of 27 curved, steel, I-girder bridge designs was completed. The intent of these studies was to establish the level of seismic effects relative to dead load levels induced at the supports and in critical cross frame members, structural elements that have been shown to be sensitive to seismic effects. Analyses were completed using ABAQUS with El Centro ground motion data, modified to reflect regional seismic conditions, used as the seismic excitation.

The following conclusions were for the bridges that were examined and ground motions that were applied:

1. In general, radius of curvature was shown to be the primary parameter affecting seismic load levels at the bearings and in critical cross frame members. However, the other parameters that were studied did have, to a lesser extent, an influence on seismic response, especially at the bearings.
2. Reactions at the interior (lowest radius) girder (G4) were shown to be heavily influenced by all parameters that were studied for the seismic excitations that were applied. At lower radii uplift under seismic loads was shown to be a concern, irrespective of the variation of other parameters, for the bridges that were examined. This would appear to suggest that, as a minimum, interior girder reactions should be studied under seismic loads for bridges similar to those examined here

to ascertain their effects.

3. Vertical excitation was shown to have a larger influence on seismic load levels, results that were attributed to the curvature of the structure generating more load sharing between girders when it was excited vertically.
4. The inclusion of lower lateral bracing was shown to be effective for mitigating seismic effects in the components that were studied. However, the use of limited bracing was shown to be just as effective as the use bracing along the entire span.

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