Seismic Performance of Single-Span Simply Supported and Continuous Slab-on-Girder Steel Highway Bridges

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Abstract: Seismic response of single-span simply supported and continuous slab-on-girder steel bridges is studied. Linear elastic and nonlinear inelastic analyses are conducted for various ground-acceleration inputs. Bearings with higher stiffnesses that are closer to the edge of the deck are found to attract larger forces than other bearings, whereas bearings with small stiffness attract almost equal forces regardless of the width of the bridge. Assuming that the anchors of otherwise stable bearings are severed, sliding of the bridges is also investigated. It is found that narrower bridges with longer spans may have considerable sliding displacements and may fall off their supports if adequate seat width is not provided. Wider bridges with shorter spans can survive rather severe earthquakes, even more so in eastern North America. It is found that if bearings are not damaged, 2- and 3-lane continuous bridges with two spans of up to 40 and 50 m each, respectively, are capable to resist earthquakes with 0.4g peak acceleration without any damage to columns. However, bridges with only about 80% of these spans can survive similarly intense earthquakes if the bearings are damaged.

INTRODUCTION

Extensive research has already been conducted to identify the structural components and parameters that significantly affect the seismic response of reinforced concrete bridges. For instance, the effectiveness of longitudinal restrainer ties at the expansion joints; the impact of column strength, detailing, and distribution in each deck segment (Tseng and Penzien 1973; Penzien and Chen 1975; Degenkolb 1978); the need to restrain the displacements of unstable rocker bearings (Douglas 1979); the effect of impacting between adjacent bridge sections on the superstructure-substructure connections (Imbsen and Penzien 1984); the need for shear keys (Imbsen and Penzien 1984); the negative influence on nonuniform column-stiffness distribution and the shear-failure vulnerability of bridge piers (Priestley 1985, 1988; Priestley and Park 1987; Ghobarah and Ali 1988; Saiidi et al. 1988); the effect of skewness on the reinforced concrete columns (Ghobarah and Tso 1974); and the effect of curvature in long curved continuous bridges (Williams and Godden 1975) have received considerable attention. This has led to considerable improvements in bridge-design codes and to better understanding of the dynamic behavior of bridge structures.

By contrast, little information is found on the seismic behavior of steel bridges in the existing literature, partly because few have been exposed to severe earthquakes in the past, at least until the Loma Prieta earthquake of 1989 (Earthquake 1990). Some research findings exist on the seismic performance of steel box sections often used in Japan for bridge piers (Macrae et al. 1990), provisions for the seismic design of steel bridges have been proposed by critically extrapolating and adding to the requirements applicable to steel buildings (Kulicki and Clancy 1993), and the seismic resistance of long-span steel bridges is currently being studied (Astaneh and Roberts 1993), but to this date no standard specifications exist in North America to provide clear guidance on this topic.

Because most steel bridges in North American inventory have been designed without any seismic resistance considerations, their seismic adequacy is legitimately questionable. Obviously, some of the general research on the behavior of generic bridges is broadly applicable to any bridge type and can be useful. For example, research on the correlation of computer analysis results with actual seismic bridge behavior have led to improved modeling techniques and computer programs (Tseng and Penzien 1973; Kawashima and Penzien 1976; Douglas 1979; Imbsen and Penzien 1979, 1984; Wilson 1986) that are used to develop analytical models for the analyses of steel bridges considered in this study. However, much research is still needed to reliably assess the seismic resistance of steel bridges deficiently or never designed to resist earthquakes.

On first examination, in slab-on-girder steel bridges commonly used throughout the North American highway system, the bearings and columns appear to be the weakest links of structural resistance to the seismic loads. In this paper, the seismic performance of existing slab-on-girder steel highway bridges is studied, first considering that damage to bearings is unacceptable, then by releasing this constraint. Although in both cases damage to columns is obviously unacceptable, the latter case is of particular interest because there exists many short- to medium-span old steel bridges supported by sliding bearings with anchor bolts not designed to resist seismic forces that are vulnerable to earthquakes. When these anchor bolts are severed, and friction forces between the disconnected bridge components are exceeded, the bridge deck may slide to a point where insufficient bearing resistance (or brittle detailing) exists to resist the suddenly abnormally located gravity reaction forces, or may even fall off its support. Due to excessive sliding displacements, columns in continuous bridges may also suffer damage before the bridge deck falls off its support.

Therefore, for the purpose of this study, both linear elastic and nonlinear inelastic dynamic analyses are performed. The objective in performing linear elastic analyses is to find the seismically induced bearings' forces and elastic column moments. The nonlinear inelastic analyses are performed (1) to find the maximum transverse sliding displacement at the bearings as a function of various parameters, considering that the bridge deck may slide in both directions and collide with the abutment walls or fall off its support; and (2) to determine the effect of sliding at the bearings on the seismic capacity of continuous bridges considering the ultimate capacity of the columns.

In all cases, abutments and foundations deformations and/or damage as well as soil-structure interaction are beyond the scope of this study. The objective is to outline the expected

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problems when the structure alone is investigated without the effect of soil-structure interaction. Therefore, a better understanding of future results generated considering soil-structure interaction will be possible when used in conjunction with the results reported here.

PROPERTIES OF BRIDGES STUDIED

A large percentage of the existing slab-on-girder steel highway bridges were constructed in the 1960s. Accordingly, to study typical old steel highway bridges and better understand their expected seismic performance, 2- and 3-lane single-span simply supported bridges with spans of 20, 30, 40, 50, and 60 m, and continuous bridges with two spans of 20, 30, 40, 50, and 60 m each, are designed in compliance with the 1961 edition of the American Association of State Highway Officials (AASHO) code (*Standard* 1961). The 2- and 3-lane bridges have, respectively, 8 and 12 m widths and girders spaced at 2-m intervals. A two-span continuous bridge is shown in Fig. 1. The deck of these bridges is generally supported by fixed bearings on one abutment and by expansion bearings on the other abutment. The deck of continuous bridges is also supported by steel columns of 6 m in height as typically found in most highway bridges, rigidly connected to each steel girder at midspan to make a moment-resisting steel frame. A simple connection is assumed at the base of each column. Columns are oriented such that strong-axis bending is mobilized by movement in the longitudinal direction of the bridges. Additionally, 2-lane bridges with 3, 4, 5, and 6 spans of 20 m each are also designed and studied. A 1-m overhang is assumed on both sides of the decks for all the bridges. Slab thickness is taken as 200 mm.

All the bridges are assumed to have sliding bearings, but elastomeric bearings are also considered in the case of single-span bridges. The fixed type of sliding bearings is shown in Fig. 2. The expansion type of bearing is nearly identical but

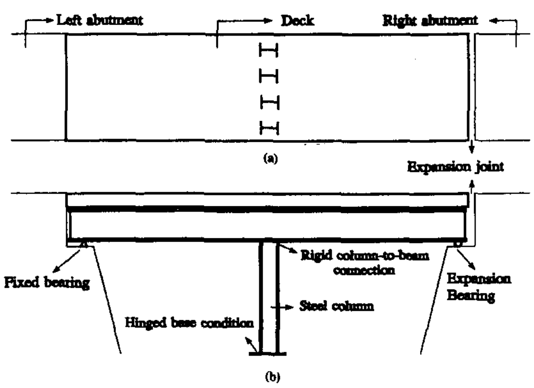

FIG. 1. Typical Single-Span Simply Supported Bridge: (a) Plan View; (b) Side View

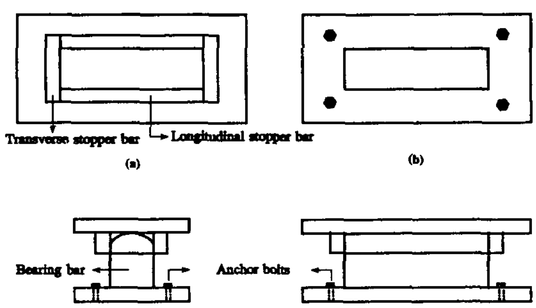

FIG. 2. Typical Fixed Sliding Bearing: (a) Top Plate; (b) Bottom Plate Plan View; (c) Side View of Bearing; (d) Front View of Bearing

without the longitudinal stopper bars. For the sliding bearings, based on the recommendations of AASHO, two anchor bolts of 32 mm in diameter and four anchor bolts of 38 mm in diameter per bearing are taken as lower and upper bounds, respectively, for the span ranges considered in this research. Considering an average-sized bearing bar and bearing plates, and using the bearing-stiffness model described later, the longitudinal stiffness is calculated as 400,000 kN/m and 800,000 kN/m for bearings with two and four bolts, respectively. Only the case of sliding bearings with 800,000 kN/m stiffness is considered in the case of continuous bridges.

The theoretical cases of bearings with zero and infinite rotational stiffness are also considered in the case of simply supported bridges. Infinite rotational stiffness about an axis perpendicular to the deck occurs when the fixed bearing is infinitely rigid in the longitudinal direction. Zero rotational stiffness occurs when the fixed bearing has negligible longitudinal stiffness (a purely theoretical case) or when the bearings are damaged. In both cases, transverse stiffness is assumed to be infinitely rigid. Also, for simply supported bridges, a 4,000 kN/m stiffness per elastomeric bearing is assumed for the span ranges considered (Heins and Firmage 1979).

EARTHQUAKE LOADING

Ground motions can be characterized by the peak ground acceleration to peak ground velocity ratio, A_p/V_p, where A_p is expressed in units of the gravitational acceleration and V_p is expressed in meters per second (Zhu et al. 1988). Ground motions with higher frequency content have higher A_p/V_p ratios. In eastern North America, generally, $A_p \geq V_p$, and in western North America, $A_p \leq V_p$. As V_p gets larger relative to A_p, higher spectral-acceleration values are obtained in the long-period region of the spectrum, and spectral acceleration values almost remain the same in the short-period region for all A_p/V_p ratios.

For the elastic analyses, the eastern Canada design spectrum with $Z_a = Z_v$ is taken as representative of eastern North America seismicity and as a conservative envelope of all cases for which $Z_a \geq Z_v$, where Z_a and Z_v are, respectively, the peak-acceleration and velocity-zone parameters (*National* 1990) and directly related to A_p and V_p. Similarly, for western North America, the western Canada design spectrum with $Z_a < Z_v$ is taken as a conservative envelope to study the response of steel bridges. The necessity and advantages of recognizing different spectral characteristics of eastern and western North America seismicity are well established (Heidebrecht et al. 1983). It is also of major significance in this study.

For the nonlinear time-history analyses, four western United States earthquakes all recorded on rock or stiff soil are chosen as representative of the strong-motion data in western North America. Two eastern Canada earthquake records, one on bedrock and another on alluvium deposit, are also considered. The properties of all the earthquake motions used in this study are shown in Table 1. The smoothed acceleration spectrum that matches the mean plus one standard deviation (MP1SD) of the spectra of the western U.S. earthquakes for 5% damping is generated (Dicleli 1989). It is in close agreement with the Uniform Building Code (UBC) spectrum (*Uniform* 1991). The response spectra of the earthquake records and the design response spectra are illustrated in Fig. 3. The vertical axis in Fig. 3 is β, which is the ratio of the pseudo spectral acceleration, S_a, to the peak ground acceleration, A_p.

LINEAR ELASTIC MODELING

The stiffness of the fixed type of sliding bearings in the longitudinal direction is formulated considering only the con-

TABLE 1. Properties of Selected Earthquakes

Site (1)	Location (2)	Date (3)	Component (4)	Magnitude (5)	Source distance (km) (6)	Soil type (7)	Maximum acceleration (g) (8)
Imperial Valley, Calif.	El Centro	1940	S00E	6.6	8	Stiff soil	0.348
Kern County, Calif.	Taft	1952	S69E	7.6	56	Rock	0.179
San Fernando, Calif.	Pacoima Dam	1971	S16E	6.6	8	Rock	1.170
Parkfield, Calif.	Chalome Shandon 2	1966	N65E	5.6	0.1	Stiff soil	0.480
Québec, Canada	Chicoutimi-Nord	1984	N18E	6.0	43	Rock	0.131
Québec, Canada	Baie-St-Paul	1982	S59E	6.0	91	Alluvium	0.174

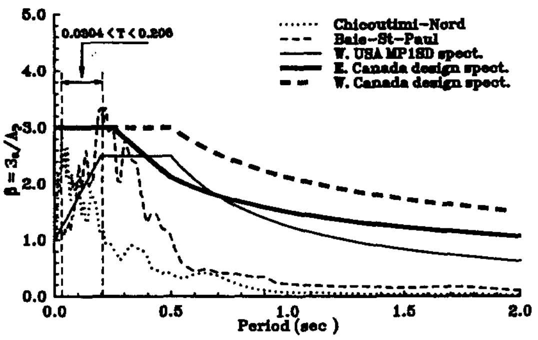

FIG. 3. Elastic-Response Spectra of Eastern Canada, Western Canada, and U.S. Earthquakes for 5% Damping

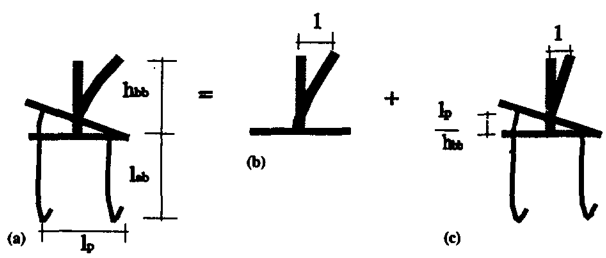

FIG. 4. Deformation of Sliding Bearings: (a) Total; (b) due to Bending of Bearing Bar; (c) due to Elongation of Anchor Bolts

tributions from bending of the bearing bar and elongation of the anchor bolts (Imbsen and Penzien 1984), as seen in Fig. 4. Knowing that the bearing bar and the anchor bolts are acting as springs connected in series, the longitudinal spring coefficient, k_{bL}, for each bearing is obtained as follows:

$$k_{bL} = 1 \Bigg/ \left\{ [1/(3EI_{bb}/h_{bb}^3)] + \left[1 \Bigg/ \left((E/h_{bb}) \sum_{i=1}^{n_{ab}} (l_p A_{ab}/l_{ab})_i \right) \right] \right\} \quad (1)$$

where E = modulus of elasticity of steel; I_{bb} = moment of inertia of the cross section of the bearing bar parallel to the deck about an axis in the bridge's transverse direction; h_{bb} = height of the bearing bar, l_p = length between an anchor bolt in tension and tip of the bottom plate; A_{ab} and l_{ab} = area and the length of the anchor bolt; and n_{ab} = number of anchor bolts. By using this spring coefficient for each bearing located under each girder, the longitudinal effect of the bearings set is transformed into one translational spring parallel to the span and one rotational spring about a vertical axis perpendicular to the bridge deck. Both springs are located at the centerline of the bridge deck and at the one end of the bridge where fixed bearings are present. In the transverse direction, sliding bearings are very rigid; therefore the bearing stiffness, k_{bT}, in this direction need not be calculated.

The bridge span is divided into three-dimensional beam segments, and full composite action between the slab and girders is assumed for the response in both orthogonal directions. For the continuous bridges, a rigid bar connected to the beam element's end node at the columns' location and oriented in the transverse direction is used to model the interaction between the columns and the bridge deck. The program SAP90 (Wilson and Habibullah 1990) is used for the response spectrum analyses of the elastic models.

NONLINEAR INELASTIC MODELING

When bearing forces calculated as per these elastic analyses exceed the bearings' resistance, the sliding of the bridge deck in the transverse direction is investigated using inelastic analysis. The program NEABS (Penzien et al. 1981) is used for this purpose. Because the nonprioritary version of this program cannot handle transverse-direction sliding assuming that the bearings are damaged, a conceptually simple and computationally effective equivalent model is used to simulate the nonlinear behavior of the actual structure.

In the model, the bearings are assumed to be immediately damaged and their initial elastic contribution to response is conservatively ignored. Thus, a rigid-plastic hysteresis model is assumed to simulate sliding of the bridge because the peak of ultimate bearing resistance prior to anchor-bolt failure occurs only once and does not significantly affect the total amount of energy dissipated through sliding. This is a reasonable simplification considering that results obtained from inelastic analyses would only be consulted if the seismically induced forces in bearings calculated using elastic analyses were found to exceed their resistance. It is noteworthy, however, that even minor earthquakes have caused failure of anchor bolts, keeper bar bolts, and welds (Federal 1987).

Moreover, assuming that the bridge deck stays elastic before and after sliding, and because elastic dynamic analyses showed that the fundamental transverse-direction mode of vibration is dominant for these bridges, the structure is idealized as a single degree of freedom (SDOF) system that slides once the friction resistance is exceeded. Because steel columns were found to have an insignificant effect on the dynamic behavior of continuous slab-on-girder steel bridges, largely due to their very low transverse stiffness relative to the in-plane stiffness of the deck, and because there is no rotational resistance at the supports when bearings are assumed to be damaged, the first mode shape of the real system can be represented by the following function:

$$\Phi(x) = \sin(\pi x/L_T)\, dx \quad (2)$$

where L_T = total end-to-end length of the bridge. Then, the generalized mass m^*, stiffness k^*, and the effective force P_{eff} for the equivalent model are calculated as follows (Clough and Penzien 1975):

$$m^* = \int_0^{L} \frac{m}{L_T} [\Phi(x)]^2\, dx = \int_0^{L} \frac{m}{L_T} \sin^2 \frac{\pi x}{L_T}\, dx = \frac{m}{2} \quad (3)$$

$$k^* = \int_0^L EI_D \left[\frac{d^2\Phi(x)}{dx^2}\right]^2 dx = \int_0^L EI_D \left(\frac{\pi^2}{L_T^2}\sin\frac{\pi x}{L_T}\right)^2 dx$$

$$= \frac{\pi^4 EI_D}{2L_T^3} \tag{4}$$

$$P_{eff} = \ddot{u}_g \int_0^L \frac{m}{L_T}\Phi(x)\,dx = \ddot{u}_g \int_0^L \frac{m}{L_T}\sin\frac{\pi x}{L_T}\,dx = \frac{2m}{\pi}\ddot{u}_g \tag{5}$$

where m = mass of the bridge; I_D = moment of inertia of the bridge deck about a vertical axis; and $\ddot{u}_g$ = acceleration of the ground motion. It is noteworthy that I_D is calculated using a modular ratio of 9 considering the composite action between the concrete slab and steel girders (Douglas 1979). For the equivalent system considered here, it is the effective force, P_{eff}, that acts on the same system, not $m^*\ddot{u}_g$. Therefore, the ground-acceleration history is corrected by an acceleration modification factor, AMF, expressed as follows:

$$AMF = P_{eff}/m^*\ddot{u}_g = 4/\pi \tag{6}$$

The idealized structure model and the actual system are illustrated in Fig. 5 for the continuous bridge. The lateral stiffness of the bridge deck is modeled using a NEABS beam element with axial stiffness equal to k^*. An expansion joint element with a sliding elastic-plastic hysteretic subelement is used for the model. The stiffness of all the columns in the transverse direction are summed up to obtain an equivalent single column stiffness. Because the axial forces in columns due to seismic excitation are negligible relative to those due to gravity loading, the column is modeled as a beam element with axial stiffness equal to the originally calculated flexural stiffness. The simply supported bridges are modeled in the same way but without the equivalent column element.

Because the seismic forces acting on the actual bridge and the equivalent model are not identical, a modified vertical reaction force, R^*, for the equivalent model is needed so that sliding occurs at the same time in both the equivalent and real systems. The modified reaction force is obtained by multiplying the actual reaction force at the supports, R, by the ratio of the seismic force acting on the equivalent model, H^*, to the total seismic force acting on the real system, H

$$R^* = (H^*/H)R = (\pi/4)R \tag{7}$$

This force is applied on the beam node connected to the expansion joint element in the equivalent model. Finally, mass proportional damping and 5% damping are considered. Incidentally, only slight modifications are needed to model continuous bridges with more spans.

LINEAR ELASTIC ANALYSIS RESULTS—TRANSVERSE DIRECTION

Expansion Joint Displacements

The displacements at the corners of the bridge decks due to support rotations resulting from transverse-direction seismic excitation are calculated using the western Canada design spectrum scaled to 0.5g peak ground acceleration. The displacements for the simply supported and continuous bridges are on average less than 15% and 22% of the expansion joint width (EJW) respectively, where the EJWs are sized to allow a temperature variation of 50°C. Therefore, collision of the deck with the abutment wall is not probable in the transverse loading case. It is noteworthy that the support rotations of 2-lane bridges are almost twice those of 3-lane bridges.

Bearing Forces

Response spectrum analyses of 2- and 3-lane simply supported and continuous bridges are conducted in the transverse direction. As illustrated in Fig. 6, the forces, B_r, in fixed type of sliding bearings are calculated first by dividing the transverse reaction force at the abutment by the number of bearings and then summing vectorially this force and the longitudinal force produced by the resistance to rotation at the bearings. As seen in that figure, bearings farthest to the bridge centerline attract larger seismic forces, and are therefore the most critical ones. Obviously, the expansion type of sliding bearing does not attract as much force as the fixed type, because the deck can freely rotate about a vertical axis. Therefore, the bearing forces in this case are calculated simply by dividing the reaction force on the abutment by the number of bearings.

The effect of end fixity on the transverse bearing forces is illustrated in Figs. 7(a) and 7(b) for 2- and 3-lane simply supported bridges, respectively. In these figures, the maximum transverse-bearing-force coefficient (TBFC) is plotted as a function of span length for various bearing stiffnesses. The TBFC is obtained by dividing the maximum of the resultant bearing forces due to seismic loading in the transverse direction by the bridge mass and the peak ground acceleration. An eastern Canada design spectrum is arbitrarily chosen for the analysis. It is observed that as the stiffness of the bearings increases, the TBFC becomes larger. For all types of bearings, except for elastomeric and those with zero rotational stiffness, the TBFC increases with span length. This is mostly due to the corresponding increase of the in-plane end moment at the support.

In the case of bearings with zero or very small longitudinal, but infinite transverse, stiffness, the TBFC is almost constant; however, absolute bearing forces actually increase with span length largely due to the increased reactive mass of longer bridges. In the case of bridges with elastomeric bearings, due to their much longer periods that fall in the declining part of the response spectra, the structures attract relatively less seismic forces and therefore very small bearing forces are obtained.

In Fig. 8(a), the ratio of the bearing forces of 2-lane to 3-lane bridges is plotted as a function of span length. For bear-

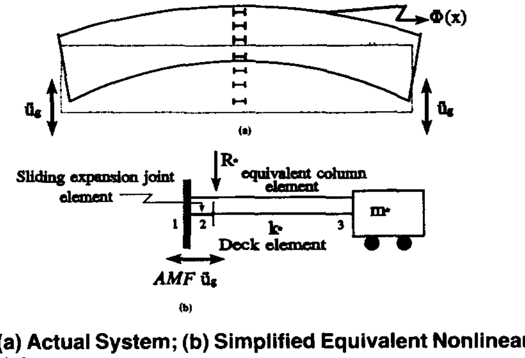

FIG. 5. (a) Actual System; (b) Simplified Equivalent Nonlinear Inelastic Model

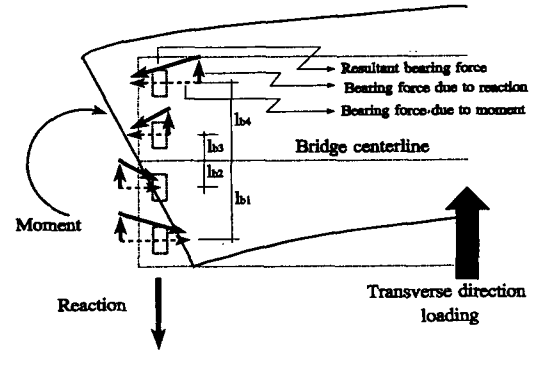

FIG. 6. Bearing Forces due to Loading in Transverse Direction

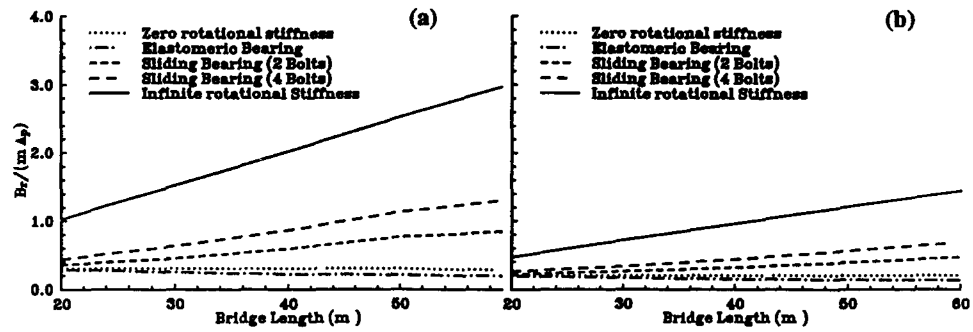

FIG. 7. Effect of Bearing's Stiffness on Transverse Bearing Force: (a) 2-Lane Simply Supported Bridges; (b) 3-Lane Simply Supported Bridges (Eastern Canada Design Spectrum)

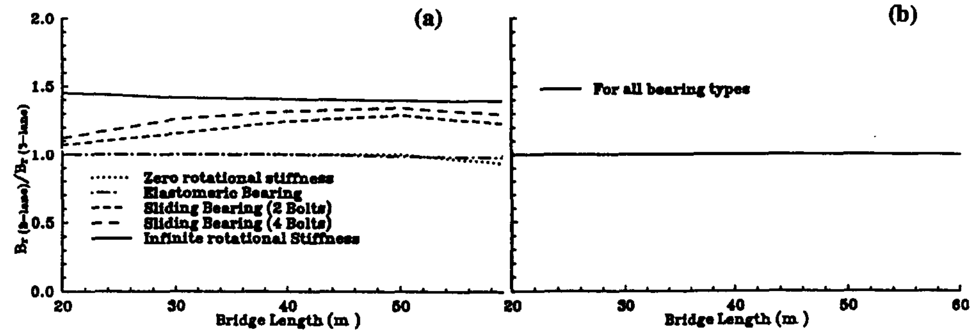

FIG. 8. Ratio for Different End-Fixity Conditions as Function of Span Length: (a) Transverse; (b) Longitudinal Bearing Forces of 2-Lane and 3-Lane Bridges

ings of infinite longitudinal stiffness, this ratio is approximately 1.5, but it is 1.0 for bearings with zero longitudinal stiffness. In the case of sliding bearings, the ratio increases with span length. This is principally due to the increase in the end moment. Thus, the TBFC obtained for narrower bridges can be used conservatively for wider bridges.

For the continuous bridges, results similar to those for simply supported bridges are observed for the bearing forces. However, in this case, the difference between the absolute bearing forces of 2- and 3-lane continuous bridges is not very large, and is about 30% for spans up to 30 m and 10% for longer spans. As an overall trend, transverse bearing forces are proportional to the mass of the bridge, hence to the span length, and are larger in 2-lane bridges.

The deck width and girder spacing may vary and therefore affect the bearing forces due to seismic excitation. To investigate this effect, 2-lane bridges with various combinations of deck width and girder spacing are considered. The practical limits of deck width and girder spacing for such bridges are assumed to vary between 6 and 10 m and 1.5 and 3 m, respectively. These bridges are analyzed using the eastern Canada design spectrum, assuming that they have sliding-bearings with four bolts. The results are compared with those of the previously considered 2-lane bridges with 8-m deck width and 2-m girder spacing. Analysis results indicate that the bearing forces of bridges with decks wider than 4 m per lane are smaller, but the difference is not very large. Furthermore, narrower bridges can have larger bearing forces, but a difference of no more than 12% was obtained for the worst case considered.

Column Response for Continuous Bridges

A review of the literature on the cyclic behavior of steel members revealed that such research has concentrated solely on building elements, and only a few researchers addressed the weak column–strong beam (WCSB) behavior more typical of steel bridges. Popov et al. (1975) tested two different compact W-shaped sections braced to prevent lateral buckling about their weak axis. Results showed that the cyclic behavior of the specimens is a function of both the applied axial load to yield axial load ratio, i.e., P/P_y, and the magnitude of interstory drift. Sudden failure was observed in specimens having P/P_y ratios larger than 0.5. Later, Takanashi and Ohi (1984) performed a shaking-table test on a small-scale three-story WCSB frame, and it collapsed during the test. Uchida et al. (1992) conducted shaking-table and static-moment-loading tests of steel cantilever beam-columns of H-shaped section about their strong axis. All specimens collapsed about their weak axis due to lateral instability. Recently, Schneider and Roeder (1992) tested five moment-resisting steel building frames. They found that frame strength began to deteriorate rapidly on an axial load increase to $0.30P/P_y$.

Most columns in old steel bridges were not designed to absorb energy through cyclic inelastic deformations and therefore are often noncompact sections. Also, unlike the columns tested by Popov et al. (1975), bridge columns are not braced laterally, and depending on their slenderness, lateral torsional buckling is possible under strong axis bending, as observed by Uchida et al. (1992). Considering these factors and the lack of information about the behavior of steel bridge columns, they are conservatively assumed to fail as soon as the capacity delimited by statically derived interaction curve is reached. The stability interaction equations proposed by Duan and Chen (1989) are used for this purpose.

Considering only one iteration for the second-order column transverse moment due to high in-plane stiffness of the continuous bridge deck, the magnified column transverse seismic moment, M_{Ey}, for a specified peak ground acceleration, A_p, is expressed as follows:

$$M_{Ey} = A_p(m_{Ey} + P_D\Delta_c) + A_p^2P_E\Delta_c \tag{8}$$

where m_{Ey}, P_E, and Δ_c = the first-order transverse seismic moment, seismic axial force, and displacement at the col-

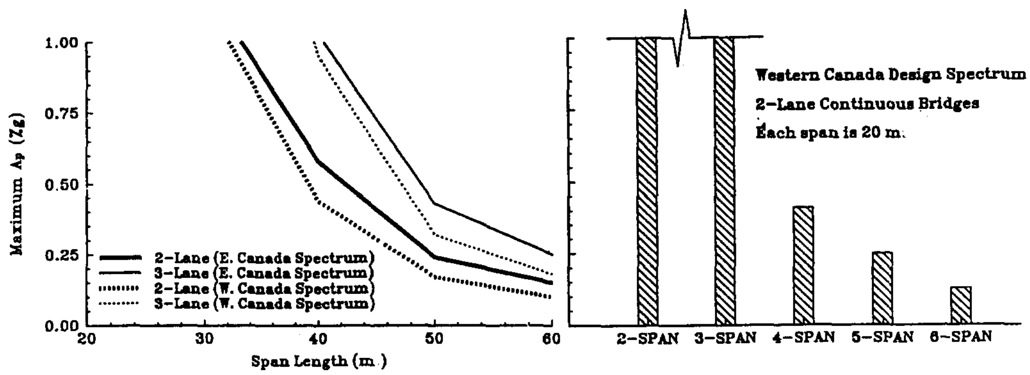

FIG. 9. (a) Maximum Resistible Peak Ground Acceleration as Function of Span Length; (b) Effect of Number of Spans on Seismic Capacity (Linear Elastic Analysis, Undamaged Bearings)

umns' location for a unit peak ground acceleration, respectively; and P_D = axial force in the column due to gravity loading. Using the maximum weak axis column moment considering stability and the magnified seismic moments as a function of A_p, the maximum resistible peak ground acceleration (MRPGA) prior to column failure is calculated for the ranges of spans considered. The results are plotted for 2- and 3-lane bridges as a function of span length in Fig. 9(a). As seen in that figure, the MRPGA is larger for 3-lane continuous bridges. It is noteworthy that as the number of lanes increases, the bridge deck gets wider and stiffer in the transverse direction. Accordingly, the calculated displacements and forces in columns are much smaller in wider bridges, and therefore they can resist larger seismic forces, as seen in Fig. 9(a). On the contrary, increasing span length has a negative impact on the seismic capacity. The lateral displacement of the deck, which is proportional to the fourth power of the span length, is much larger in longer bridges. Therefore, the calculated first- and second-order forces in columns are larger in longer bridges, which, as a result, can only resist smaller seismic forces, as seen in Fig. 9(a). It is noteworthy that 2-lane bridges up to 40 m and 3-lane bridges up to 50 m can survive rather strong earthquakes of up to $0.4g$ peak ground acceleration without any damage to columns provided that bearings are not ruptured at the abutments.

Continuous Bridges with More than Two Spans

Response spectrum analyses of two-lane continuous bridges of 3, 4, 5, and 6 spans of 20 m each in length are conducted using the western Canada design spectrum. In Fig. 9(b), the MRPGA is plotted as a function of number of spans. It is observed that as the number of spans increases, the seismic capacity rapidly decreases. This is mainly due to the increased mass and flexibility of the structure, which subsequently results in higher displacements at the column locations for a given peak ground acceleration. In Fig. 10, the ratio of the MRPGA of multiple-span bridges to that of two-span bridges of identical total length is illustrated. Generally, multiple-span bridges can accommodate peak ground accelerations larger than those obtained for two-span bridges of identical total length. Bridges with an even number of spans are more vulnerable to seismic excitations than those with an odd number of spans. If the number of spans is even, there is always a column located at the midspan where the displacement is high. However, if the number of spans is odd, the columns are located away from the midspan, hence they have less displacement. This effect vanishes as the span length and number of span increases.

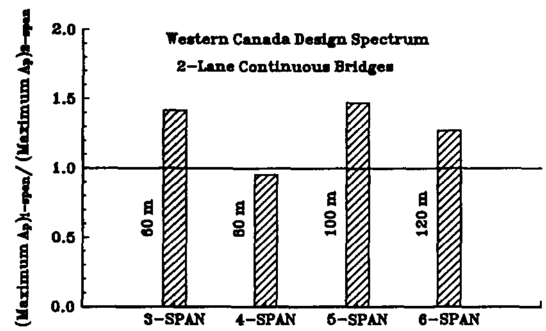

FIG. 10. Ratio of Maximum Resistible Peak Ground Acceleration for Multiple-Span Bridges to that for Two-Span Bridges of Equal Total Length (Linear Elastic Analysis, Undamaged Bearings)

LINEAR ELASTIC ANALYSIS RESULTS—LONGITUDINAL DIRECTION

Expansion Joint Displacements

Assuming that the bridges have sliding bearings, the longitudinal displacements are calculated for the western Canada design spectrum scaled to $0.5g$. These displacements are found to be about 18% of the EJW. Therefore, unless the failure of the fixed bearings takes place, the collision of the deck with the abutment wall is unlikely in the case of sliding bearings. For simply supported bridges only, the longitudinal displacements are also calculated assuming that they have elastomeric bearings for the same western Canada design spectrum scaled to $0.5g$. This time, the displacements are almost three times larger than the EJW for the range of spans considered. Therefore, collision of the deck with the abutment walls may take place when elastomeric bearings are used.

Bearing Forces

Response spectrum analyses of the bridges are conducted in the longitudinal direction using the eastern Canada design spectrum. In Figs. 11(a) and 11(b), the longitudinal bearing force coefficient (LBFC) for 2- and 3-lane simply supported bridges is plotted as a function of span length for various bearing types. The LBFC is obtained by dividing the bearing force due to seismic loading in longitudinal direction by the bridge mass and peak ground acceleration. As seen in these figures, elastomeric bearings have the smallest LBFC and sliding bearings have the largest LBFC compared to other bearing types. The LBFC for elastomeric bearings decreases with span length, but the absolute bearing forces increase as the mass of the bridge also increases. Finally, although the LBFC for 2-lane bridges are larger than those for 3-lane bridges, as illustrated in Fig. 8(b), the actual bearing forces are equal

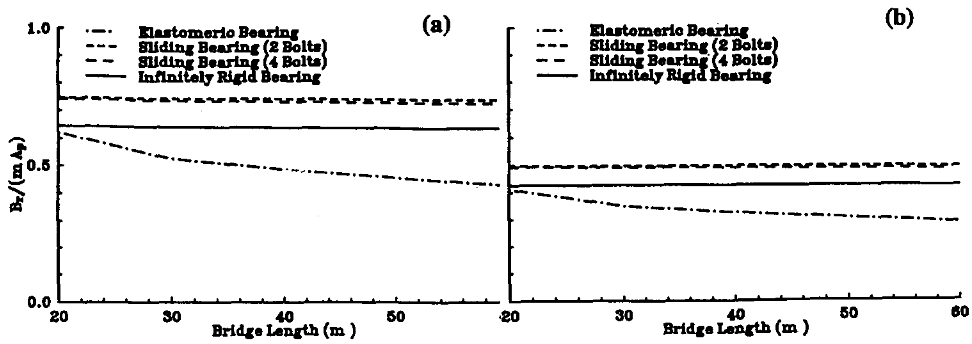

FIG. 11. Effect of Bearing's Stiffness on Longitudinal Bearing Force: (a) 2-Lane Simply Supported Bridges; (b) 3-Lane Simply Supported Bridges (Eastern Canada Design Spectrum)

because each bearing has the same tributary mass. The same trend is observed for continuous bridges.

In summary, when a bridge is subjected to seismic excitation in the longitudinal direction, bearing forces increase with span length. For a given girder spacing, the number of lanes does not have any effect on the bearing forces since for all types of bearings, the total longitudinal force is shared equally by each bearing provided that they have the same stiffness.

Column Response for Continuous Bridges

Assuming that the bearings are damaged but abutments and column footings conserve their integrity, the bridge deck may displace in the longitudinal direction until it collides with the abutment walls. The structure may survive the earthquake if its columns are able to displace by as much as the EJW without failing. The columns may safely overcome that much displacement if the applied moments are less than the maximum strong axis moments that initiate instability.

Assuming that the fixed bearings are severed (or roller bearings are present at both abutments), the magnified moment, $\beta_{mx} M_{x0}$, considering the second-order effects, is obtained as follows:

$$\beta_{mx} M_{x0} = \lim_{n \to \infty} \sum_{i=0}^{n} \left(\frac{P}{k_{cL} h_c} \right)^i h_c k_{cL} \Delta_{x0} = \frac{h_c k_{cL} \Delta_{x0}}{1 - (P/k_{cL} h_c)} \quad (9)$$

where Δ_{x0} = first-order elastic displacement; and k_{cL} and h_c = longitudinal direction stiffness and height of a column, respectively. Knowing that this magnified elastic moment should not exceed the maximum strong axis moment M_{ax} that initiates instability, and using (9), the maximum applicable first-order elastic displacement is expressed as follows:

$$\Delta_{x0} = \{(1/k_{cL} h_c) - [P/(k_{cL} h_c)^2]\} M_{ax} \quad (10)$$

The strong axis moments, stiffnesses, and heights of the continuous steel bridge columns considered in this study are substituted in this equation to obtain the longitudinal displacement as a function of span length. The displacements and EJWs of the bridges are plotted in Fig. 12. As seen in the figure, shorter-span bridges can accommodate larger displacements due to lower axial loads on the columns, which subsequently results in smaller second-order moments. Furthermore, shorter-span bridges have smaller EJWs, which adds an extra safety. Bridges of spans longer than delimited by the intersection of the lines representing the EJW and the calculated maximum permissible first-order displacement may be a problem if the bearings are damaged. However, the friction forces between the disconnected parts may dissipate energy, resulting in less displacement than expected. This will be investigated in a subsequent section.

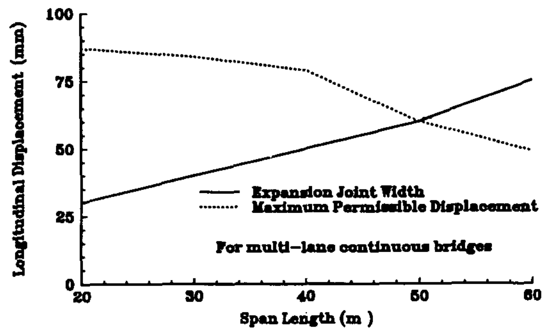

FIG. 12. Maximum Permissible First-Order Longitudinal Displacement Prior to Column Instability as Function of Span Length (Linear Elastic Analysis, Damaged Bearings)

NONLINEAR TIME HISTORY ANALYSES

Sliding of Single-Span Bridges in Transverse Direction

A total of 300 cases are analyzed using the program NEABS to obtain the transverse sliding displacements at the support of the previously described 2- and 3-lane simply supported bridges as a function of span length for various friction coefficient to peak ground acceleration ratios, μ_f/A_p (A_p is expressed as a percentage of g), using the four western U.S. and two eastern Canada earthquakes listed in Table 1. The averaged results from the four western U.S. earthquakes are depicted in Fig. 13, whereas results for eastern Canada earthquakes are shown in Fig. 14. In these figures, the vertical axis is the support sliding displacement (in millimeter per unit peak ground acceleration) and the horizontal axis is the span length. Additionally, keeping the μ_f/A_p ratio constant but changing the magnitude of the peak ground acceleration, 60 other cases are also analyzed to obtain a relationship between the magnitude of the peak ground acceleration and sliding displacement. It is found that for the same μ_f/A_p ratio, the sliding displacement is linearly proportional to the amplitude of the peak ground acceleration. This dependency can be explained by an energy formulation of the sliding problem (Dicleli 1993).

As seen in Fig. 13, for western North America earthquakes, the sliding displacement increases with increasing span length and decreasing μ_f/A_p ratio. Obviously, larger forces are applied on the bridges with longer span. This subsequently results in a higher amount of energy to be dissipated by friction and consequently larger sliding displacements. For the range of spans considered, the attracted seismic forces per unit mass are smaller for wider bridges than those for narrower bridges of the same length and therefore their sliding displacements are smaller, as seen in Figs. 13(a) and 13(b).

As for the eastern North America type of earthquake, almost the same trend as for western U.S. earthquakes is ob-

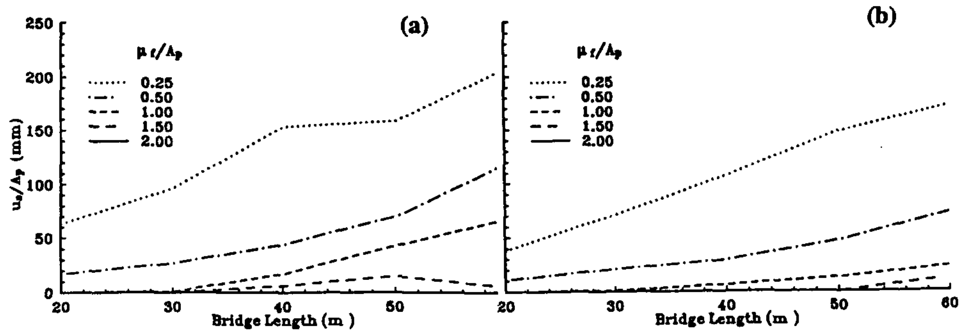

FIG. 13. Transverse Sliding Displacement per Peak Ground Acceleration (%g) for Various Friction Coefficient to Peak Ground Acceleration Ratios (Average of Western North America Earthquakes): (a) 2-Lane Bridges; (b) 3-Lane Bridges

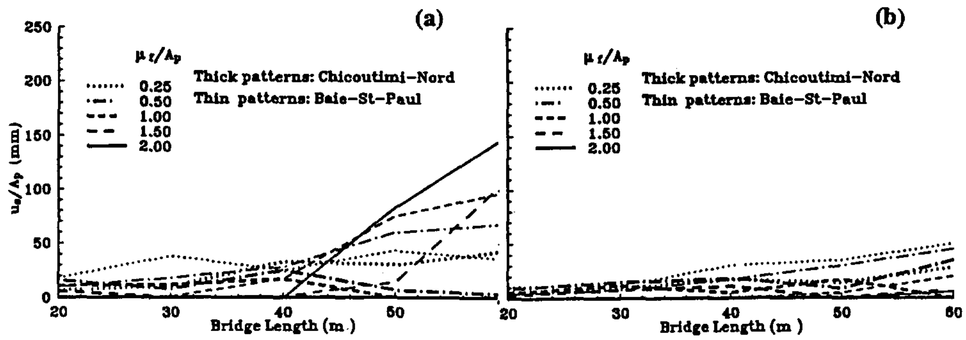

FIG. 14. Transverse Sliding Displacement per Peak Ground Acceleration (%g) for Various Friction Coefficient to Peak Ground Acceleration Ratios (Eastern North America Earthquakes): (a) 2-Lane Bridges; (b) 3-Lane Bridges

served in the case of the Baie-St-Paul earthquake. However, in the case of the Chicoutimi-Nord earthquake, sliding displacements start decreasing for bridges of 50-m span and longer ($T_1 > 0.20$ s). As observed from the earthquake spectrum of Fig. 3, for structures with fundamental periods longer than 0.2 s, the force applied on the structure decreases, thus the sliding displacement decreases.

For the same bridges, the average of sliding displacements obtained using western North America earthquakes are larger than those obtained using eastern North America ones. Eastern North America earthquakes, which have very high A_p/V_p ratios, contain very high frequency pulses. Therefore, once the structure slides, this sliding motion cannot be sustained for a long time because the force applied on the system remains above the threshold of friction only for a short duration. Western North America earthquakes, which have intermediate A_p/V_p ratios, contain frequency pulses longer than those of eastern North America earthquakes. Thus, the sliding motion can be sustained for a longer time. Consequently, ground motions with high A_p/V_p ratios generally produce smaller sliding displacements than those with relatively lower A_p/V_p ratios. This is confirmed by comparison of Figs. 13 and 14.

Sliding of Continuous Bridges in Transverse Direction

Assuming that the anchor bolts in sliding bearings are severed, inelastic dynamic analyses of the continuous bridges are conducted to investigate the effect of sliding on the seismic capacity of the structure. More than 400 analyses were performed using the program NEABS to establish the relationship between the MRPGA and span length for various friction coefficients. This relationship is determined using the average of results obtained from the four western U.S. earthquakes and considering the stability of columns in the transverse direction. The results are presented in Fig. 15. As seen in that figure, the MRPGAs are larger for 3-lane bridges than for 2-lane bridges. Also, bridges with a higher friction coefficient at the bearings can obviously sustain larger earthquakes and have a larger MRPGA. In fact, based on the results obtained, should a friction coefficient of 0.4 exist at the supports, 3-lane continuous bridges of spans shorter than 40 m and 2-lane continuous bridges of spans shorter than 30 m may survive high-intensity earthquakes even if their bearings are damaged, provided that during the earthquake other structural elements preserve their integrity.

Neglecting column instability, sliding displacements of 2- and 3-lane continuous bridges are plotted in Fig. 16 as a function of span length for various friction coefficients, using the S00E component of the 1940 El Centro earthquake scaled to 0.4g. These sliding displacements are nearly twice those of simply supported bridges of identical total length. This behavior is attributable to lower gravity forces acting on the abutment bearings in the continuous bridges, which provide a smaller frictional resistance and thus larger sliding displacements.

Effect of Number of Spans on Transverse Sliding Displacement

To briefly investigate the effect of number of spans on the seismic response, nonlinear dynamic analysis of a continuous bridge with three spans of 20 m each (60 m total end-to-end length) is conducted using the S00E component of the 1940 El Centro earthquake scaled to 0.4g peak ground acceleration, and friction coefficients of 0.2 and 0.4. The resulting sliding displacements are, respectively, 82 and 61 mm. These displacements are compared with those of a continuous bridge with two spans of 30 m each. The two bridges have equal total lengths, but the sliding displacements of the three span bridge are 2.0 and 1.2 times larger than those of the two-span bridge for friction coefficients of 0.2 and 0.4, respectively. Logically, as the number of spans increases the proportion of the total gravity load transferred to the abutments decreases. This results in larger sliding displacements compared

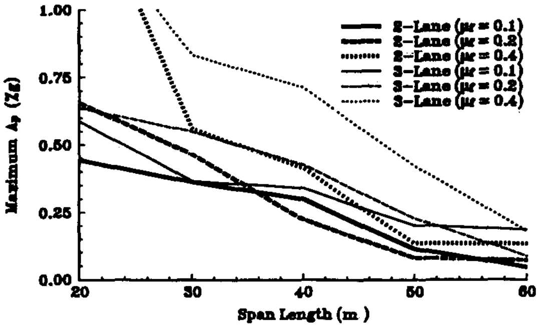

FIG. 15. Maximum Resistible Peak Ground Acceleration as Function of Span Length, Average of Western U.S. Earthquakes (Nonlinear Inelastic Analysis, Damaged Bearings)

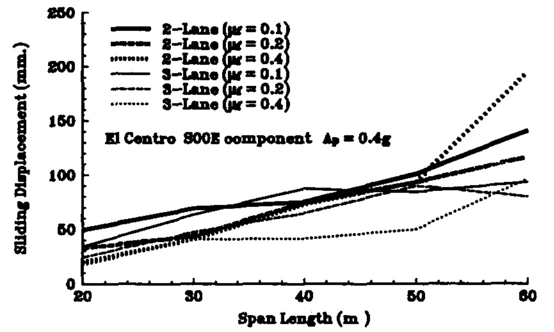

FIG. 16. Sliding Displacement at Support of 2- and 3-Lane Bridges (Nonlinear Inelastic Analysis, Damaged Bearings)

to those of the bridges of equivalent total length but smaller number of spans. These large sliding displacements may reduce the seismic capacity of the structure because the columns may not be capable to accommodate that much displacement. This further illustrates the insignificant contribution of steel columns to the transverse load resistance of bridges.

Sliding of Bridges in Longitudinal Direction

When bridges slide in the longitudinal direction, friction forces at the fixed and expansion bearings are produced. These friction forces may dissipate energy, limiting the magnitude of displacements to acceptable values, even for a high-intensity earthquake. To illustrate this, sliding displacement of a continuous bridge with two spans of 60 m each is calculated considering and ignoring friction. As seen in Fig. 12, a longitudinal displacement of 49 mm is actually required to fail the column due to instability. Considering a coefficient of friction of 0.4. and using the average of the results obtained from western U.S. earthquakes, a peak ground acceleration of approximately 0.30*g* is needed to produce this displacement. If friction was ignored, a peak ground acceleration of 0.30*g* would produce a displacement of 335 mm if unrestrained by abutments. The columns alone would be grossly inadequate in resisting lateral forces in this case. This shows that in some cases, dissipation of energy through friction at the supports could be an effective way of reducing the displacements at the column's location and therefore preventing premature failure of the structure.

CONCLUSIONS

Response of single-span simply supported and continuous bridges to seismic excitations was investigated considering two possible cases. First, seismically induced damage to bearings was assumed to be undesirable and linear elastic analyses were conducted to investigate the response of bearings and columns. As a second possible case, assuming that seismically induced damage to bearings is stable and acceptable, nonlinear inelastic analyses were conducted to investigate the effect of sliding on the seismic response and capacity of bridges. Conclusions can be grouped accordingly.

Elastic Spectral Analysis Results

Bearing forces due to seismic excitation in both transverse and longitudinal directions are proportional to the mass of the bridge and thus to the span length. Their magnitude is independent of the number of lanes when subjected to seismic excitation in the longitudinal direction. However, when subjected to seismic excitation in the transverse direction, their magnitude decreases as the number of lanes increases. Bearings with higher longitudinal stiffness and closer to the edge of the bridge deck attract forces larger than those in other bearings. However, bearings with negligible longitudinal stiffness attract almost equal forces regardless of the number of lanes of bridges. If bearings in continuous bridges are not damaged, 2-lane bridges up to 40-m span and 3-lane bridges up to 50-m span are found capable to survive earthquakes with 0.4*g* peak ground acceleration without any damage to columns.

As the number of spans of continuous bridges increases, seismic capacity decreases. However, multiple-span continuous bridges can generally accommodate larger peak ground acceleration than two-span continuous bridges of equal total length. It is also found that bridges with an even number of spans are slightly more vulnerable to seismic excitations than those with an odd number of spans. This can be explained by the relative insignificant contribution of steel columns to resist loads transversely applied to the bridge's deck; the resistance of a column is therefore dictated by its displacement compatibility with the deck, and consequently by its longitudinal location along the bridge.

Inelastic Time History Analyses Results

When bearings are stable and can slide freely once their anchors are ruptured by earthquakes, damage to the bearings may be acceptable. Nonlinear inelastic analyses were conducted to investigate the magnitudes of sliding displacements of bridges as a function of various parameters.

For the same friction coefficient to peak ground acceleration ratio, μ_f/A_p, the sliding displacement of a structural system is linearly proportional to the peak ground acceleration. The sliding displacement increases with increasing span length and decreasing μ_f/A_p ratio, but decreases as the bridge gets wider.

Ground motions with high frequency content, or high A_p/V_p ratios may produce smaller sliding displacements than ground motions with relatively lower A_p/V_p ratios. Hence, steel bridges located in eastern North America will be subjected to smaller sliding displacements than their identical counterparts in western North America. Nevertheless, the displacements are not considerable for the earthquakes and range of spans of simply supported bridges considered. According to these results, few simply supported bridges should suffer significant damage due to bearing displacements.

Studies of the effect of some parameters on the seismic response of continuous bridges demonstrate that the sliding displacement of bridges with larger number of spans is greater than that of bridges with identical end-to-end length but smaller number of spans. The magnitude of the friction coefficient at the bearings has a considerable impact on the seismic capacity of these bridges; generally, as it increases, the maximum sliding displacement of the bridge decreases. This subsequently produces smaller displacements at the column locations, and even more severe earthquakes are needed to fail the columns. For continuous bridges with sliding bearings, it is found that 2- and 3-lane bridges of spans shorter than 30 and 40 m, respectively, may survive earthquakes of 0.35g peak ground acceleration even if the bearings are damaged, provided that they have adequate seat width in the transverse direction.

As a general trend, regardless of whether the bearings are damaged or undamaged, the peak ground acceleration required to damage the continuous bridges is found to be larger for 3-lane than for 2-lane bridges. For expediency, results obtained in this study for the latter case can always be used to obtain a conservative but rapid quantitative and qualitative assessment of the seismic behavior and capacity of continuous steel bridges considering both damaged and undamaged bearing conditions. These can be useful to determine which steel bridges may require seismic retrofitting, particularly in regions of moderate and low seismicity. Additional research is also under way to develop analytical expressions more accurately quantifying this behavior to be used in an automated quantitative systematic rapid seismic evaluation procedure.

ACKNOWLEDGMENTS

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