Seismic performance of multispan simply supported slab-on-girder steel highway bridges

Murat Dicleli and Michel Bruneau

Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada, K1N 6N5
(Received July 1993; revised version accepted November 1993)

The seismic response of existing multispan simply supported slab-on-girder steel highway bridges, never designed to resist earthquakes, is studied. Elastic response spectrum analyses are conducted for bridges with different bearing stiffness, span length and number of spans. It is found that the response of two-span simply supported bridges is highly dependent on the stiffness of fixed bearings on the abutments, but this effect vanishes as the number of spans increases. The transverse direction seismic capacity of bridges having more than two spans is not a function of the number of spans. These bridges may be damaged by earthquakes having peak accelerations less than 0.20 g. However, bridges with identical end-to-end length but subdivided into a smaller number of spans are found to be more vulnerable to seismic excitations than those with larger number of spans. Increasing span length is also found to have a negative impact on the seismic capacity of these bridges. Additionally, analytical expressions to calculate the minimum required seat width are developed.

Keywords: steel, multispan, simply supported bridges, seismic excitation, bearing stiffness, seismic capacity

Multispan simply supported bridges are the type of bridge which have suffered the most damage in past earthquakes. Considerable research has been conducted to identify the structural components and parameters which significantly affect the seismic response of reinforced concrete multispan simply supported bridges. For example, the importance of longitudinal restrainer ties and the shear keys at the expansion joints[1–4], the effect of impacting between the adjacent bridge sections on the superstructure-substructure connections[1], the effect of skewness on the reinforced concrete columns[5], adequacy of lateral reinforcement, end anchorage and splice length in reinforced concrete columns[6], nonuniform distribution of column stiffnesses along the bridge and disproportionate flexural and shear strengths of the columns[7], and the shear-failure vulnerability of bridge piers[7–11] have received considerable attention. However, information on the seismic behaviour of multispan simply supported steel bridges remains scarce in the literature. Some research findings exist on the seismic performance of steel box-sections often used in Japan for bridge piers[12], and provisions for the seismic design of steel bridges have been proposed by critically extrapolating and adding to the requirements applicable to steel buildings[13], but to this date no standard specification exists in North America to provide clear guidance on this topic.

0141-0296/95/010004–11

There are tens of thousands of steel highway bridges in North America. Most of them have never been designed to resist earthquakes and are located in what are now considered to be seismic regions. In this paper, the seismic performance of existing multispan simply supported slab-on girder steel highway bridges, designed without the consideration of seismic forces, is studied. Earthquake excitations along both the transverse and longitudinal bridge directions are considered separately. If the abutments, bearings and foundations can survive earthquakes without any damage, the seismic capacity of these bridges is governed by the capacity of the columns in both directions or the opening of the expansion joints until a deck falls off its support (in this paper, unless indicated otherwise, the word 'deck' is used as an inclusive term for the combination of slab and girders). Linear elastic response spectrum analyses in the transverse direction are conducted to find the bearings transverse seismically induced forces and column moments. It is noteworthy that in 85% of the cases analysed, either the columns reached their ultimate capacity prior to impact between the two decks, or the peak accelerations required to produce collision were very high. Therefore, a nonlinear inelastic dynamic analysis that considers the effect of collision was not required in the transverse direction. In the longitudinal direction, the effect of impacting between the adjacent bridge sections on the seismic performance of multispan simply supported bridges has been investigated[1,2,4,14] and is therefore not considered in this study. However, linear elastic analyses are conducted to investigate other factors affecting the seismic capacity of multispan simply supported steel bridges in that direction. Additionally, analytical expressions are derived to check the adequacy of seat width in existing multispan simply supported steel highway bridges. In all cases abutments and foundations deformation and/or damage as well as soil–structure interaction are beyond the scope of this study.

Properties of the 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, two-lane and three-lane bridges with two spans of each 20, 30, 40, 50 and 60 m were designed in compliance with the 1961 edition of the American Association of State Highways Officials (AASHO) code[15]. The two- and three-lane bridges have, respectively, 8 m and 12 m widths and girders spaced at 2 m intervals. As shown in *Figure 1*, the left span of these bridges is supported at one end by fixed bearings resting on the abutment and at the other end by expansion bearings resting on the columns distributed across the deck. In the subsequent sections this span will be referred to as the abutment-fixed deck. The right span is supported at one end by fixed bearings resting on the same columns which support the left span and at the other end by expansion bearings resting on the abutment; it will be referred to as the column-fixed deck hereafter. Since the bridge spans are both simply supported on the columns, to obtain a stable structure, the columns' ends must be rigidly fixed to the foundation. Additionally, some two-lane bridges having from 3 to 5 spans of 40 m each are also studied. In all cases studied, a 1 m deck-slab overhang is assumed to exist at both sides of the bridges. The steel columns are assumed to be 6 m high, as is commonly found in many North American highway bridges for clearance reasons, and are assumed to be oriented such that they work in strong axis bending under forces applied parallel to the longitudinal direction of the bridge. The deck-slab thickness is 200 mm.

The bridges are assumed to have sliding-bearings of the type often found in short- to medium-span old bridges. The fixed type of sliding-bearings is shown in *Figure 2*. The expansion type is nearly identical but without the longitudinal stopper bars. Based on the recommendations of AASHO, two anchor bolts of 32 mm diameter and four anchor bolts of 38 mm diameter per bearing are taken as the lower and upper bounds, respectively, of possible anchorage for the span ranges considered. Considering an average size bearing bar and bearing plates, and using the model described later to assess the stiffness of bearings, the longitudinal stiffness is calculated as 400 000 kN m^{-1} for the bearing with two bolts, and 800 000 kN m^{-1} for the other bearing.

Other than the above bearing stiffnesses, the theoretical cases of bearing-sets with zero and infinite rotational stiffness about a vertical axis (i.e. an axis perpendicular to the plane of the bridge deck) are also considered. Infinite rotational stiffness occurs when the fixed bearings are infinitely rigid in the longitudinal direction. Zero rotational stiffness occurs when the fixed bearings have negligible longitudinal stiffness or when the bearings on the abutments are damaged. In all the cases the transverse stiffness is assumed to be infinitely rigid.

The mean plus one standard deviation (MP1SD) spectrum of four western USA earthquakes for 5% damping is used in the linear response spectrum analyses[16]. These earthquakes are: (i) El Centro, 1940, E00S; (ii) Taft, 1952, S69E; (iii) San Fernando (Pacoima Dam), 1971, S16E; and (iv) Parkfield (Cholame Shand. 2), 1966, N65E. For comparison, this spectrum and the Uniform Building Code (UBC) spectrum[17] are illustrated in *Figure 3*. The vertical axis in this figure is, β, which is the ratio of the pseudospectral acceleration, S_a, to the peak ground acceleration A_p.

Modelling

Linear elastic response spectrum analyses are conducted. Full composite action between the deck-slab and steel beams is assumed. In the transverse direction, each span rotates separately about a vertical axis perpendicular to the bridge deck, therefore, continuity of the existing multispan simply supported bridges studied herein is provided only by the simple connections of the decks to the abutments and columns. For this reason, in the simplified structural model shown in *Figure 4*, the two simply supported spans of the bridge are separated, introducing a gap equal to the width of the expansion joint between the abutment-fixed

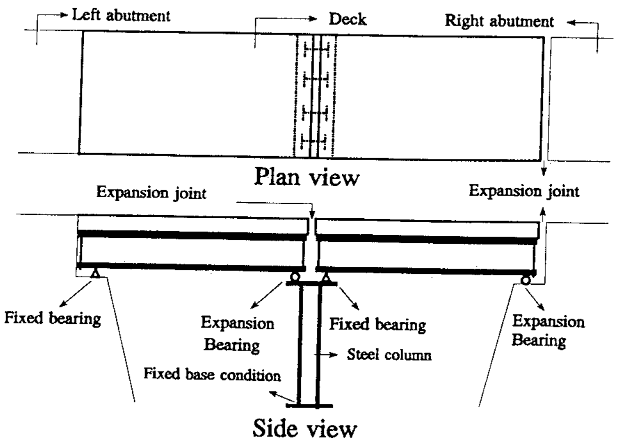

Figure 1 Typical two-span simply supported bridge

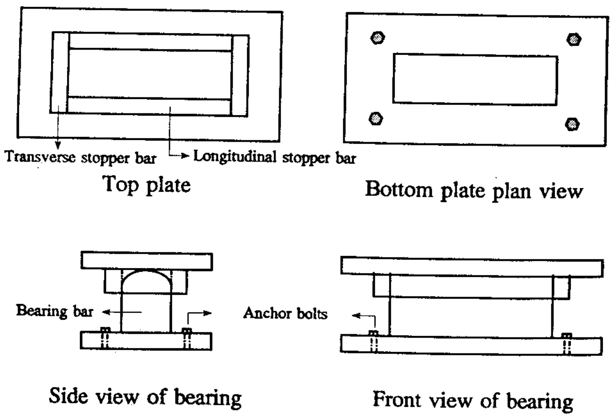

Figure 2 Typical fixed sliding bearing

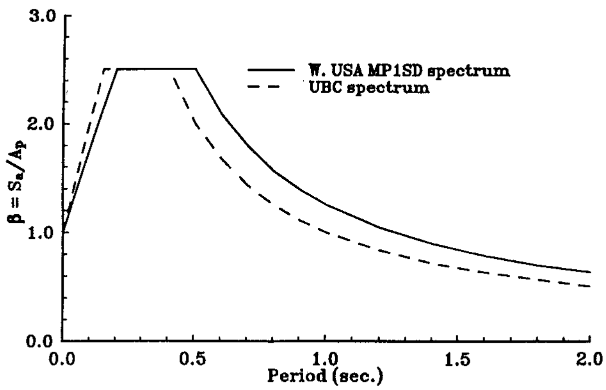

Figure 3 UBC spectrum and MP1SD spectrum of western USA earthquakes for rock or stiff soil and 5% damping

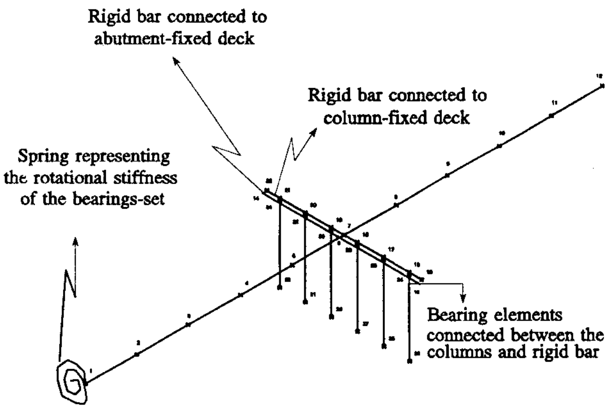

Figure 4 Linear elastic model of two-span simply supported bridges

and column-fixed decks. Then, each deck is divided into segments, and modelled as a 3D beam element. A transverse rigid bar of length equal to the deck width is connected to the tip of the beam element at the end of the column-fixed deck where the columns are located and oriented in the transverse direction. This rigid bar is used to model the interaction between the translation of the columns in the transverse and longitudinal direction, and the translation as well as the rotation of the column-fixed deck about a vertical axis. First, the bearing elements are connected to this rigid bar at the column locations and moments about the longitudinal and transverse global axes are locally released at the upper end of each bearing element. The bridge columns are then connected to these bearings. To measure the relative longitudinal displacement between the adjacent decks at the expansion joint, another transverse rigid bar is also connected to the tip of the beam element at the end of the abutment-fixed deck. For compatibility, the transverse displacement and torsional rotation about an axis parallel to the bridge, are constrained to be identical where both the abutment-fixed and column-fixed decks meet. The translational degree-of-freedom of the node at the end of the abutment-fixed deck is set free in the longitudinal direction to allow the expansion of the deck.

A procedure similar to that above for a two-span simply supported bridge is followed when modelling the multispan simply supported bridges considered in this study. In the models of these bridges, rigid bars are used at each expansion joint.

The stiffness of each fixed bearing at the abutment is calculated using the following equation[18]

$$k_{bL} = \frac{1}{\dfrac{1}{\dfrac{3\,E\,I_{bb}}{h_{bb}^3}} + \dfrac{1}{\dfrac{E}{h_{bb}} \sum_{i=1}^{n_{ab}} \left(\dfrac{l_p\,A_{ab}}{l_{ab}}\right)_i}} \tag{1}$$

where, E is the modulus of elasticity of steel, I_{bb} is the 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} is the height of the bearing bar, l_p is the length between the anchor bolt and tip of the bottom plate, A_{ab} and l_{ab} are, respectively, the area and the length of the anchor bolt, and n_{ab} is the number of anchor bolts. In the transverse direction sliding-bearings are very rigid, hence the bearing stiffness, k_{bT}, in this direction need not be calculated. In the case of sliding-bearings on the columns, their longitudinal stiffness is calculated considering only the deformation of the bearing bar and their deformation in the transverse direction is neglected.

By using this stiffness for each bearing located under each girder, the longitudinal effect of the fixed 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. The stiffness K_{bL} of the translational spring is the sum of the stiffnesses of all the bearings and the stiffness $K_{b\theta}$ of the rotational spring is expressed as[18]

$$K_{b\theta} = \sum_{i=1}^{n_b} k_{b_i}\, l_{b_i}^2 \tag{2}$$

where n_b is the number of bearings and l_{bi} is the distance of bearing i to the centreline of the bridge deck.

Effect of bearing stiffness on the stiffness of the structure

The rotational stiffness on the fixed bearings-set and the stiffness of the bridge deck contributes to the overall stiffness of the abutment-fixed deck. The translation and rotation of the column-fixed deck at the columns' location is resisted by the rotational and translational stiffness of the columns-set (obtained by modelling each column and its bearing above as springs in series) and the stiffness of the deck itself. The transverse stiffness, K_{TT}, of a two-span bridge is the sum of the stiffnesses of the abutment-fixed and column-fixed decks. This stiffness is highly dependent on the stiffness of the bearings. The stiffness of the abutment-fixed deck can contribute to the overall stiffness only if a rotational resistance is developed by the fixed bearings-set at the abutments. With a null rotational stiffness, that part of the structure behaves like a mechanism. Consequently, the rotational stiffnesses of the fixed bearings-set located on the abutment also considerably affects the period, of two-span simply supported bridges. However, if the number of spans is more than two, the effect of bearing stiffness becomes more localized, and its importance eventually vanishes with an increasing number of spans. For example, in a three-span simply supported bridge, the stiffness of bearings at the abutment affects the displacement of the abutment-fixed deck, and therefore, only the displacement of the first nearest columns-set. The relative transverse displacement of the other columns-set will not be affected.

It is noteworthy that the total stiffness of the structure is largely affected by the transverse stiffness of the columns which is partly a function of the stiffness of the bearings located on the columns. For example, bearings with small stiffness, such as elastomeric bearings, may affect the overall stiffness of the structure considerably, but, other less flexible bearings do not. In that latter case, the total stiffness of the structure is mostly affected by the stiffness of the columns, especially in bridges with more than two spans.

Seismic behaviour of steel bridge columns

A review of the literature on the cyclic behaviour of steel members revealed that such reseach has concentrated solely on building elements and only a few researchers have addressed the weak column strong beam (WCSB) behaviour more typical of steel bridges. Popov *et al.*[19] tested two different compact W-shape sections braced to prevent lateral buckling about their weak axis. Results showed that the cyclic behaviour 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 the interstorey drift. Sudden failure was observed in specimens having P/P_y ratios larger than 0.5. Later, Takanashi and Ohi[20] performed a shaking table test on a small-scale three-storey WCSB frame, and it collapsed on the shaking table during the test. Uchida *et al.*[21] 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[22], tested five moment resisting steel building frames. They found that frame strength began to deteriorate rapidly when the axial load increased to 0.30 P/P_y.

Bridges generally have much longer columns than those commonly used in buildings. The slenderness ratios of bridge columns are also much higher. Furthermore, most of the columns in old steel bridges were not designed to absorb energy through cyclic inelastic deformations and therefore are often noncompact sections. Finally, unlike the columns tested by Popov *et al.*[19], 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.*[21]. Considering the above factors and the lack of information about the behaviour of steel bridge columns, in this study, 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[23] are used for this purpose.

Derivation of moment magnification factors for columns

Consider a two-span simply supported bridge. When it is subjected to seismic excitation in the transverse direction, the columns displace in both transverse and longitudinal directions. The longitudinal displacement of the columns is due to the rigid body rotation of the column-fixed deck about a vertical axis and is proportional to the distance of the columns to the centreline of the bridge deck. Consequently, first- and second-order moments are generated in both directions. To obtain the transverse direction magnified moments, the incremental shear forces resulting from the transverse direction second-order moments are summed up and a new incremental transverse loading is obtained. The shear forces produced from longitudinal direction second-order moments are multiplied by the distance, d_{cj}, of each column to the centreline of the bridge deck and an inplane incremental torsional moment, acting on the expansion joint, is obtained. Rigid body rotation is assumed for the column-fixed deck and the incremental transverse load and torsional moment is then applied on the structure at the columns-set location to obtain the new incremental displacements.

Following the above procedure for each step and applying the rules of induction, the moment magnification factors in the transverse and longitudinal directions due to earthquake excitation in the transverse direction are obtained as

$$\beta_{my} = 1 + \frac{P}{k_{cT}\,h_c}\,\frac{1}{1-\lambda} \tag{3}$$

$$\beta_{mxy} = 1 + \frac{P}{k_{cL}\,h_c}\,\frac{1}{1-\lambda} \tag{4}$$

where

$$\lambda = \frac{P}{K_{TT}\,h_c}\left(n_c + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2}\right) \tag{5}$$

In the above equations P is the axial load on the column, n_c, h_c, k_{cL} and k_{cT} are the number, height, longitudinal and transverse stiffness of the columns, respectively, and L is the length of the column-fixed deck. Although these equations are derived for two-span simply supported bridges, they can equally be used to obtain the magnified elastic sway moment of multispan simply supported bridge columns. In fact, the applicability of these equations to bridges with more than two spans is due to the discontinuous nature of the structure. It is noteworthy that the transverse stiffness, K_{TT}, may change depending on the location of the columns-set. If the moment magnification factors for columns which are not adjacent to the abutment-fixed deck is required then only the stiffness of the column-fixed deck contributes to the transverse stiffness. A typical derivation of K_{TT} is presented elsewhere[18].

Analyses results – transverse direction excitation

Fundamental period

The period of a two-span bridge is highly dependent on the stiffness of the bearings used. For example, the transverse direction fundamental period of two-lane bridges with two spans ranges between 1.7 and 2.2 s when bearings with zero rotational stiffness are used, and between 0.18 and 1.13 s when bearings with infinite rotational stiffness are used. The transverse direction fundamental periods of two- and

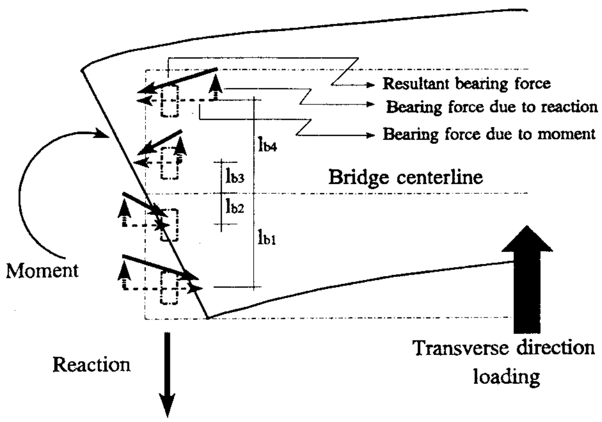

Figure 5 Bearing forces due to loading in transverse direction

three-lane bridges are also very close when bearings at the abutments have zero rotational stiffness, but the difference again becomes larger as the bearings' stiffness increases.

Bearing forces at the abutments

Response spectrum analyses of five two-lane and five three-lane simply supported bridges are conducted in the transverse direction for four different bearing stiffnesses. As illustrated in *Figure 5*, the forces, B_r, in a fixed type of sliding-bearings at the abutments are calculated first by dividing the total transverse reaction force by the number of bearings, then, by summing vectorially this force and the longitudinal force produced by the resistance to rotation at the bearings. As seen in *Figure 5*, bearings farthest to the bridge centreline attract larger seismic forces, and are therefore the most critical bearings. Obviously, the expansion type of sliding-bearings does not attract as much force as the fixed type since the bridge deck can freely rotate about a vertical axis. Therefore, in that case, the bearing forces in the transverse direction are calculated simply by dividing the total force on the bridge abutment by the number of bearings.

The transverse bearing coefficient (TBFC) for various bearing types is plotted as a function of span length in *Figure 6*. TBFC is a dimensionless parameter 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 acceleration of the ground motion. As seen in *Figure 6*, in the case of bearings-set with infinite rotational stiffness, the TBFC and bearing forces increase with span length for spans up to 40 m, beyond this range the TBFC decreases, but the absolute bearing forces slightly increases. However, the forces in bearings-set with no rotational stiffness are nearly zero. This is largely attributable to the bridge's behaviour, since the decks can rotate freely about their supports at the abutments, and columns are flexible, the bridge acts as a mechanism.

In *Figure 7* the ratio of bearing forces of two- or three-lane bridges is plotted as a function of span length. It is observed that for bearings-set with zero rotational stiffness this ratio is almost constant and approximately equal to 1. However, for other types of bearings, the ratio is larger than 1 for short span bridges and gradually decreases with increasing span length. When sliding-bearings are used, the bearing forces in two-lane bridges are larger than those in three-lane bridges up to 40 m span and then they become smaller with increasing span length, but the difference is not very large. In the case of bearings with infinite rotational stiffness, the forces in the bearings of two-lane bridges are always larger than those in three-lane bridges for the range of spans considered but the difference becomes smaller as the span length increases.

Columns' response

Response spectrum analyses of the two-span bridges are conducted to obtain the transverse and longitudinal direction seismic moments for a unit peak ground acceleration. These moments are first magnified and then substituted into the biaxial stability interaction equation proposed by Duan and Chen[23], to obtain the maximum resistible peak ground accelerations (MRPGA) that can be reached prior to column instability. The results are plotted as a function of span length in *Figure 8*. The presented MRPGAs are those corresponding to the columns which are closer to the edge of the deck and are the most vulnerable ones; incidentally, for obvious reasons, this would be an even bigger problem in skewed bridges[5]. As can be seen, the MRPGAs for three-lane bridges are larger than those for two-lane bridges when bearings that develop rotational resistance at the supports are used. It is noteworthy that the contribution of the deck stiffness to the overall transverse stiffness of two-span simply supported bridges is more effective when bearings having larger stiffness are used at the abutments, resulting in less lateral displacements of the columns. Consequently,

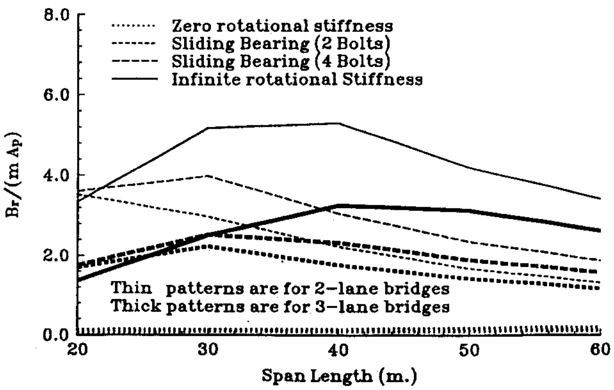

Figure 6 TBFC for two-span simply supported bridges

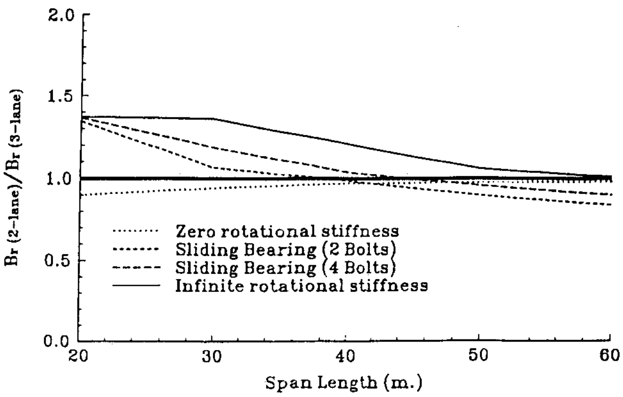

Figure 7 Ratio of transverse bearing forces of two-lane and three-lane bridges for different bearings as a function of span length

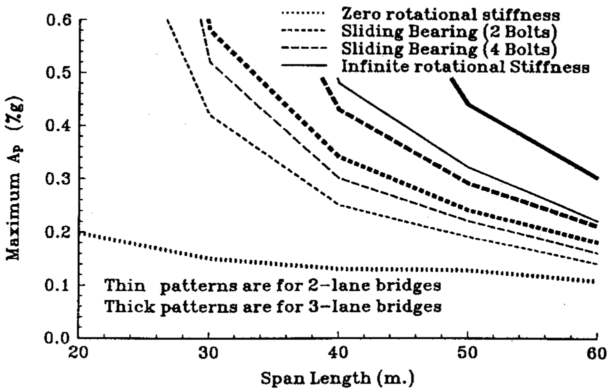

Figure 8 Maximum resistible peak ground acceleration as a function of span length

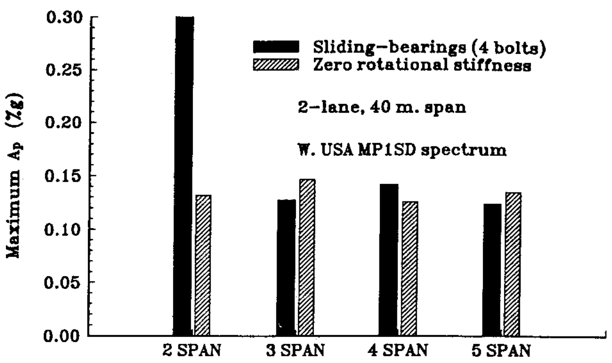

Figure 9 Effect of number of spans on seismic capacity

the difference between the MRPGA of two- and three-lane bridges becomes larger as the rotational stiffness of the bearings-set increases. Note that when the bearings-set have zero rotational stiffness, the stiffness of the abutment-fixed deck does not contribute to the lateral stiffness of the structure. Therefore, the MRPGAs are identical for two- and three-lane bridges and very low.

As observed in *Figure 8*, increasing span length has a negative impact on the seismic capacity. Although longer bridges may attract smaller spectral accelerations due to their long periods, their displacement at the columns location is very large. Therefore, the calculated first- and second-order forces in columns become dominant in longer bridges, resulting in smaller resistance to seismic forces.

Bridges with more than two spans

Two-lane simply supported bridges with 2, 3, 4 and 5 spans are considered. Each span is assumed to be 40 m long. The bridges are assumed to have sliding-bearings with four bolts or bearings-set of zero rotational stiffness. For each type of bearings, response spectrum analyses of the bridges are conducted using the MP1SD spectrum of western USA earthquakes.

From these response spectrum analyses, the transverse and longitudinal direction seismic moments of the columns are obtained. It is found that, in the case of bridges with sliding-bearings, moments in the columns adjacent to the abutment-fixed deck are lower than those of the other columns. For each bridge, only the columns with largest seismic moments are considered. Using these moments and their magnification factors, the MRPGA considering column instability is obtained for each bridge. The results are illustrated in *Figure 9*. As seen in *Figure 9*, the two-span bridge has a relatively high capacity when sliding-bearings are used. However, the capacity rapidly drops by more than half when bearings-set with zero rotational stiffness are used. The effect of bearing stiffness on the seismic capacity practically disappears for bridges having more than two spans. The seismic capacity of these bridges is almost identical. In fact, the seismic capacity of a multispan simply supported bridge, regardless of the bearing stiffness, is nearly equal to that of a two-span bridge with bearings-set of zero rotational stiffness and identical individual span lengths.

The maximum transverse direction displacements of bridges with different number of spans were also compared. It was found that, for bridges with bearings-set of zero rotational stiffness, the variation of maximum transverse displacement as a function of number of spans is not significant. For bridges with sliding-bearings, although the maximum transverse displacement of the two-span bridge is much smaller than the others, the displacements of bridges having a larger number of spans are also comparable. Empirically, it was found that the maximum transverse displacement of a bridge with n_s spans, supported by sliding bearings, is close to that of a bridge with $n_s - 1$ spans supported by bearings-set with zero rotational stiffness.

Additionally, the response of a three-span, simply supported bridge with 120 m end-to-end length is compared to that of a two-span simply supported bridge of identical end-to-end length. It has been demonstrated earlier that bearing stiffness has a considerable effect on the response of two-span bridges. Therefore, the bridges are assumed to have bearings-set with zero rotational stiffness. It is found that the seismic capacity of a three-span bridge is 35% higher than that of a two-span bridge of identical end-to-end length. This is a result of several factors. Firstly, since the length of each individual span is longer in bridges with smaller number of spans, larger forces due to gravity loading are produced in the superstructure components. This results in larger member sizes. Therefore, the total mass of bridges with a smaller number of spans is larger. This produces higher seismic forces in the structure. Secondly, the axial forces due to gravity loading in the columns of bridges with fewer spans are larger, therefore its negative impact on the flexural capacity is more significant. Thirdly, the stiffness of the columns in bridges with fewer spans is larger, therefore greater seismic moments are attracted. And finally, the number of columns-set in bridges with fewer spans is less than that of bridges with a larger number of spans. Consequently, their lateral resistance is not as great. This results in larger transverse displacements and larger first- and second-order seismic moments in the columns.

It is noteworthy that the MRPGAs obtained from the analyses are very small regardless of the number of spans. These results are not surprising. Imbsen and Penzien[4] studied the seismic response of the Fields Landing Overhead which suffered major damage during the Trinidad-Offshore, California Earthquake of 1980. They estimated that a peak ground acceleration of about 0.1 to 0.2g occurred at the bridge site during the earthquake. Furthermore, Longinow *et al.*[24] categorized multispan simply supported bridges as unsound i.e. certain to fail during earthquakes, if not retrofitted. The results of the current study agree with these findings, and provide a quantitative measurement of this vulnerability for typical slab-on-girder steel highway bridges.

Analyses results – longitudinal direction excitation

In the longitudinal direction, the abutment-fixed deck is very rigid. Therefore, its displacement is negligible. However, any column-fixed deck is very flexible since the longitudinal movement of the deck is resisted only by the steel columns. Consequently, these displacements can be very high but the decks may collide before the displacements reach the point at which the columns would fail. To investigate this, the maximum longitudinal displacement, Δ_{cL}, that the columns can accommodate before failure is calculated by dividing the maximum resistible strong axis moment by the column's height and its longitudinal stiffness. These displacements and the expansion joint widths (EJW) calculated considering a temperature difference of 50°C, are plotted in *Figure 10* as a function of span length for the bridges considered in this study. It is observed that the columns can easily sustain longitudinal displacements much larger than the EJW prior to failure, and that the impact of the decks is a more important problem for longitudinal seismic excitation.

Neglecting the effect of travelling seismic waves for the range of spans considered, when multispan simply supported bridges with identical column-fixed decks are subjected to longitudinal seismic excitation, the displacements of all the column-fixed decks are in phase until there is a collision. The column displacements due to seismic forces are magnified by second-order effects. For impact to occur this magnified displacement should be equal to the EJW. Knowing this, the minimum peak ground acceleration for first impacting to occur expressed as a fraction of gravitational acceleration, g, is

$$A_p = \frac{n_c\, k_{cL}\, EJW}{\beta\, m_D\, g}\left(1 - \frac{P}{k_{cL}\, h_c}\right) \qquad (6)$$

where, m_D is the mass of the column-fixed deck and other terms are as defined earlier. Using the above equation, the minimum peak ground acceleration required to produce collision is calculated for the bridges considered in this study. It is found that the peak ground accelerations needed to produce collision increase with span length but are all less than 0.1g.

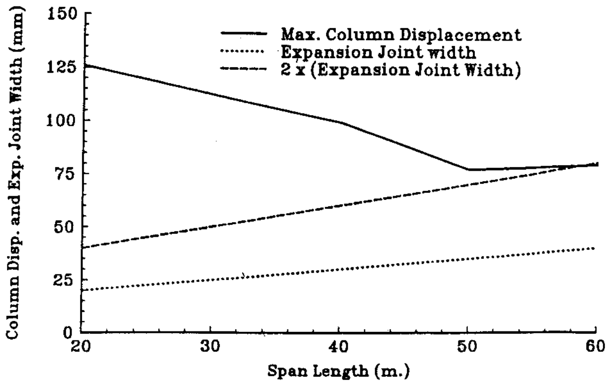

Figure 10 Comparison of maximum deck displacement prior to columns' instability failure with expansion joint width

Response of bridges with two spans

For the two-span simply supported bridges considered in this study, the movement of the column-fixed deck in the longitudinal direction is restricted by the width of the expansion joint, which is roughly a maximum of 4–5 cm for the ranges of spans considered. When collisions occur, the total seismic load of the bridge deck is transmitted to the foundation primarily through the impact between the deck and abutment, with the columns carrying only a small percentage of the total load[25]. However, it has been reported that in reinforced concrete bridges, impacting between two adjacent sections of the superstructure causes high shear forces and possibly failure in the bearings located above the columns[4,14]. Such failure could make the structural system unstable since the superstructure becomes disconnected from the columns and the simply supported decks may fall off their supports if the seat width is not adequate[14]. Although it is unclear whether bridges having steel columns of lower reaction mass would suffer similarly high impact-induced bearing shear forces, it would be safe to conservatively assume they would while awaiting further evidence.

If the fixed bearings at the abutment are severed, each individual deck can potentially have a maximum displacement equal to the sum of the EJWs. There are two expansion joints in a two-span simply supported bridge and the sum of their widths can add up to a maximum of 8–10 cm for the range of spans considered. In *Figure 10* the sum of the EJWs is also plotted in addition to previously presented information. It is observed that steel columns in the cases studied can accommodate displacements as large as twice the EJW. Therefore, two-span bridges are considered to be safe under longitudinal direction seismic excitation if: other components of the bridge are not damaged, the seat width at the supports is adequate, and the deformations of the abutments are negligible.

Response of bridges with more than two spans

As the number of spans increases, the sum of the EJWs also increases. This sum may be larger than the maximum displacement that the steel columns can accommodate before failure. In this case the safety of the columns in the longitudinal direction cannot be ensured. In fact, expansion joint openings (EJO) equal to the sum of the total EJWs can occur[4,14]. However, if the sum of the EJWs is less than the maximum column displacements, then the columns are expected to survive seismic excitations in the longitudinal direction. Knowing this, the number of spans for which columns are considered to be safe are plotted in *Figure 11* as a function of span length. As seen in Figure 11, the columns in multispan simply supported bridges of 20 m spans with up to six spans are considered to be safe, while 60 m bridges with only a single span (without columns) are considered to be safe in the longitudinal direction.

Minimum required seat width at the expansion joints

The minimum required seat width at the expansion joint is obtained by considering the most critical of: firstly, the maximum possible expansion joint opening; and secondly, the maximum displacement of the columns prior to failure due to longitudinal and transverse direction seismic excitations, respectively. These will be assessed individually in the following subsections.

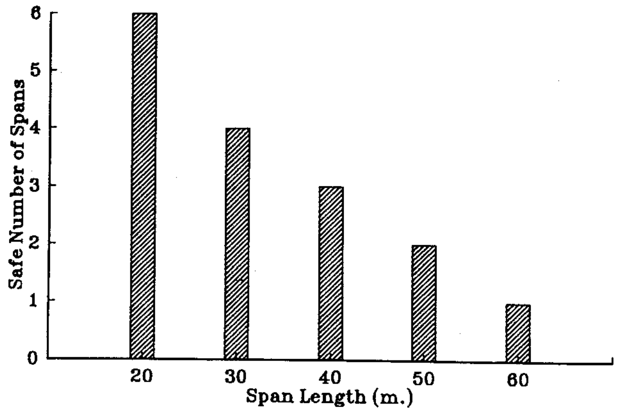

Figure 11 Safe number of spans considering failure of columns due to longitudinal direction seismic excitation

Maximum possible expansion joint openings

Maximum possible EJOs of a two-span and a four-span simply supported bridge in the longitudinal direction are illustrated schematically in *Figure 12*. Assuming that all the joints have the same width and neglecting the deformation of the abutments, the two- and four-span bridges can have maximum EJOs equal to one and three times the EJW, respectively. Similarly, in the transverse direction, due to the rotation of each span, the corners of the decks at the expansion joints displace longitudinally, and eventually, contact each other and the abutment walls. Thus, neglecting the deformation of the abutments, the maximumum possible opening in the expansion joints is also the same in this direction.

The EJOs obtained from case studies of multispan simply supported bridges conducted by Tseng and Penzien[1], Zimmerman and Brittain[14] and Imbsen and Penzien[4] are compared with the maximum possible EJOs of the bridges they studied. Most of these results are close to the maximum possible EJOs. This confirms that seismically-induced EJOs in multispan simply supported bridges do reach close to the maximum physically possible value.

If the fixed bearings at the left abutment seen in *Figure 12* are also damaged, the abutment-fixed deck is then also free to slide in the longitudinal direction, and this deck can potentially have a maximum displacement equal to the sum of the EJWs. It is noteworthy that the sum of the EJWs is calculated simply by multiplying the total end-to-end length, L_T, of the bridge by the coefficient of thermal expansion and design differential temperature. Depending on the temperature on the day of an earthquake, the bridge spans will have contracted or elongated from their initial as-built condition. When the temperature is at its minimum design value, the expansion bearing is closer to the seat edge. Therefore, an allowance for temperature changes should be made when evaluating the adequacy of the seat width against seismic actions. This allowance is conservatively calculated by multiplying the length, L_i, (in metres) of the span supported by the expansion bearings at expansion joint i by the coefficient of thermal expansion and the design differential temperature. Additionally, a 50 mm distance between the edge of the support and the centreline of the bearings is provided to prevent failure of the structure due to local damage at the support edge. Using the above information, and assuming a maximum temperature differential of 70°C in North America, the minimum required seat width SW_i (in mm) at expansion joint i is expressed by

$$SW_i = 50 + 0.84\,(L_T + L_i) \tag{7}$$

Note that the above equation gives an estimate of the minimum distance needed between the centreline of the bearing and the edge of the support to prevent bridge decks from falling off their seats.

Expansion joint openings due to longitudinal direction displacements of columns

In this section, the maximum possible EJOs due to seismic excitation in the longitudinal direction are studied. The sum of the EJWs is assumed to be larger than the EJOs due to the maximum relative displacement of the columns before they are damaged. Accordingly, the maximum EJOs are controlled by the maximum deformation capacity of the columns prior to failure. The maximum possible EJOs due to relative displacement of columns are illustrated in *Figure 13* for a two-span and a multispan simply supported bridge. For the two-span simply supported bridge the EJO is equal to the maximum displacement of the column before failure. For the multispan simply supported bridge, the opening of the *i*th expansion joint is equal to the sum of the maximum

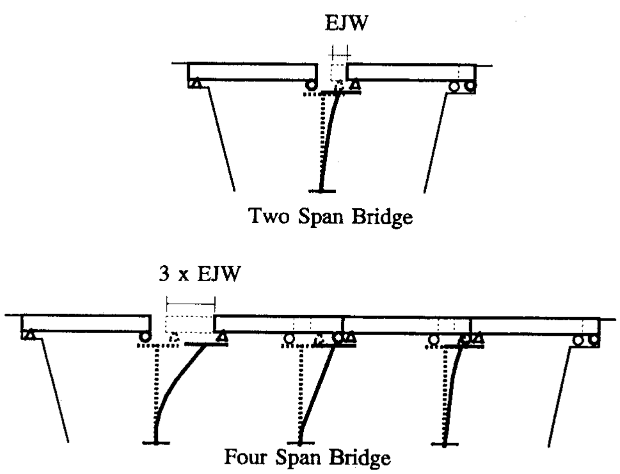

Figure 12 Maximum possible expansion joint opening due to longitudinal direction displacement of structure

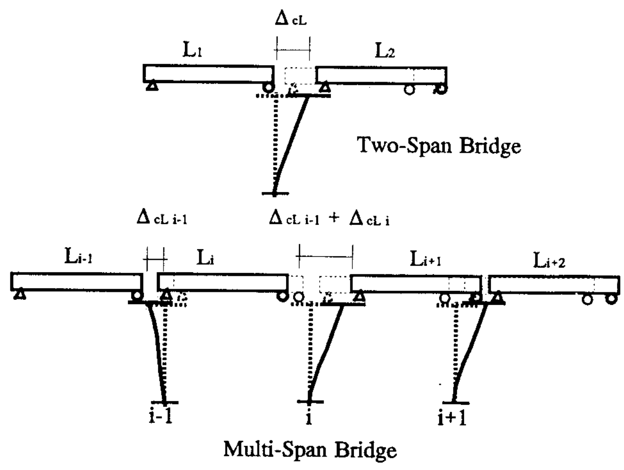

Figure 13 Maximum expansion joint opening before columns fail due to seismic excitation in longitudinal direction

displacements of columns $i-1$, and i before failure. As a general rule, when considering the opening of an expansion joint, the column at the expansion joint of interest and the neighbouring column connected to the deck by fixed bearing should always be considered. Then the EJO due to relative displacement of columns is expressed as

$$\Delta_{ejo_i} = \Delta_{cL_{i-1}} + \Delta_{cL_i} \tag{8}$$

Considering the movement of the bridge deck due to temperature difference and providing again a distance of 50 mm between the edge of the support and the centreline of the bearings for safety, the minimum required seat width, SW_i at expansion joint i is expressed as

$$SW_i = 50 + 0.84\,L_i + \Delta_{cL_{i-1}} + \Delta_{cL_i} \tag{9}$$

Although the maximum column displacements before failure can be obtained by dividing the maximum resistible strong axis moment, M_{ax}[23] by the column height and stiffness for each specific case, for practical applications, the above equation needs to be simplified and represented as a function of column height and span length. Using the maximum possible column displacements before failure obtained using 20 existing steel bridges with different span lengths and column heights, the maximum possible displacements of the columns as a function of span length and column height is obtained as

$$\Delta_{cL} = (150 - L)\,\frac{h_c}{5} \tag{10}$$

The above function is plotted in *Figure 14* together with some of the analytical results denoted by solid circles. As seen in *Figure 14*, although the above equation yields conservative values, the difference is not major.

Note that, the axial forces on the columns of a multispan simply supported bridge are obtained by summing the reaction forces from the two neighbouring spans supported by the columns. Then, the column sizes are determined using these axial forces which are proportional to the span length. Accordingly, for multispan simply supported bridges with different individual span lengths, the average length of two adjacent spans can be used in the above equation. Knowing this, the above equation is represented in a more general form as

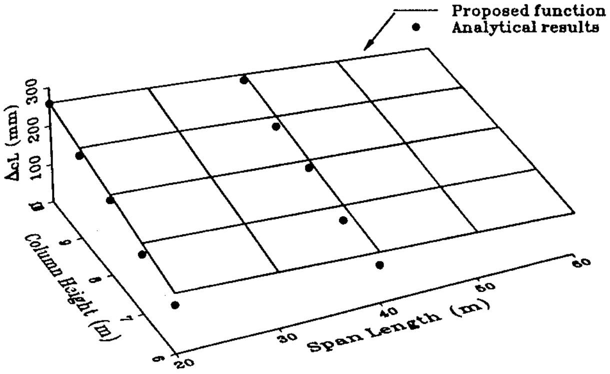

Figure 14 Comparison of exact analytical results with proposed function for maximum longitudinal displacement of columns before failure as a function of span length and column height

$$\Delta_{cL_i} = \left(150 - \frac{L_i + L_{i+1}}{2}\right)\frac{h_{c_i}}{5} \tag{11}$$

where L_i and L_{i+1} are the lengths of two adjacent spans supported by column i. The above equation is substituted in equation (9) and rearranged to obtain the minimum required seat width as a function of span length and column height

$$SW_i = 50 + 0.84\,L_i + \left(30 - \frac{L_{i-1} + L_i}{10}\right)h_{c_{i-1}} + \left(30 - \frac{L_i + L_{i+1}}{10}\right)h_{c_i} \tag{12}$$

Expansion joint openings due to transverse direction displacements of columns

The maximum possible EJOs due to seismic excitation in the transverse direction are now considered. Potentially, large openings in the expansion joints may develop due to rotation of the decks when the bridge displaces in the transverse direction. The sum of the EJWs is assumed to be larger than the EJOs due to maximum relative displacement of the columns before they are damaged. Accordingly, the maximum EJOs are controlled by the maximum deformation capacity of the columns before failure. The EJOs of a multispan bridge are shown in *Figure 15*. The assumed deformed geometry in Figure 15 produces the maximum rotation at the decks hence a maximum EJO. The rotation of a deck is controlled by the maximum transverse displacement of the columns before failure at each end of the deck. This rotation is obtained by dividing the sum of the maximum transverse displacements of the columns by the span length. Then, the EJOs are obtained by multiplying the rotations of the decks at each side of the expansion joints by the half of the width, b, of the bridge deck and summing the results. Considering longitudinal movements of the bridge deck due to temperature variations and providing a distance of 50 mm between the edge of the support and the centreline of the bearings for safety, the minimum required seat width, SW_i at expansion joint i is expressed as

$$SW_i = 50 + 0.84\,L_i + \frac{b}{2}\left(\frac{\Delta_{cT_{i-1}} + \Delta_{cT_i}}{L_i} + \frac{\Delta_{cT_i} + \Delta_{cT_{i+1}}}{L_{i+1}}\right) \tag{13}$$

A procedure similar to the one described earlier is followed to obtain the minimum required seat width expressed below, as a function of span length and column height

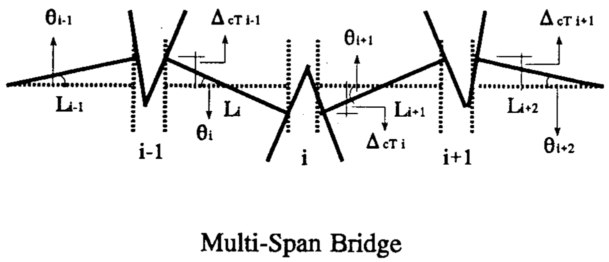

Figure 15 Maximum expansion joint opening of a multispan simply supported bridge before columns fail due seismic loading in transverse direction

$$SW_i = 50 + 0.84\, L_i + \frac{b}{2}\left(\frac{50}{L_i} - 0.15\left(1 + \frac{L_{i-1}}{L_i}\right)\right)h_{c_{i-1}} + \frac{b}{2}\left(\frac{50}{L_i} + \frac{50}{L_{i+1}} - 0.15\left(2 + \frac{L_{i+1}}{L_i} + \frac{L_i}{L_{i+1}}\right)\right)h_{c_i} + \frac{b}{2}\left(\frac{50}{L_{i+1}} - 0.15\left(1 + \frac{L_{i+2}}{L_{i+1}}\right)\right)h_{c_{i+1}} \quad (14)$$

Interpretation of derived equations for minimum seat width

Three equations are derived to define the minimum required width. Equations (12) and (14) are derived considering the maximum displacement of the columns prior to failure due to longitudinal and transverse direction seismic excitations, respectively. The purpose of these equations is to determine if a bridge is likely to fall off its supports before the columns reach their capacity. Due to the nature of earthquake excitations, it is found conservative and logical to use the larger of the results obtained from these equations to calculate the minimum required seat width. However, neglecting the deformation of the abutments, the EJO defined by the maximum displacement of the columns cannot be larger than the sum of the EJWs. Therefore, the larger of the results obtained from equations (12) or (14) should be compared with the result obtained from equation (7) and the smaller of these defines the minimum required seat width. However, calculation of seat width for 16 bridges with different numbers of spans and span length showed that equation (14) provides results smaller than those of equation (12) for all the cases considered. Equation (14) may yield larger seat widths only for wide- and short-span bridges where the deck rotations, hence the EJOs, are larger. However, for short-span bridges, equation (7) yields results smaller than those of equation (14) and therefore it governs. Accordingly, equation (14) need not be used in most cases.

Conclusions

Bearing stiffness is found to considerably affect the seismic response of two-span simply supported bridges. The transverse direction periods of these bridges are highly dependent on the stiffness of the bearings used. In particular it is observed that the transverse direction fundamental periods of two- and three-lane bridges are very close when bearings-set at the abutments have zero rotational stiffness, but the difference becomes larger as the stiffness increases. However, for bridges with more than two spans, the effect of bearing stiffness is localized and it vanishes as the number of spans increases.

Beyond the stiffness and period issues, the level of force acting on the bearings of two-span bridges is also very important. For a bearings-set of infinite rotational stiffness, the forces in the bearings at the abutments increase with span length significantly for spans up to 30–40 m, but beyond this range, the increase is not as much. For a bearings-set of infinite rotational stiffness, the forces in the bearings of narrower bridges are always larger for the range of spans considered, but the difference becomes smaller as the span length increases. When sliding-bearings are used, the bearing forces at the abutments of two-lane bridges are larger than those in three-lane bridges up to 40 m span and then they become smaller with increasing span length, but the difference is not very large. The forces in a bearings-set with zero rotational stiffness are approximately the same for two- and three-lane bridges. However, they are negligibly small compared to the forces in other types of bearings.

The seismic capacity of multispan simply supported bridges considering column instability is studied. It is found that for two-span simply supported bridges, the MRPGAs for wider bridges are larger when bearings that develop rotational resistance at the supports are used. The difference becomes more pronounced as the rotational stiffness of the bearings-set increases. However, this effect vanishes for bridges with a larger number of spans. For bridges with a bearings-set of zero rotational stiffness, the MRPGAs are identical for two- and three-lane bridges. However, they are greately reduced compared to those of the bridges with other types of bearings considered in this study. It is found that bridges with this type of bearings may be damaged by earthquakes of peak accelerations less than 0.20g. Increasing span length is also found to have a negative impact on the seismic capacity due to high moments exerted on the columns.

The seismic response of bridges with more than two spans is also studied. It is found that in the case of bridges with a bearings-set of zero rotational stiffness, the variation of maximum transverse displacement as a function of the number of spans is not significant. In the case of bridges with sliding-bearings, the maximum transverse displacement of two-span bridges is found to be much smaller than that of those with more spans. However, the difference between the maximum transverse displacements of bridges with more than two spans is not considerable. Therefore, the transverse seismic capacities of multispan simply supported bridges are almost identical. In fact, regardless of the bearing stiffness, the transverse seismic capacity of multispan simply supported bridges is nearly identical to that of a two-span simply supported bridge with a bearings-set of zero rotational stiffness and identical individual span length. Furthermore bridges with fewer spans are found to be more vulnerable to seismic excitations than those with more spans of identical end-to-end length.

For the seismic response in the longitudinal direction, it is found that columns in multispan simply supported bridges can sustain displacements as much as twice the EJW. Therefore, the columns in two-span simply supported bridges are considered to be safe in the longitudinal direction. However, for bridges with more than two spans, the safety of the columns in the longitudinal direction cannot be assured. It is also found that the peak ground accelerations required for collision increases with span length, but they are all less than 0.1g. Therefore, collision of the decks in the longitudinal direction is inevitable. It is noteworthy that impacting between the two adjacent sections of a bridge superstructure upon collision may cause high shear forces in the bearings and therefore these components may fail before the columns. This would produce highly unstable systems for which the superstructure is disconnected from the columns, with a high risk of having simply supported decks falling off their support. Therefore, impacting should certainly be prevented to avoid damage to the bearings and hence to the structure.

It is noteworthy that the figures developed throughout this research can be used to obtain a conservative, but rapid, quantitative and qualitative assessment of the seismic behaviour and capacity of multispan simply supported steel bridges. Further research is being undertaken to develop analytical expressions which will quantify this behaviour more accurately.

Acknowledgments

Financial assistance provided by the Natural Sciences and Engineering Research Council of Canada is gratefully acknowledged. The findings and recommendations in this paper, however, are those of the writers and not necessarily those of the sponsor.

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