QUANTITATIVE APPROACH TO RAPID SEISMIC EVALUATION OF SLAB-ON-GIRDER STEEL HIGHWAY BRIDGES

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ABSTRACT: A quantitative approach follows for developing a methodology for the rapid seismic evaluation and ranking of slab-on-girder steel highway bridges. In the development of this methodology, analytical expressions are introduced to calculate the seismically induced forces and displacements in bridge components, first assuming that damage to bearings is not acceptable and then removing this constraint. In the latter case, the superstructure is assumed to become unrestrained and slide on its supports. The seismic screening of the bridges is based on the calculation of a ranking index defined as the product of an importance index and overall damage index. Unlike other existing methodologies, the overall damage index of the structure is determined considering the impact of damage to each component on the successive failure of other components and structure as a whole. A cut-off mechanism is introduced to prevent the potentially undue impact some dominating societal aspects could have on the ranking of existing bridges with otherwise excellent seismic-resistance adequacy. The proposed methodology also quantitatively addresses the risk inherent to all seismic hazard zones.

INTRODUCTION

A sizable number of steel bridges in North America have been designed and constructed at a time when seismic-resistant requirements were nonexistent or inadequate by today's standards. Recent earthquakes such as the 1989 Loma Prieta, 1994 Northridge, (Earthquake Engineering Research Institute 1990, 1994) and 1995 Kobe earthquakes (Bruneau et al., in press, 1995) have demonstrated the seismic vulnerability of existing highway structures designed in those conditions. To avoid bridge collapses, reduce the risk of extensive damage in future earthquakes, and most effectively allocate the limited financial resources available for this task, the bridges most in need of seismic retrofit must be identified, taking into account their structural seismic deficiencies as well as their consequences of failure on the economic, social, and emergency deployment aspects. To identify those bridges, engineers must determine the physical state of each bridge based on engineering drawings and field inspection and calculate their response to the probable seismic excitations at the site. Assessing the seismic response of each specific bridge by rigorous structural analysis is a long and tedious process. Thus, a methodology for rapid but refined seismic evaluation of existing steel bridges is desirable.

While methodologies allow to rapidly identify the seismically deficient bridges and rank them in terms of their respective vulnerability [Applied Technology Council (ATC) 1983; CALTRANS 1992, Filiatrault et al. 1994], it remains that the bridge vulnerability aspect of these methodologies is crude and generally limited to simple recognition of undesirable structural features known to have performed inadequately in past earthquakes. Thus, bridges sharing such features would also share the same rating, independently of variations in geometry and other structural properties. As one step to improve on this situation, a methodology is proposed to perform rapid, yet quantitative, seismic vulnerability assessments. It was developed and applied for a special class of bridges, namely single-span and multispan slab-on-girder highway steel bridges. In the latter case, only steel columns were considered, as these have received little attention in past research [whereas the behavior of multispan bridges having concrete columns has already received considerable attention (e.g., Priestley 1985, 1988; Priestley and Park 1987; Ghobarah and Ali 1988 and Saiidi et al. 1988)].

The spans of the simply supported bridges studied herein are assumed to be supported by fixed bearings at one end and by expansion bearings at the other end. For continuous bridges, fixed bearings are assumed to exist at one abutment, expansion bearings at the other, and the columns are assumed to frame continuously into the girders. The superstructure in most of these bridges is supported by steel sliding bearings. The fixed type of such bearings is shown in Fig. 1. The expansion type is nearly identical but without the longitudinal stopper bars. Even minor earthquakes have caused the failure of anchor bolts, keeper bar bolts, and welds in such bridge bearings [Federal Highway Administration (FHWA) 1987].

Accordingly, first assuming that damage to bearings is unacceptable, analytical expressions for the fundamental periods, seismically induced bearing forces and column moments in both orthogonal directions are introduced throughout the development of the methodology. Then, it is assumed that stable damage to bearings is possible and acceptable, as would be the case if anchor bolts of stocky sliding bearings rupture during an earthquake. This would allow the superstructure to slide on its abutments' support. A chart of the expected sliding displacements at these supports as a function of simple bridge parameters (Dicleli and Bruneau 1995a) is then used to derive analytical expressions for column moments. Using these expressions and findings from previous studies (Dicleli 1993;

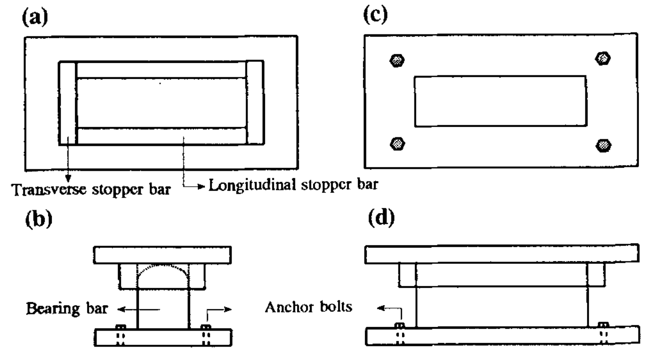

FIG. 1. Typical Fixed Sliding Bearing: (a) Top Plate; (b) Side View of Bearing; (c) Bottom Plate Plan View; (d) Front View of Bearing

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Dicleli and Bruneau 1995a,b,c), a methodology for rapid seismic evaluation and ranking of steel highway bridges is developed.

METHODOLOGY—BASIC CONCEPT

In the methodology proposed, a ranking index, I_R, is used to identify and rank the bridges in greatest need of retrofit. It is defined based on the interaction between bridge importance and vulnerability, as expressed by two variables: the importance index, I_i, and the overall damage index, I_d, of the bridge.

The importance index varies between 0 and 1 as a function of the impact of damage on the social, economic, and practical aspects of the problem. The importance of a bridge is evaluated by considering all consequences of its damage/loss on the highway system and local community, the ability to provide emergency services, the national security/defense network and overall postearthquake recovery activities in the affected area. The ratio of repair or replacement cost to seismic retrofit cost is another important aspect addressed by the importance index. The procedure followed to classify bridges according to their importance is well established (CALTRANS 1992) and need not be modified. In summary, bridges with an importance index of 0 are those for which all consequences of failure are acceptable, whereas, bridges with an importance index of 1 are those whose loss is unacceptable; bridges in the latter case would urgently need to be retrofitted if deficient.

The overall damage index, I_d, of a bridge is a function of the damage indices of its different components. Component damage indices reflect the ability of various key structural components to resist earthquakes likely to occur at a site with a preselected probability of exceedance, and are called the site earthquake thereafter. Damage indices are expressed as demand/capacity ratios for these components; values less than 1.0 indicate that the corresponding component is unlikely to fail during the site earthquake, whereas values greater than 1.0 denote possible failure. Obviously, the bridge components that could potentially be damaged during an earthquake vary depending on the type of bridge. For the steel bridges studied here, three types of component damage indices are considered. These are the seat-width index, I_{sw}, bearing-damage index, I_{bd}, and column-damage index, I_{cd}. These indices will be described in detail in the subsequent sections. Component damage indices for the foundations and abutments are not considered since they are beyond the scope of the present study, but could be easily included in the proposed methodology. Foundation and liquefaction issues have been addressed by other researchers (e.g., Kawashima 1990; Youd 1993; and Seed and Idriss 1982). Likewise, the effect of seismic restrainers was neglected but could be considered if these bridges were evaluated for possible further retrofits. Skewed or curved bridges are also beyond the scope of the present work.

SEAT-WIDTH INDEX

Single-Span Simply Supported and Continuous Bridges

For single-span simply supported and continuous bridges, the seat-width index is defined in both transverse and longitudinal directions. In the transverse direction, it is defined as

$$I_{swT} = (u_s + 50)/SWT \quad (1)$$

where u_s = transverse sliding displacement of a bridge structure (mm) whose bearing would have been damaged, and is obtained from Fig. 2 using the site peak ground acceleration; and SWT = seat width (mm) measured in the transverse direction from the edge of the exterior bearing to the edge of the abutment. An additional length of 50 mm is provided in the

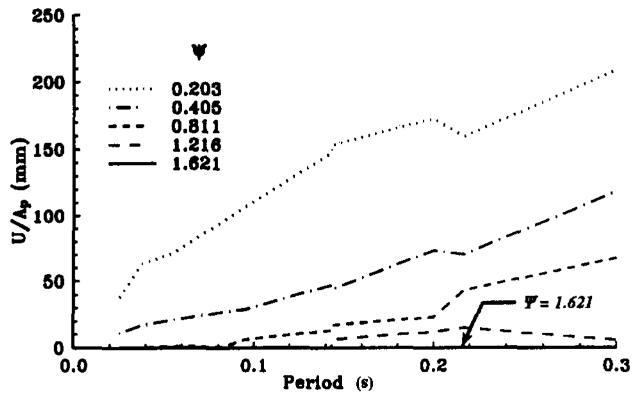

FIG. 2. Sliding Displacement per Peak Ground Acceleration (%g) as Function of Period for Various ψ Values (Mean of Western U.S. Earthquakes)

previous equation considering that damage may occur to the unreinforced part of the abutment when the superstructure slides.

In the longitudinal direction, the movement of the deck is restricted by the abutment walls at both ends (in the present paper, unless indicated otherwise, the work "deck" is used as an inclusive term for the combination of slab and girders). Therefore, neglecting deformations of the abutments, the sliding displacement of the bridge cannot be larger than the expansion-joint width (EJW). Accordingly, the seat-width index in the longitudinal direction is defined by the following equation:

$$I_{swL} = \frac{u_s + 50}{SWL} \le \frac{0.84L_T + 50}{SWL} \quad (2)$$

where the term $0.84L_T$ = EJW (mm) obtained by conservatively assuming a temperature differential of 70°C (Dicleli and Bruneau 1995c); L_T = total end-to-end length of the bridge (m); and SWL = seat width (mm) measured in the longitudinal direction from the centerline of the bearings to the edge of the abutment. Note that a parameter accounting for deformation of the abutments could be added to the preceding equation if such deformations are expected.

Fig. 2, key to the calculation of this index, is constructed using a number of western U.S. earthquakes and is used to obtain the sliding displacement of bridges. In the figure, the vertical axis is the sliding displacement, U, normalized by peak ground acceleration expressed as a fraction of gravitational acceleration, A_p, and the horizontal axis is the fundamental period. Each line corresponds to a dimensionless ratio, ψ, equal to the sliding resistance of a given structure, F_s, divided by the product of its effective modal mass, M_{eff}, and the peak ground acceleration, A_p. For single-span simply supported and continuous bridges

$$\psi = 8w_f\mu_f/\pi^2A_p \quad (3)$$

where w_f = percentage of the weight of the structure transferred to all supports where friction resistance exists; and μ_f = friction coefficient. Note that the product of w_f and μ_f is equal to the ratio of F_s over the structure's total weight.

Fig. 2 is obviously useful only if the peak acceleration at the site is larger than the minimum peak ground acceleration required for sliding. For single-span simply supported and continuous bridges, this minimum required peak ground acceleration for sliding, as a fraction of gravitational acceleration, for both transverse and longitudinal directions of earthquake excitations is expressed as (Dicleli and Bruneau 1995b)

$$A_p = (\pi^2/8\beta)w_f\mu_f \quad (4)$$

where β = ratio of spectral acceleration S_a to peak ground acceleration A_p. For single-span simply supported bridges, the total weight of the structure is transferred to the abutments. Therefore, w_f is 1.0 for the transverse direction sliding but only 0.5 for the longitudinal direction sliding since such bridges are designed to move freely at one of their ends in this direction. For continuous bridges, some portion of the weight is transferred to the columns, and therefore w_f is a function of the number of column bents in the structure. However, it can be calculated approximately by considering the tributary weight transferred to the supports where sliding can occur.

To obtain the seat-width index for single-span simply supported and continuous bridges, the transverse and longitudinal direction elastic fundamental periods of the structures must first be calculated using, respectively, (5) and (6) derived as

$$T_1 = \sqrt{\frac{2mL_T^2}{\frac{\pi^2 EI_D}{2L_T} + \frac{L_T^2}{\pi^2}\sum_{i=1}^{n_{cs}} K_{cTi}\left(\sin\frac{\pi x_{ci}}{L_T}\right)^2 + K_{b\theta}\left(1 - \frac{1}{(3EI_D/L_T K_{b\theta})/+2}\right)^2}} \quad (5)$$

$$T_1 = \sqrt{\frac{4\pi^2 mL}{3EA_D}\left[1 + \frac{EA_D}{LK_{bL}}\left(3 - \frac{1}{(EA_D/LK_{bL}) + 1}\right)\right]} \quad (6)$$

In the preceding equations, m = total mass; L_T = total end-to-end length of the bridge; E = modulus of elasticity of steel; A_D = cross-sectional area of the entire deck-and girder superstructure transformed into steel using a modular ratio of 9; I_D = moment of inertia of the composite bridge superstructure calculated considering the composite action between the concrete slab and steel girders (Douglas 1979); n_{cs} = number of columns set; K_{CT_i} = transverse stiffness of the columns set i located at a distance x_{ci} from the support; and K_{bL} and $K_{b\theta}$ = the longitudinal and rotational stiffness of the bearings group located at the support and are expressed as (Dicleli and Bruneau 1995a,c)

$$K_{bL} = \sum_{i=1}^{nb} k_{bL_i} \quad (7)$$

$$K_{b\theta} = \sum_{i=1}^{nb} k_{bL_i} l_{b_i}^2 \quad (8)$$

where n_b = number of bearings; and k_{bL_i} and l_{b_i} = longitudinal stiffness and distance to the centerline of the bridge deck for bearing i. Note that $K_{b\theta}$ is zero in the case of the damaged bearing condition since Fig. 2 has been derived using this assumption. Fig. 2 has also been obtained assuming unrestrained sliding. However, if the distance between the fixed-end bearing and its abutment is considerably smaller than the required unrestrained sliding, the engineer may wish to adjust the value of u_s in accordance with sliding energy dissipation principles (Dicleli and Bruneau 1995b). As for impact with the abutment at the sliding-bearing end, it is prevented by (2).

Also, steel columns typically encountered in existing continuous slab-on-girder highway bridges, because of their small stiffness relative to the stiffness of the deck, were found to only contribute negligibly to seismic resistance and to the restraining of seismically induced lateral deformations of single-span simply supported bridges and continuous bridges (Dicleli and Bruneau 1995a,b). Therefore, when calculating the transverse direction fundamental period of these bridges, the second term in the denominator of (5), which represents the contribution of the columns' stiffness, could be ignored. Interestingly, this behavior is quite different in multispan simply supported steel bridges where the column stiffness plays a dominant role (Dicleli and Bruneau 1995c).

In the case of rigid bearings, since the longitudinal stiffness of the bridge superstructure is large, the longitudinal direction fundamental period is obviously very small and would not need to be calculated if a conservative design spectrum having a constant acceleration region at low periods was used. However, the methodology proposed here considers the true spectral shape, with its increasing acceleration level at low periods; this has a considerable impact on the conclusions of a seismic evaluation or vulnerability assessment study at low periods.

In summary, to determine the seat-width index in this case, the engineer must: (1) calculate periods using (5) and (6); (2) obtain β in both orthogonal directions from a design spectrum or a spectrum developed specifically for the site under consideration; (3) obtain the site peak ground acceleration, A_{ps}, as a fraction of gravitational acceleration from a seismic map or site-specific information; (4) determine the friction coefficient for the bearings at the abutments, considering the condition of the abutments and type of bearings; (5) use (4) to obtain A_p required for sliding in both orthogonal directions; (6) if $A_p > A_{ps}$, take u_s as 0, otherwise calculate ψ using (3) and obtain u_s from Fig. 2; and (7) substitute u_s obtained for transverse and longitudinal directions, respectively, in (1) and (2) to obtain the seat-width indices in both orthogonal directions. The larger of these two is the seat-width index, I_{sw}, of the structure.

It is noteworthy that the bearings are assumed to be damaged, and therefore their strength is not considered for the calculation of the seat-width index. The reasons for this will become clear when the procedure to calculate the overall damage index is presented.

Multispan Simply Supported Bridges

For multispan simply supported bridges, the seat-width index is defined as

$$I_{sw} = (SW_i/SWL_i)_{\max} \quad (9)$$

where SWL_i = width (mm) of the bearing seat supporting the unrestrained expansion end measured from the centerline of the bearing to the support edge at expansion joint i; and SW_i = minimum required seat width (mm) defined by the smaller of the results obtained from the following two equations (Dicleli and Bruneau 1995c):

$$SW_i = 50 + 0.84(L_T + L_i) \quad (10)$$

$$SW_i = 50 + 0.84L_i + \left(30 - \frac{L_{i-1} + L_i}{10}\right) h_{c_{i-1}} + \left(30 - \frac{L_i + L_{i+1}}{10}\right) h_{c_i} \quad (11)$$

where h_{c_i} = height of column i; L_i and L_{i+1} = lengths of two adjacent spans supported by column i; and L_T = total end-to-end length of the bridge. The ratio of SW_i to SWL_i is calculated for each expansion joint, and the maximum of these defines the seat-width index.

BEARING-DAMAGE INDEX

Single-Span Simply Supported and Continuous Bridges

The bearing-damage index for single-span simply supported and continuous bridges is defined as the ratio of the bearing-force demand, B_r, to the ultimate capacity, B_{rc}, of the bearing

$$I_{bd} = B_r/B_{rc} \quad (12)$$

The bearing-force demand is defined by the following equation:

$$B_r = \sqrt{(C_1 b_{ry})^2 + (C_1 b_{rxy} + C_2 b_{rx})^2} \quad (13)$$

where C_1 and C_2 = correlation factors to account for the simultaneous occurrence of seismic excitations in both orthogonal directions, and the directional uncertainty of the earthquake motions, b_{ry} and b_{rxy}, are respectively transverse and longitudinal direction bearing forces produced by transverse direction seismic excitation and are expressed as

$$b_{ry} = \frac{4\beta A_p}{n_b}\left[\frac{m}{\pi^2} + \frac{K_{b\theta}}{(L_T\omega_1)^2}\left(1 - \frac{1}{(3EI_D/L_T K_{b\theta}) + 2}\right)\right] \quad (14)$$

$$b_{rxy} = \frac{4\beta A_p}{L_T\omega_1^2}\left(1 - \frac{1}{(3EI_D/L_T K_{b\theta}) + 2}\right) l_{be} k_{bL} \quad (15)$$

and b_{rx} = longitudinal direction bearing force produced by longitudinal direction seismic excitation expressed as

$$b_{rx} = \left[\frac{3(2 + (L_T K_{bL}/EA_D))^2}{(3 + 2(L_T K_{bL}/EA_D))^2 + 3}\right]\frac{m\beta A_p}{n_b} \quad (16)$$

In the preceding equations, l_{be} = distance of the exterior bearing to the centerline of the bridge deck; ω_1 = fundamental circular frequency in the transverse direction; and all other terms have been defined previously.

It is noteworthy that bearing forces produced by transverse direction seismic excitation are the result of two force components as seen in Fig. 3. The first component is produced by the reaction force at the support and oriented in the transverse direction (b_{ry}). The other component is produced by the in-plane support moment and oriented in the longitudinal direction (b_{rxy}). The resulting bearing force is the vectorial sum of these forces.

To account for bidirectional seismic excitations, two load cases are considered to obtain the bearing-force demand. For the first load case, C_1 and C_2 are respectively taken as 1.0 and 0.3, and for the second load case they are taken as 0.3 and 1.0 (FHWA 1987). The largest of the results from these two load combinations is used to determine the bearing-damage index.

The capacity of traditional type of bearings is assumed to be governed by the shear capacity of the anchor bolts. Accordingly, the bearing capacity is defined as

$$B_{rc} = n_{ab} A_{ab} \tau_y \quad (17)$$

where τ_y, A_{ab}, and n_{ab} = the shear strength, area, and number of anchor bolts. However, other local failure modes should not be overlooked if probable.

Hence, the procedure to determine the bearing-damage index for single-span simply supported and continuous bridges requires (1) calculating the fundamental periods in the transverse and longitudinal directions using (5) and (6); (2) obtaining β in both orthogonal directions; (3) obtaining A_{ps} as a fraction of gravitational acceleration; (4) calculating the bearing forces due to seismic excitations in both orthogonal directions using (14), (15), and (16); (5) substituting these forces in (13) to obtain the bearing-force demand and; (6) determining the capacity of the bearings using (17). The bearing-damage index of the structure is the bearing-force demand divided by the capacity.

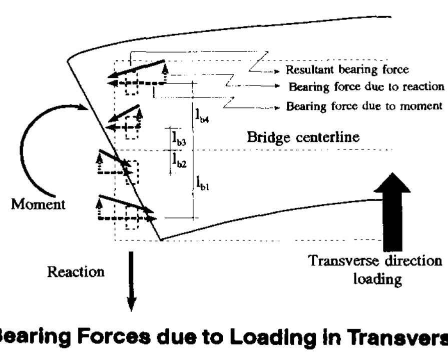

FIG. 3. Bearing Forces due to Loading in Transverse Direction

Multispan Simply Supported Bridges

In the case of multispan simply supported bridges, due to nonlinearities resulting from the collision of adjacent superstructure components in the longitudinal direction even in the absence of bearing failure, simple analytical expressions for the bearing-force demands cannot be obtained. However, it is generally accepted that impact may produce high forces in the bearings and damage them (Zimmerman and Brittain 1981; Imbsen and Penzien 1986). Accordingly, the capacity of the bearings is conservatively assumed to be limited by the peak ground acceleration, A_{pc}, required for collision of the column-fixed decks (i.e., decks connected to the column bent by fixed bearings) and is defined as (Dicleli and Bruneau 1995c)

$$A_{pc} = \frac{n_c k_{cL} \text{EJW}}{\beta m_D g}\left(1 - \frac{P_D}{k_{cL} h_c}\right) \quad (18)$$

where k_{cL} and h_c = longitudinal direction stiffness and height of the column; P_D = axial force in the column due to dead load of the structure; n_c = number of columns at a bent; and m_D = mass of the superstructure fixed to the column bent. The bearing-damage index for multispan simply supported bridges is then defined as the ratio of site peak ground acceleration to the peak ground acceleration required for collision

$$I_{bd} = A_{ps}/A_{pc} \quad (19)$$

To calculate the bearing-damage index in this case, it is recommended to consider the column-fixed deck adjacent to the narrowest expansion joint, or, if the EJWs are identical, the deck with the longest span. Also, the period of the column-fixed deck, needed to determine β, is

$$T_1 = 2\pi\sqrt{m_D/K_{cL}} \quad (20)$$

where K_{cL} = stiffness of the column set obtained by summing up the column stiffnesses.

COLUMN-DAMAGE INDEX

Using case studies and capacity design concepts, the writers observed that seismically induced shear failures in the steel columns of the type of bridges considered here are unlikely, contrary to what has been observed in the past in reinforced concrete bridge columns. Accordingly, only the effect of axial and flexural forces is considered here. Conservatively, using a linear biaxial moment-interaction relationship, the column-damage index is defined as the sum of the ratios of seismic moment demands of the columns to their flexural capacities in both orthogonal directions

$$I_{cd} = \frac{C_1 M_{Ey} + C_2 M_{Eyx}}{M_{ay}} + \frac{C_1 M_{Exy} + C_2 M_{Ex}}{M_{ax}} \quad (21)$$

where M_{ay} and M_{ax} = transverse and longitudinal direction flexural capacities of the column accounting for the presence of axial load due to weight of the structure. These flexural capacities can be obtained from a stability-interaction equation (e.g., Duan and Chen 1989), and the steel bridge columns are conservatively assumed to fail as soon as the capacity delimited by this statically derived interaction curve is reached (Dicleli and Bruneau 1995a,c). M_{Ey} and M_{Eyx} are the magnified (i.e., including second-order effects) transverse seismic moment demands due to seismic excitation in the transverse and longitudinal directions, respectively. M_{Eyx} has not been used in the present study but was included previously to make the equation more general. M_{Ex} and M_{Exy} are the magnified longitudinal seismic moment demands due to seismic excitation in the longitudinal and transverse directions, respectively. The largest of the column-damage indices resulting from the two same load cases considered to account for bidirectional earthquake excitation (i.e., C_1 = 1.0, C_2 = 0.3 and C_1 = 0.3, C_2 = 1.0) is retained.

Continuous Bridges

For continuous bridges, the seismic moments in the columns are determined by the displacement of the deck at the columns' location. Therefore, the damage index of a bridge having identical column sizes and heights at each bent need only be calculated for the columns closer to the midspan, since these have larger transverse direction displacement and larger seismic moments. In other cases, each column set should be checked separately.

To calculate the column-damage index for the undamaged bearing condition, only the transverse direction response need to be considered since, in the longitudinal direction, the stiffness of the deck is relatively high resulting in negligible deformations and column seismic moments. In that case, M_{Exy}, M_{Ex}, and M_{Eyx} are zero and the transverse seismic moment demand, M_{Ey}, is

$$M_{Ey} = \frac{4\beta A_p}{\pi\omega_1^2} \sin\left(\frac{\pi x_c}{L_T}\right)(k_{cT}h_c + P_D) \tag{22}$$

where k_{cT} = transverse stiffness of the column, and the column damage index is given directly by (21).

However, if the bearings are damaged, the deck may slide and produce seismic moments in both transverse and longitudinal directions. Therefore, to calculate the column-damage index for the damaged bearing condition: (1) calculate the stiffness and flexural capacities of the columns in both orthogonal directions; (2) use (5) and (6) to calculate the fundamental periods in the transverse and longitudinal directions assuming zero rotational stiffness for the bearing group; (3) use these periods to obtain β in both orthogonal directions; (4) obtain A_{ps} as a fraction of gravitational acceleration; (5) assess the friction coefficient for the bearings at the abutments and; (6) obtain A_p required for sliding in both orthogonal directions using (4). Then, depending on the magnitude of A_p, the following four cases arise:

1. If $A_p > A_{ps}$ in both orthogonal directions, then u_s is 0, and follow the procedure for the undamaged bearing condition to calculate the column-damage index.
2. If $A_p > A_{ps}$ in the transverse direction and $A_p < A_{ps}$ in the longitudinal direction, use (22) to calculate the seismic moment demand in the transverse direction. Then, get u_s from Fig. 2 for the longitudinal direction and calculate the longitudinal seismic moment demand as

$$M_{Ex0} = u_s(k_{cL}h_c + P_D) \le \frac{0.84L_T}{1{,}000}(k_{cL}h_c + P_D) \tag{23}$$

   where upper limit is obtained knowing that the sliding displacement of the bridge in the longitudinal direction is restricted by EJW of the bridge (i.e., $0.84L_T$).
3. If $A_p < A_{ps}$ in the transverse direction and $A_p > A_{ps}$ in the longitudinal direction, ignore the response in the longitudinal direction, obtain u_s from Fig. 2 for the transverse direction, and calculate the transverse seismic moment demand as

$$M_{Ey} = \left(u_s + \frac{\pi\mu_f w_f}{2m\omega_1^2}\sin\frac{\pi x_c}{L}\right)(k_{cT}h_c + P_D) \tag{24}$$

4. If $A_p < A_{ps}$ in both orthogonal directions, obtain u_s in both orthogonal directions and substitute them in (23) and (24) to obtain the transverse and longitudinal seismic moment demands.

Multispan Simply Supported Bridges

Eigenvalue analyses for multispan simply supported bridges showed that their transverse fundamental periods are governed by that of the column bent having the largest tributary mass to column-stiffness ratio. This period can be closely approximated when that bent is analyzed as part of a two-span simply supported bridge having zero rotational stiffness at its abutment supports (Dicleli 1993; Dicleli and Bruneau 1995c). This is expected since a multispan simply supported bridge is a discontinuous structure (i.e., no rotational continuity between two adjacent spans) composed of discrete column bents. If a bridge has identical column sizes and heights at each bent, the two adjacent spans with the largest average length and the columns supporting them are selected. Otherwise, the columns at each bent should be considered separately and the largest damage index is retained.

In a multispan simply supported bridge subjected to seismic excitation in the transverse direction, the exterior columns of any given column set are the most vulnerable due to their higher longitudinal seismic moments resulting from the rigid-body rotation of the column-fixed deck. Therefore, these exterior columns are considered for the calculation of the column-damage index.

Using the total mass of the selected two adjacent spans and the transverse and longitudinal direction stiffnesses of the columns supporting them, the transverse fundamental period of the structure is

$$T_1 = \sqrt{\frac{4\pi^2 m}{3\left(K_{cT} + \frac{K_{b\theta}}{L_1^2} + \frac{K_{c\theta}}{L_2^2}\right)}} \tag{25}$$

where L_1 and L_2 = lengths of the selected two adjacent spans, with columns connected to the span having length L_2; and $K_{c\theta}$ = torsional stiffness of the column set defined as

$$K_{c\theta} = \sum_{i=1}^{n_c} k_{cL}d_{c_i}^2 \tag{26}$$

where d_{c_i} = distance of column i to the centerline of the bridge deck. Using the calculated period, the corresponding β, and A_{ps}, the seismic moment demands due to transverse direction seismic excitation are given by

$$M_{Ey} = \frac{3\beta A_p}{2\omega_1^2}\beta_{my}k_{cT}h_c \tag{27}$$

$$M_{Exyi} = \frac{3\beta A_p}{2L_2\omega_1^2}\beta_{mxy}k_{cL}h_c d_{ci} \tag{28}$$

where β_{my} and β_{mxy} = moment magnification factors to account for the transverse and longitudinal direction second-order seismic moments resulting from transverse displacement and torsional rotation of the column bent due to seismic excitation in the transverse direction (Dicleli and Bruneau 1995c). It is noteworthy that if collision between the decks occurs at the expansion joint due to the relative rotation of the adjacent spans, then (27) and (28) may not give correct results. However, as demonstrated elsewhere (Dicleli and Bruneau 1995c), in most cases, steel columns are likely to be severely damaged before impact takes place. Therefore, the equations derived can generally be used to predict the columns' seismic moments.

In the longitudinal direction, due to nonlinearities resulting from the collision of adjacent superstructure components, the seismic behavior of these bridges becomes very complex.

However, if the bearings are not damaged as a consequence of these impacts, displacements as large as the sum of the EJWs at one or the other side of the column under consideration can possibly develop (Dicleli 1993; Dicleli and Bruneau 1995c). Therefore, the maximum possible longitudinal seismic moment for the kth column is defined as

$$M_{Ex} = \max \left\{ \sum_{i=1}^{k} \text{EJW}_i, \sum_{i=k+1}^{n_{ej}} \text{EJW}_i \right\} (k_{cL}h_c + P_D) \quad (29)$$

where n_{ej} = number of expansion joints; EJW_i = expansion-joint width of joint i; and all other terms have been defined previously.

OVERALL DAMAGE INDEX OF STRUCTURE

Single-Span Simply Supported Bridges

Bearings are the most vulnerable superstructure components in the case of single-span simply supported bridges. Fortunately, damage to these components does not necessarily result in failure of the structure if they can remain stable while sliding, as in the case considered here. However consequences of this damage should be estimated. Accordingly, the overall damage index, I_d of the structure is defined considering two of the following possible cases:

1. If the bearings are not damaged ($I_{bd} < 1.0$), the damage index of the structure is defined as the smaller of the bearing damage or seat-width indices.
2. If the bearings are damaged ($I_{bd} > 1.0$) the damage index of the structure is defined only by the seat-width index of the structure.

The first of the preceding two cases deserves additional explanations. Consider two bridges, A and B, with identical bearing damage indices of 0.7 but seat-width indices of 1.4 and 0.5, respectively. In the case of bridge A, since the bearings are not likely to be severed, the structure is unlikely to slide. Therefore, the fact that the seat-width index is larger than 1.0 does not pose any danger, and the bearing-damage index is selected as the overall damage index for bridge A. In the case of bridge B, the seat-width index is smaller than the bearing-damage index, indicating that this bridge is even "safer" than indicated by the bearing-damage index (i.e., even if the bearings were severed, the structure would not fall off its supports). Therefore, the seat-width index is the proper overall damage index in this latter case. It is particularly important to preserve a correct relative ranking, even among bridges presently identified as unlikely to fail, in the perspective that seismicity maps (and local seismicity conditions obtained from site-specific determination) are still evolving and still subject to change in many regions of North America.

Continuous Bridges

Bearings and columns are the most vulnerable superstructure components of continuous bridges. Damage to columns may result in the total failure of structure, but damage to bearings does not have significant consequences unless the structure slides excessively. When sliding occurs, the structure may fall off its support if there is not adequate seat width, or the columns may be damaged due to large displacements at the columns' locations produced by the combined effect of sliding and elastic deformation of the structure. Accordingly, the overall damage index, I_d of the structure is defined considering three of the following possible cases:

1. If $I_{bd} < 1.0$ and $I_{bd} < I_{cd}$, then $I_d = I_{cd}$.
2. If $I_{bd} < 1.0$ and $I_{bd} > I_{cd}$, then the consequences of damage to bearings should be investigated. Accordingly, assuming that the bearings are damaged, seat-width and column-damage indices I^*_{sw} and I^*_{cd} are calculated, and the larger of these is selected as the temporary damage index, I^*_d. Then, overall damage index of the structure is determined considering the following three possible outcomes:
   - If $I^*_d > I_{bd}$, then $I_d = I_{bd}$.
   - If $I_{cd} < I^*_d < I_{bd}$, then $I_d = I^*_d$.
   - If $I^*_d < I_{cd}$, then $I_d = I_{cd}$.
3. If $I_{bd} > 1.0$, then I_d is determined by the larger of I_{cd} or I_{sw}.

In the first of the previous three cases, since the column-damage index is larger than the bearing-damage index and the bearings are not damaged, the failure of the structure can only result from damage to the columns, and therefore the column-damage index is the overall damage index of the structure. The second case is explained by the following example. Consider two bridges, A and B, with identical bearing and column-damage indices of 0.95 and 0.60, respectively. Since the bearing and column-damage indices are identical for both bridges, they may be ranked as equally vulnerable if the consequences of damage to bearings are not considered. Now assume that bridge A has a very low friction coefficient at the bearings, and therefore it may have large sliding displacements if the bearings are severed, whereas bridge B has a larger friction coefficient at the bearings, and therefore the sliding displacements are smaller (Dicleli and Bruneau 1995b). Assuming that the bearings are severed, the seat-width and column-damage indices are calculated as 1.4 and 1.1 for bridge A and 0.4 and 0.8 for bridge B. Accordingly, the temporary damage index is the larger of the seat-width and column-damage indices and is 1.4 and 0.8 for bridges A and B, respectively. For bridge A, since the temporary damage index is larger than 1.0, the result of damage to bearings is the total failure of the structure. Consequently, the bearings are the fuse elements of the bridge, and therefore the bearing-damage index is selected as the overall damage index. For bridge B, the consequence of damage to bearings is only to increase the risk of damage to the structure from 0.6 to 0.8. Accordingly, the temporary damage index is selected as the overall damage index of the structure.

It is noteworthy that, in some occasions, sliding may produce less displacements at the column locations than those produced by the elastic deformation of the structure before the bearings are damaged. This may happen if the ground motion has a high frequency content or high A_p/V_p ratio (Dicleli and Bruneau 1995b). Consequently, the column-damage index calculated assuming that the bearings are severed may be smaller than the one calculated assuming that bearings are not severed. The third element of the second case addresses this particular aspect of behavior.

In the third outcome of the previous three cases, the behavior of the structure after the bearings are severed is considered. In this case, the structure may get damaged either if it falls off its support or if the columns are damaged. Therefore, the overall damage index of the structure is determined by the larger of the seat-width or column-damage indices.

Multispan Simply Supported Bridges

In the case of multispan simply supported bridges, damage to the bearings on the columns may create an unstable structure and result in failure. Inadequate seat width and column capacity are also equally responsible for the failure of the structure. Accordingly the largest of the seat-width, bearing, and column-damage indices defines the overall damage index.

ALTERNATIVE SIMPLIFIED APPROACH FOR OVERALL DAMAGE INDEX

An even more rapid and conservative evaluation of damage indices can be used by taking the larger of I_{cd} or I_{sw} as the overall damage index for any particular bridge, assuming that bearings will always be damaged by a severe earthquake. Furthermore, if bearings are of a type that is unable to sustain any damage in a stable manner, then the overall damage index of the bridge should be taken as the largest of I_{bd}, I_{sw}, and I_{cd} calculated, assuming undamaged bearings.

RANKING INDEX

The prioritization of seismic retrofitting of bridge structures is based on the calculation of a ranking index defined as the product of the importance and damage indices. According to this, bridges with a higher ranking index have higher priority for retrofitting. At first glance, this approach appears logical and properly considerate of other nonstructural but important societal issues. It is similar in this respect to the other existing methodologies. However, there is a philosophical deficiency in its application if left without a cut-off mechanism to alleviate the potential undue impact of dominating societal aspects in existing bridges with already existing excellent seismic-resistance adequacy. To correct this inconsistency and to ensure that the perceived priorities concur with actual needs, it is suggested that the ranking index be set to zero when the damage index falls below a certain value, i.e., a cut-off value. Recall that damage indices, calculated using the equations proposed earlier, are functions of peak ground acceleration at the site obtained from a seismic map. However, there is always a risk that, at any site, peak ground acceleration specified in probabilistic seismic maps are exceeded. Therefore, the upper-bound value below which the damage index can be assumed to be zero must be calculated by relating the risk level adopted in current seismic maps and a lower predetermined acceptable risk for the cut-off level. For example, in North America, seismic maps are generally constructed assuming a 10% probability of exceedance in 50 years. In the present study, a 5% probability of exceedance in 50 years is chosen as an appropriate lower-risk level to trigger the aforementioned cut-off value. To obtain this cut-off value, the ratios, R_A, of the peak ground accelerations, A_{p5}, for a 5% probability of exceedance in 50 years to A_{p10} for a 10% probability of exceedance in 50 years are obtained for various seismic regions where the earthquake magnitude M_{10} is greater than 5.0. These calculations are performed using seismologic maps where each seismic region is attributed a single Richter's law relationship. Such regions must be defined prior to using the Cornell McGuire method to develop the equal probability of exceedance contour maps commonly found in building codes (Basham et al. 1983). Then, each R_A is multiplied by the area of the seismic region and the results are summed up. This sum is divided by the total area of all regions to obtain a weighted average of the peak acceleration ratios. This is equivalent to calculating an average seismic-risk exposure of the bridges inventory, assuming that the number of bridges in these seismic regions is proportional to the size of the region. Moreover, it also attenuates the impact of extreme R_A values applicable only over very small seismic regions.

For example, the weighted average, R_{wa}, of the acceleration ratios is obtained as 1.239 for western Canada. This shows that when the probability of exceedance is reduced from 10 to 5% in 50 years, the site peak ground acceleration increases approximately 25%. Consequently, structures with a damage index of 0.80 (i.e., 1/1.239) are considered to have reached the targeted cut-off risk of damage. Accordingly, the ranking index proposed in the present study is defined as

$$I_R = \begin{Bmatrix} I_i I_d & \text{if } I_d \geq 0.8 \\ 0.0 & \text{if } I_d < 0.8 \end{Bmatrix} \tag{30}$$

It is noteworthy that this cut-off value can be obtained for any other seismic regions with magnitude-frequency and acceleration-attenuation relations different than those found in western Canada, but the previously proposed cut-off value is believed to be a reasonable broadly applicable value. Two examples illustrating the proposed ranking procedure are presented in the following sections.

EXAMPLES

Two examples are considered to illustrate the preceding methodology. The first example (example 1) is a three-span continuous highway bridge, whereas the second one (example 2) is a three-span simply supported bridge. The specifics of these bridges are summarized in Table 1. In both bridges, sliding-type bearings support the superstructure under each girder. Each bearing has a 90 × 400 × 220 mm bearing bar and a 230 × 400 × 50 mm base plate anchored by two 32 mm diameter, 400 mm long anchor bolts with 230 MPa shear yield strength. The capacity of each bearing, calculated using (17), is 370 kN. The translational stiffness of each bearing in the longitudinal direction is 320,000 kN/m and the translational and rotational stiffness of the bearing group is calculated to be 1.92×10^6 kN/m and 51×10^6 kN/m, respectively. A base friction coefficient of 0.4 is proposed between the concrete abutment and the bearings. Finally, all steel columns of those bridges are oriented to develop strong axis bending in the longitudinal direction, and importance indices are arbitrarily selected. Intermediate and final results are summarized in Table 2.

For the first example, all calculations are straightforward applications of the previous equations, as indicated in the table. The bearing-damage index is first calculated and found to

TABLE 1. Physical Properties of Bridges Considered in Examples 1 and 2

Components of damage-index calculations (1)	Example 1 (2)	Example 2 (3)
Number of lanes	4	2
Number of spans	3	3
Span lengths (m)	35, 25, 30	35, 43, 37
Superstructure		
Width (m)	16	7.4
Girder sections	6WWF1200 × 333	4WWF1200 × 333
Girder spacing (m)	3	2
Structure design	composite	composite
Slab thickness	190	175
Pavement thickness (mm)	70	65
Mass (tons/m)	12.56	5.88
A_D (m^2)	0.592	n/a
I_D (m^4)	13.9	n/a
Expansion joints width (mm)	n/a	25, 35, 30
Columns		
Section	W310 × 79	WWF400 × 178
Height (m)	6	5.5
M_{AY} (kN · m)	115	415
M_{AX} (kN · m)	295	1,207
k_{cT} (kN/m)	111	847
k_{cL} (kN/m)	492	2,475
P_D (kN)	683	612
β_{my}	n/a	1.151
β_{mxy}	n/a	1.052
Column bent		
K_T (kN/m)	666	3,388
K_L (kN/m)	2,952	9,888
Seat widths		
Transverse (mm)	85	110
Longitudinal (mm)	175	175
A_{PS} (g)	0.3	0.3
I_i	0.9	0.8

Note: n/a means not applicable.

TABLE 2. Results for Examples 1 and 2 (Refer to Equation Number Used to Calculate Results)

Components of damage-index calculations (1)	Example 1 (2)	Example 2 (3)
Direction	Transverse Longitudinal	Transverse Longitudinal
Bearing-damage-index calculation		
T (s)	0.3 [Eq. (5)] 0.165 [Eq. (6)]	1.26 [Eq. (25)] 1.0 [Eq. (20)]
β	2.50 2.24	0.99 1.25
b_{ry} [Eq. (14)] (kN)	563	n/a
b_{rxy} [Eq. (15)] (kN)	1,320	n/a
b_{rx} [Eq. (16)] (kN)	1,068	n/a
B_r [Eq. (13)]		
Load case 1 (kN)	1,589	n/a
Load case 2 (kN)	1,329	n/a
I_{bd} [Eq. (12)]	4.30	2.73
Column-damage index calculation		
M_{EY} (kN · m)	70 [Eq. (24)]	942 [Eq. (27)]
M_{EXY} (kN · m)	n/a	204 [Eq. (28)]
M_{EX} (kN · m)	148 [Eq. (23)]	926 [Eq. (29)]
I_{cd} [Eq. (21)]		
Load case 1	0.36	1.93
Load case 2	0.68	1.28
Direction	Transverse Longitudinal	n/a
Seat-width damage index calculation		
T (s)	0.34 [Eq. (5)] 0.118 [Eq. (6)]	n/a
β	2.50 1.89	n/a
A_p [Eq. (4)] (g)	0.07 0.051	n/a
ψ [Eq. (3)] (kN)	0.389 0.211	n/a
u_s (Fig. 2) (mm)	46 45	n/a
I_{swT} [Eq. (1)]	1.13 —	n/a
I_{swL} [Eq. (2)]	— 0.86	n/a
Resulting indices		
I_{Sw}	1.13	1.05
I_{cd}	0.76	1.93
I_{bd}	4.30	2.73
Damage index		
I_d	1.13	2.73
Ranking index		
I_R [Eq. (30)]	1.02	2.18

Note: n/a means not applicable.

exceed 1.0. Hence, the bearings are taken as damaged, and the peak ground acceleration required to trigger sliding is calculated. To that end, the fraction of the superstructure weight transferred to the supports is estimated approximately using the tributary area of each end span. The resulting fraction of the weight at the left and right abutments are 17.5/90 = 0.195 and 15/90 = 0.167, where 90 is the total end-to-end length of the bridge. Therefore, the proportion of the weight effective for resisting the sliding in the transverse direction is 0.36 (i.e., 0.195 + 0.167) and 0.195 to resist longitudinal direction sliding. Using this, the seat-width damage indices for the transverse and longitudinal directions are calculated, as shown in Table 2. Note that since the peak ground accelerations, A_p, required for sliding in both orthogonal directions are less than the site peak ground acceleration of 0.3 g, the superstructure will slide if the bearings are damaged. Sliding displacements, seat-width indices, and other intermediate results for the transverse and longitudinal directions are tabulated. Then, the column damage indices are calculated directly using (23), (24), and (21) for the two load cases specified. In this example, the damage index is the larger of the seat-width and column-damage indices. It is noteworthy that conservatively assuming that the bearings are damaged, the simplified alternative approach yields the same result.

Calculation of the bearing-damage index for the second example follows a different procedure. First, the two adjacent spans with the largest average length must be selected (i.e., those having 43 and 37 m lengths in this case). The total mass of this subassembly is calculated to be 470 tons. Then, (25) and (20) are used to determine the transverse and longitudinal direction fundamental periods of the selected subassembly, the column-fixed deck with the longest span, and the corresponding β values. Finally, using (18) and considering the deck with the longest span, the minimum required peak ground acceleration for collision to occur is obtained as

$$A_{p_c} = \frac{n_c k_{cL} EJW}{\beta m_D g}\left(1 - \frac{P}{k_{cL} h_c}\right) = \frac{4 \times 2{,}472 \times 0.035}{1.25 \times 253 \times 9.81} \times \left(1 - \frac{612}{2{,}472 \times 5.5}\right) = 0.11g \quad (31)$$

Dividing the peak ground acceleration of the site by this acceleration, the bearing-damage index is obtained as 2.73. As for the seat-width index, knowing that the seat width is 175 mm at every bent, it is only necessary to check the bent that supports the longest span (i.e., the most critical one). Using (10) and (11), the minimum required seat width is obtained as 183 mm at the expansion joint adjacent to the longest span, and, as per (9), the resulting seat-width index is 1.05. Calculation of the column-damage index proceeds as for the previous example, except that different equations are used to calculate the seismic moment demands. The resulting damage index of this bridge is selected as the larger of bearing damage, seat-width, and column-damage indices.

CONCLUSION

In this paper, a rapid seismic evaluation and ranking methodology for steel highway bridges has been introduced. It is more complex than other existing methodologies that are limited to simple recognition of undesirable structural features known to have performed inadequately in past earthquakes. However, the proposed methodology is based on a quantitative approach that takes advantage of knowledge on the elastic and inelastic seismic response of these types of bridges, and addresses the risk inherent to all seismic hazard zones. Contrary to other existing methodologies, the overall damage index of the structure considers the impact of damage to each component on the successive failure of other components and the structure as a whole. A cut-off mechanism is also introduced to prevent the potentially undue impact of dominating societal aspects in existing bridges with otherwise excellent seismic-resistance adequacy.

It is also possible to use the proposed methodology as a second-level evaluation to estimate the seismic performance of a class of steel bridges without the need for complex modeling and nonlinear analyses. For this purpose, simple question-and-answer type of computer programs can easily be developed using the methodology presented in this paper. Finally, the approach proposed here could be followed as a model to prioritize seismic retrofit activities for other bridge types.

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