Reliability-Based Dynamic Load Allowance for Capacity Rating of Prestressed Concrete Girder Bridges

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Abstract: The current highway bridge design in the United States follows the AASHTO-LRFD specifications, which prescribe a dynamic load allowance, *IM*, of 0.33 for the dynamic effect of truck/tandem loading. Studies have shown that the *IM* value prescribed by the LRFD code may underestimate this dynamic effect under poor road surface conditions (RSCs). One reason for this underestimation is that the *IM* value employed in the AASHTO specifications was obtained from the statistical properties of the *IM* relative to average RSC, as defined by the ISO 1995 standards. In addition, the *IM*, which is a random variable with certain statistical properties, was modeled as a deterministic constant in the code calibration process. In this paper, the reliability indexes of a selected group of prestressed concrete girder bridges, designed following the AASHTO-LRFD code, are calculated by modeling the *IM* explicitly as a random variable for different RSCs. It is found that although the calculated bridge reliability indexes are usually above the target reliability index value of 3.5 under above-average RSCs, they can be significantly below the target value of 3.5 when the RSCs are below average. Following the load rating procedure proposed by the AASHTO load and resistance factor rating (LRFR) manual, it is also found that the code-employed *IM* value may overestimate the rating factors when RSCs are below average. Based on these results, appropriate *IM* values are suggested for different RSCs to achieve a consistent target reliability index and a reliable load rating. The results presented in this paper are particularly valuable for the rating of existing prestressed concrete girder bridges, for which the actual RSCs can be directly evaluated. The RSCs must be properly taken into account to accurately estimate the actual safety of the considered bridge. **DOI:** 10.1061/(ASCE)BE.1943-5592.0000178.

CE Database subject headings: Load and resistance factor design; Reliability; Prestressed concrete; Girder bridges; Ratings.

Author keywords: Dynamic load allowance; Load and resistance factor design (LRFD); Road surface condition; Load and resistance factor rating (LRFR); Reliability index.

Introduction

The current highway bridge design in the United States follows the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2004). The LRFD code has two significant advantages over the previous *Standard Specifications for Highway Bridges* (AASHTO 2002) and general allowable stress design procedures (Nowak and Collins 2000). First, the variability of both the resistance and load effects is taken into consideration, and appropriate resistance and load factors are used. Second, more uniform levels of safety can be achieved for different limit states and bridge types. The LRFD code sets the basic design formula in the following form:

$$\phi R \geq \sum \gamma_{Q_i} Q_i \tag{1}$$

where ϕ = resistance factor; R = nominal resistance; and Q_i = effects of the ith design load component with load factor γ_{Q_i}. The load and resistance factors in Eq. (1) were calibrated so that the safety of different bridges designed according to the code should be at the same preselected target level, which is usually measured by the reliability index β (Nowak 1995). A reliability index value $\beta = 3.5$ is targeted in the AASHTO-LRFD code.

Corresponding to the LRFD philosophy, AASHTO has developed a load and resistance factor rating (LRFR) manual (AASHTO 2003) for bridge rating. Currently, the AASHTO-LRFR methodology is used for bridges that were designed based on LRFD codes. It is noteworthy that for bridges designed based on allowable stress or load factor design using the AASHTO standard specifications, the traditional *Manual for Condition Evaluation of Highway Bridges* (AASHTO 1994) is being used.

The design formula provided by the AASHTO-LRFD code suggests a constant value of 0.33 for the (vehicular) dynamic load allowance, *IM* (given as dynamic load allowance percent in the code), which provides the relative increment of the static effects produced by the truck/tandem live loads due to the dynamic effects on bridges. This *IM* value is based on the study by Hwang and Nowak (1991), in which the statistical model of the dynamic effect of vehicle loads was obtained from the numerical simulation of the dynamic behavior of bridges under vehicle loading. However, over the past few decades, numerous numerical simulations and field testing showed that the value of *IM* prescribed by the LRFD code may underestimate the actual *IM* for short bridges when the road surface conditions (RSCs) are poor (Billing 1984; O'Connor and Pritchard 1985; Shi et al. 2008). It is noteworthy that the numerical simulations presented in Hwang and

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Note. This manuscript was submitted on March 20, 2010; approved on September 1, 2010; published online on March 12, 2011. Discussion period open until April 1, 2012; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Bridge Engineering***, Vol. 16, No. 6, November 1, 2011. ©ASCE, ISSN 1084-0702/2011/6-872–880/$25.00.

Nowak's work were performed assuming a road roughness coefficient of 0.64×10^{-6} m^3/cycle, which corresponds to an average RSC according to the ISO (1995) standard. The use of this value for the road roughness coefficient in the numerical simulation resulted in a mean IM of less than 0.17 with a coefficient of variation of 0.80 for the single truck case (Hwang and Nowak 1991).

A roughness coefficient corresponding to average RSC does not represent the RSCs of all existing bridges and may result in biased estimates of the IM. In particular, the values of the IM can be significantly underestimated for bridges with below-average RSCs. However, for bridges with smooth road surfaces, the IM suggested in the code may overestimate the actual dynamic effects due to truck loads. For this reason, the commentary to the LRFR manual (AASHTO 2003) suggests $IM = 0.1$ for smooth riding surface at approaches, bridge decks, and expansion joints, and $IM = 0.2$ for minor surface deviations or depressions. Furthermore, in the calibration process of the LRFD code (Nowak 1995), the live load and the dynamic effect of the vehicle load were modeled as a single combined random variable with a coefficient of variation of 0.18. Because the IM has a coefficient of variation as large as 0.80 (Hwang and Nowak 1991), the use of a coefficient of variation of 0.18 for the combined random parameter may underestimate the true variability of the dynamic effect of the live load and, thus, lead to biased estimates of the reliability indexes.

In this paper, using the statistical properties of the IM developed in a previous study, the reliability indexes for a selected group of prestressed concrete girder bridges are calculated for different RSCs by explicitly modeling the IM as an individual random variable. The obtained reliability index values are compared with the target value $\beta = 3.5$ that is assumed in the AASHTO-LRFD code. An inventory load rating is also performed for the bridges considered in this study following the equation provided in the LRFR manual (AASHTO 2003). Appropriate IM values are proposed with respect to bridge RSCs to achieve a consistent target reliability index and a reliable load rating. Finally, the newly proposed IMs are compared with the IMs suggested in a previous study (Deng and Cai 2010), in which those values were obtained based on probabilistic considerations similar to the ones used to define safety coefficients.

Description of the Benchmark Bridges

In the present study, seven typical prestressed concrete girder bridges with span lengths ranging from 9.14–39.62 m (30–130 ft) are investigated. These bridges were used in a previous study (Deng and Cai 2010). Five of the seven bridges (i.e., Bridges 1, 2, 5, 6, and 7) consist of five identical simply supported girders with girder spacing of 2.13 m (7 ft), and have a roadway width of 9.75 m (32 ft) and a bridge deck thickness of 0.20 m (8 in.). A typical cross section of these five bridges is shown in Fig. 1. In addition to end diaphragms, intermediate diaphragms are also used to connect the five girders depending on their span lengths (see Table 1).

The other two bridges (i.e., Bridges 3 and 4) have different girder spacing and cross-section width compared to the other five bridges and were selected to study the effect of the girder spacing and bridge width on bridge reliability. Both bridges have the same span length as Bridge 2 (16.76 m). Bridge 3 was obtained from Bridge 2 by increasing the girder spacing from 2.13 to 2.90 m, and Bridge 4 was obtained from Bridge 2 by adding two more girders and keeping the girder spacing unchanged. These modifications led to a total width of 14.33 m (47 ft) for both bridges. Table 1 shows the detailed properties of the seven bridges used in this study.

Load and Resistance Models

Bridges in service are subjected to a combination of different loads, including dead loads, live loads, impact loads, environmental loads (e.g., wind, snow, earthquake, temperature), and special loads (e.g., collision loads). According to the AASHTO-LRFD bridge design

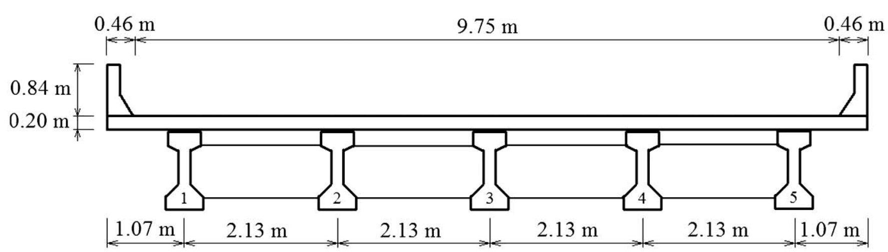

Fig. 1. Typical cross section of bridges considered in this study

Table 1. Detailed Properties of Benchmark Bridges

Bridge number	Span length (m)	Fundamental natural frequency (Hz)	Girder: AASHTO type	Girder: Cross-sectional area (m^2)	Girder: Inertia moment of cross section (10^{-2} m^4)	Number of intermediate diaphragm
1	9.14	15.508	II	0.238	2.122	0
2	16.76	6.581	II	0.238	2.122	1
3	16.76	6.114	II	0.238	2.122	1
4	16.76	6.642	II	0.238	2.122	1
5	24.38	4.598	III	0.361	5.219	1
6	32.00	3.203	IV	0.509	10.853	2
7	39.62	2.664	V	0.753	32.859	2

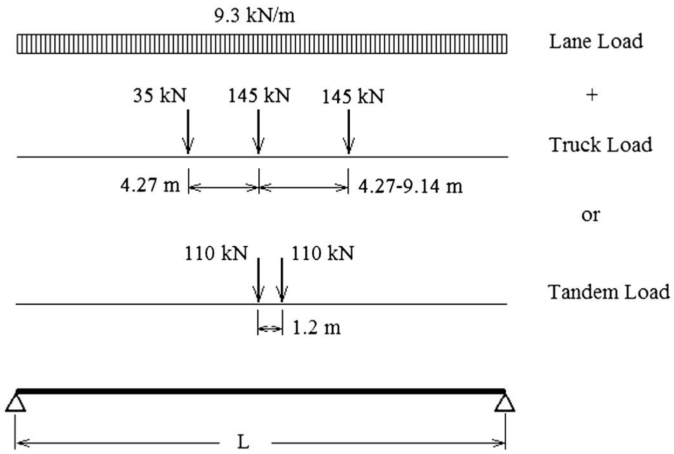

Fig. 2. Live loads prescribed by the AASHTO-LRFD code

Table 2. Statistical Properties for Load and Resistance Used in the Calibration of AASHTO-LRFD Code

Variable		Bias	COV	Distribution type
Dead load	Precast concrete	1.03	0.08	Normal
	Cast-in-place concrete	1.05	0.10	Normal
	Asphalt	1.00	0.25	Normal
Live load	Moment	1.27–1.36	0.18	Extreme Type I
	Shear	1.20–1.28	0.18	Extreme Type I
Resistance	Moment	1.05	0.075	Lognormal
	Shear	1.165	0.16	Lognormal

specifications (2004), the design loads for the concrete girders should consider the combination of dead loads and live loads. The dead loads consist of the weight of the structural and nonstructural components of the bridge, e.g., girders, deck, wearing surface, and railing, and can be calculated by using the volumes of the components and specified densities of the materials. For the live loads, the maximum of the "lane load + truck load" and "lane load + tandem load" must be considered. Fig. 2 shows the live loads prescribed by the AASHTO-LRFD code.

The resistance of a bridge is primarily determined by the material strength and the dimensions of its components. Many bridge resistance models are available in the literature for different applications. For purposes of comparison and consistency with the AASHTO-LRFD code, the load and resistance models used by Nowak (1995) were employed in the present study, with the statistical model of the moment resistance given in Nowak et al. (1994) and the statistical parameters of the loads available in Nowak (1993, 1995). The statistical characterization of the load and resistance models, including the bias, coefficient of variation (COV), and distribution type for the load and resistance, are shown in Table 2. For the road wearing surface, an asphalt surface with mean thickness of 75 mm is used in the present study, consistent with Nowak (1995).

Reliability Analysis

Structural reliability analysis requires the definition of a limit state function, also called performance function. In the AASHTO code, the limit state function, denoted as g, is expressed as the difference between the random resistance of the structure (also called capacity), C, and the random load effect (also called demand) on the structure, D, as

$$g = C - D \tag{2}$$

Several random variables are involved in the above limit state function, representing all pertinent sources of variability of the resistance, such as cross-sectional dimensions and material properties, and of the load effects on the structure. The limit state function identifies a failure domain ($g \leq 0$), a safe domain ($g > 0$), and a failure surface ($g = 0$) in the domain of all random variables. The probability of failure (P_f), defined as the probability content of the failure domain, is usually represented by the reliability index, β. Using the first-order reliability method (Ditlevsen and Madsen 1996), the failure probability and the reliability index satisfy the following relation:

$$P_f = \Phi(-\beta) \tag{3}$$

where $\Phi(\ldots)$ denotes the standard normal cumulative distribution function.

A number of procedures are available to calculate the reliability index in the literature (Rackwitz and Fiessler 1978; Thoft-Christensen and Baker 1982; Madsen et al. 1986; Ditlevsen and Madsen 1996; Ayyub and McCuen 1997; Estes and Frangopol 1998; Nowak and Collins 2000). In the present study, the iterative Rackwitz-Fiessler algorithm is employed to calculate the bridge reliability index. This procedure requires the knowledge of the joint probability density function for all the random variables. The details of the Rackwitz-Fiessler algorithm can be found in Nowak and Collins (2000).

Recalculated Reliability Indexes for the Selected Bridges

In the calibration procedure of the current LRFD code (AASHTO 2004), if only dead loads and vehicle live loads are considered, the following design equation is recommended:

$$\phi R \geq 1.25DC + 1.50DW + 1.75LL_l + 1.75(1 + IM)LL_t \tag{4}$$

where R = nominal value for the resistance; DC = effects due to design dead loads excluding the weight of wearing surface; DW = effects due to wearing surface weight; LL_l = effects due to design lane load; LL_t = effects due to the most demanding between truck and tandem load; $IM = LL_{t,\mathrm{dyn}}/LL_t - 1 = 0.33$ denotes the dynamic load allowance, in which $LL_{t,\mathrm{dyn}}$ = effects due to the most demanding between truck and tandem load including the dynamic effects; and ϕ = resistance factor. For prestressed concrete bridges, the current AASHTO-LFRD code suggests a resistance factor of 1.0 and 0.85 for moment and shear, respectively, based on the study by Nowak (1995).

The statistical properties of the IM were obtained in a previous study (Deng and Cai 2010), in which five concrete girder bridges (Bridges 1, 2, 5, 6, and 7 in Table 1) were used. Five different RSCs as defined by the ISO (1995) standard and seven different vehicle speeds ranging from 30–120 km/h were considered to cover most of the conditions encountered in actual bridges. Two vehicle loading conditions were considered, with one truck or two trucks side-by-side traveling across the bridge. It was found that the two loading conditions produce IMs with very close statistical properties. The distribution type of the IMs was determined by performing the chi-square test on the IM data. The test results indicate that an Extreme Type I distribution is appropriate for the IM for each of the five road surface conditions considered. The statistical properties of the IM corresponding to the one-truck case were used in the present study and are summarized in Table 3. The original data can be found in Deng (2009).

Table 3. Statistical Properties of Dynamic Load Allowance Used in the Present Study

Road surface condition	Dynamic load allowance		
	Mean	COV	Distribution type
Very poor	0.94	0.52	Extreme Type I
Poor	0.41	0.53	Extreme Type I
Average	0.22	0.60	Extreme Type I
Good	0.12	0.57	Extreme Type I
Very good	0.08	0.64	Extreme Type I

By using the statistical properties for load/resistance parameters and *IM* given in Tables 2 and 3, respectively, and assuming that the section capacity of all seven bridges is perfectly designed (neither overdesigned nor underdesigned) based on Eq. (4), the reliability indexes of the seven selected bridges and the five RSCs considered here are calculated using the Rackwitz-Fiessler algorithm, based on the following strength limit state function, g_2:

$$g_2 = C - D_{D,p} - D_{D,c} - D_{D,A} - D_{L,l} - (1+\zeta)D_{L,t} \qquad (5)$$

where C = random variable representing the capacity (moment or shear); $D_{D,p}$ and $D_{D,c}$ = random variables representing the effects due to the dead loads of prestressed concrete and cast-in-place concrete bridge components, respectively; $D_{D,A}$ = random variable representing the effects due to the wearing surface weight; $D_{L,l}$ and $D_{L,t}$ = random variables representing the effects due to lane load and the most demanding between truck and tandem load, respectively; and ζ = random variable representing the *IM*. It is noteworthy that introducing the random variable ζ for the *IM* is the only difference between Eq. (5) and the strength limit state function used by Nowak (1995) in calibrating the LRFD bridge design code.

Fig. 3 shows the newly calculated reliability indexes for both moment and shear using the strength limit state function given in Eq. (5). Based on the results presented in Fig. 3, the following observations are made: (1) the current AASHTO-LRFD code produces relatively consistent reliability indexes for the seven bridges under study, especially for moment, for each given RSC; (2) the calculated reliability indexes of the seven bridges under study are above the target 3.5 when the RSCs are above average; (3) for below-average RSCs, all reliability indexes for the seven bridges are below 3.5 for moment strength limit state, and most of the reliability indexes are below 3.5 for shear strength limit state; and (4) both the girder spacing and bridge width have a very small effect on the calculated bridge reliability index, as can be seen from comparison of the calculated reliability indexes of Bridges 2, 3, and 4. These results are consistent with what has been observed by other researchers (Billing 1984; O'Connor and Pritchard 1985; Shi et al. 2008).

Using the mean values of the *IM* under different RSCs given in Table 3, an inventory load rating was performed for the seven benchmark bridges considered in this study, following the equation provided in the LRFR manual (AASHTO 2003):

$$RF = \frac{C - \gamma_{DC}DC - \gamma_{Dw}DW}{\gamma_L LL_l + \gamma_L(1+IM)LL_t} \qquad (6)$$

where RF = rating factor; C = capacity; and γ_{DC}, γ_{DW}, and γ_L = LRFD load factors corresponding to DC, DW, and live loads, respectively.

Fig. 4 shows the inventory load rating results considering both the moment and shear strength limit states. It is noteworthy that the *RF* for all the seven bridges would be 1.0 if the value of $IM = 0.33$ were used in the rating process. It should also be pointed out that the AASHTO code specifications do not distinguish the *IM* in inventory rating and operating rating. Therefore, the methodology and results presented in this paper are valid for both inventory rating and operating rating. Fig. 4 clearly shows that employing the dynamic load allowance $IM = 0.33$ suggested by the code without accounting for the actual RSCs produces significantly inaccurate *RF*s compared to the *RF*s obtained when actual RSCs are considered. In particular, using $IM = 0.33$ overestimates the *RF*s (1) for below-average RSCs in the case of the moment strength limit state, and (2) for very poor RSCs in the case of shear strength limit state.

Proposed Dynamic Load Allowance *IM* for Existing Bridges

The results shown in Figs. 3 and 4 clearly indicate that use of the *IM* value of 0.33 specified by the LRFD code and LRFR manual can produce significantly inaccurate estimates of the reliability index and RF of existing bridges. In particular, for bridges with below-average RSCs (e.g., old bridges whose RSCs have deteriorated because of factors such as aging, corrosion, and increased gross vehicle weight), the reliability indexes and *RF* can

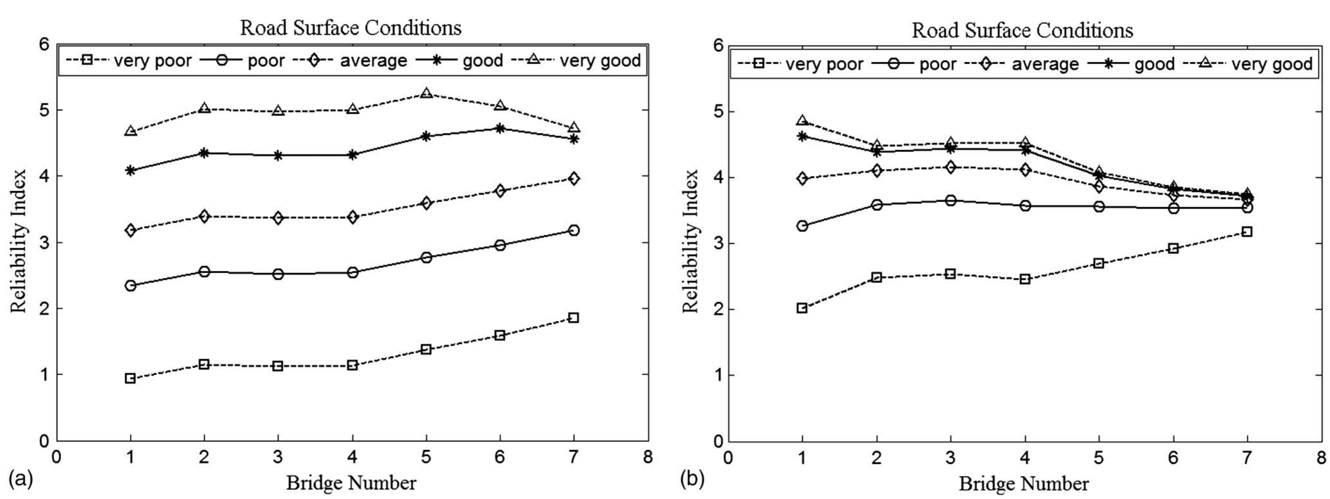

Fig. 3. Reliability indexes computed by using Eq. (5) for the seven bridges designed based on Eq. (4) and according to the five road surface conditions defined in ISO 1995: (a) moment; (b) shear strength limit states

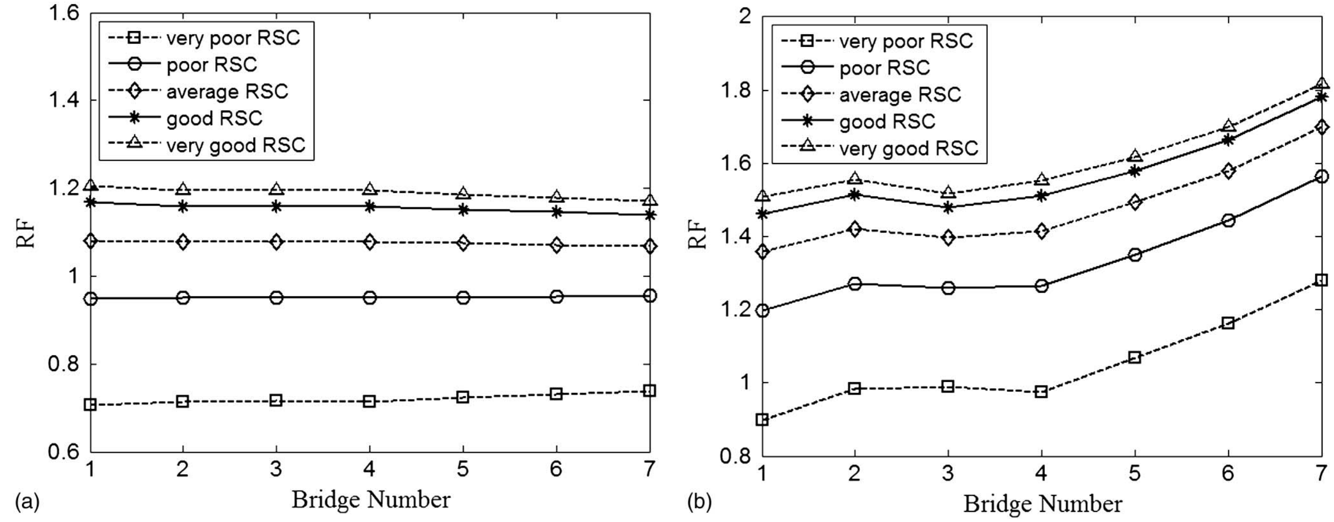

Fig. 4. Load rating factors using the mean values of *IM* under different RSCs: (a) moment; (b) shear strength limit states

be significantly overestimated. Results from bridge field tests have shown that the actual *IM* could be much higher than 0.33. O'Connor and Pritchard (1985) observed that the *IM* values varied from −0.08 to 1.32 during the field tests conducted on a composite girder highway bridge. Kwasniewski et al. (2006) evaluated the dynamic effect on a field bridge. With the actual RSC, *IM* values as high as 0.82 and 0.51 were recorded for the one-truck and two-trucks cases, respectively. By placing a wooden plank on the road surface to represent severely deteriorated RSCs at bridge ends, an *IM* value as high as 1.64 was observed.

According to a recent report by the U.S. Department of Transportation (2002), more than 5% of all bridge decks in the United States have poor conditions or worse, and another 12% of all bridge decks have fair conditions. The fair condition here is interpreted as the average or close to average condition defined by ISO (1995). Based on another recent report by AASHTO (2008), the average age of bridges in the United States has reached 43 years. The proportion of bridges with poor bridge decks is expected to increase because of the aging of the bridge infrastructure system. Moreover, the weight of additional wearing surfaces for maintenance purposes adds more loads to the bridge and, thus, can further reduce the bridge reliability. Therefore, to ensure a consistent safety on all bridges independently of their RSCs, larger *IM*s should be employed to assess the safety of these old bridges.

In practical applications for bridge ratings, the profile of the bridge surface in the field can be measured by using a laser profiler. The road surface condition can then be determined by analyzing the measured road surface profile based on the guidance provided by the ISO. When the engineer has no information about the road surface condition, an average road surface condition can be assumed, and a dynamic impact factor value $IM = 0.33$ (corresponding to the value suggested by the code) can be used.

To determine appropriate *IM* values that can lead to the target reliability index of 3.5 specified by the LRFD code for each RSC, a few selected *IM* values were checked and used to recalculate the reliability indexes of all bridges. Both moment and shear strength limit states were considered in this work, and the results are shown in Figs. 5 and 6 for moment and shear strength limit states, respectively.

Figs. 5(a)–5(e) show the newly calculated reliability indexes of all seven bridges under all five RSCs for the moment strength limit state using properly selected *IM* values that produce reliability indexes close to the target reliability index of 3.5. Although an *IM* value of 0.33 is enough to achieve a reliability index of 3.5 for very good and good RSCs, larger *IM* values are needed for average or below-average RSCs. In particular, for the moment strength limit state, an *IM* value as small as 0.01 is enough to achieve a reliability index of 3.5 or above for all the seven bridges considered when the RSC is very good, and an *IM* value between 2.40 and 2.60 is needed to achieve a reliability index of 3.5 for all the seven bridges considered when the RSC is very poor.

Figs. 6(a)–6(e) show the newly calculated reliability indexes relative to the shear strength limit state for all seven bridges and five RSCs. It is observed from these five subfigures that the required *IM* values to achieve the target reliability index of 3.5 for the shear strength limit state are considerably smaller than the corresponding required *IM* values for moment. For average and above-average RSCs, the code-prescribed *IM* of 0.33 is enough to achieve a reliability index of 3.5 for all bridges considered. However, for very poor RSC, an *IM* value of 1.90 is needed to achieve the same target reliability index.

Based on the results shown in Figs. 5 and 6, appropriate *IM* values were suggested to be used in the assessment of prestressed concrete girder bridges for different RSCs (see Table 4). To verify whether the proposed *IM* can lead to a consistent reliability index of 3.5 for all RSCs, the reliability indexes were recalculated by using the limit state function in Eq. (5) for the seven bridges designed following Eq. (4) and employing the proposed *IM* values. The recalculated reliability indexes are shown in Figs. 5(f) and 6(f) for the moment and shear strength limit states, respectively. These results confirm that using the proposed *IM* values produce more consistent reliability indexes that are very close to the target reliability index of 3.5, regardless of what the actual RSC is.

Table 4 provides, for all the RSCs considered, (1) the minimum values of *IM* required to reach the target reliability index of 3.5 for both moment and shear strength limit states for all the bridges considered here, (2) the proposed values of *IM* for rating of existing bridges, and (3) the values of *IM* proposed in a previous study (Deng and Cai 2010). From Table 4, it is observed that the *IM*s proposed in the present study are close to those proposed in Deng and Cai (2010), except for the case of very poor RSC.

In the previous work by Deng and Cai (2010), expressions for calculating the *IM*s were suggested based on a regression analysis of proposed *IM*s for different RSCs and the consideration of

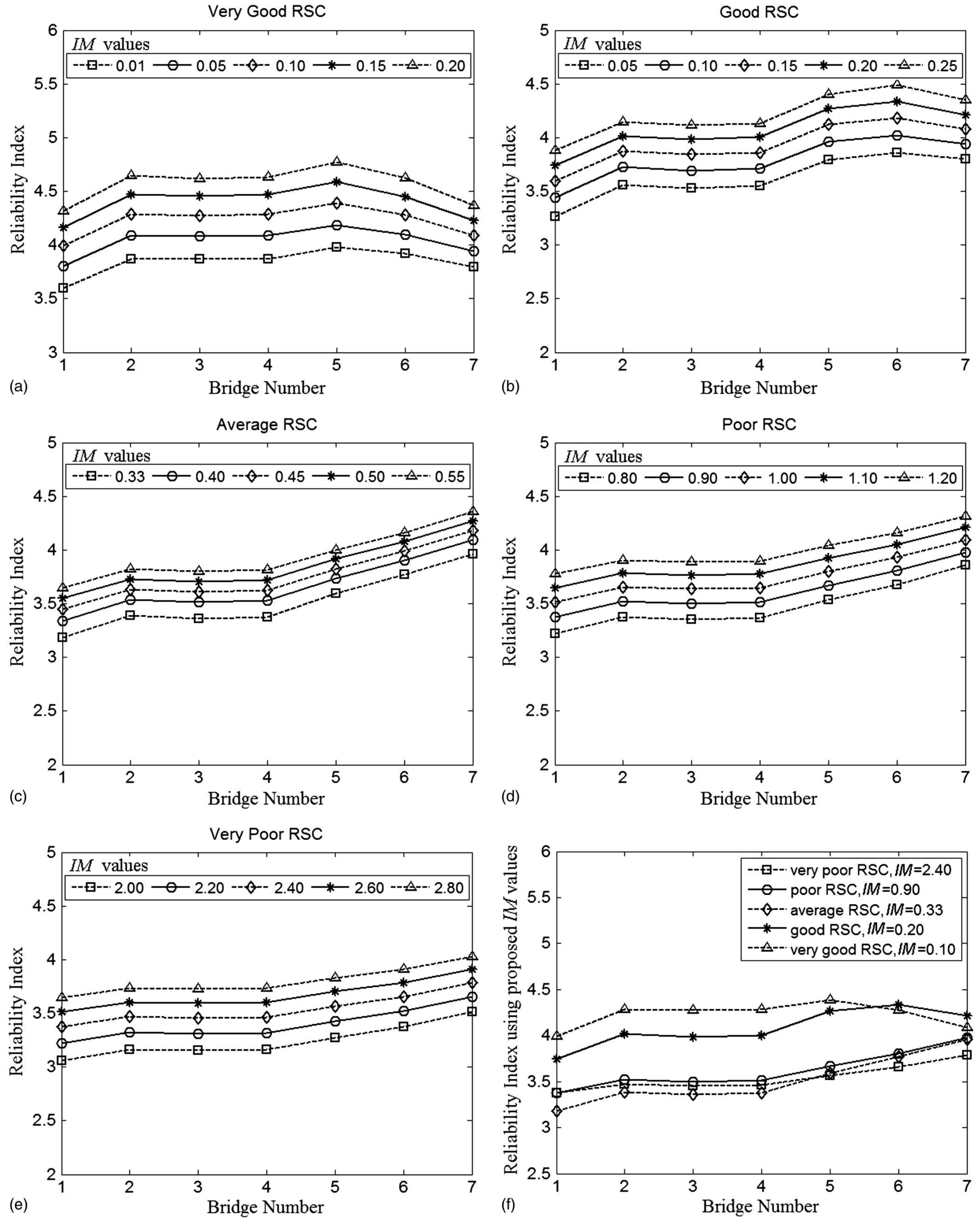

Fig. 5. Recalculated reliability indexes corresponding to the moment strength limit state for all seven bridges and all five road surface conditions

engineering practice. In the proposed expressions, a road surface index (RSI) was used to represent the effect of different RSCs. The proposed *IM*s were determined so that their values would fall in a confidence interval between 95 and 99%, i.e., imposing that the probability that the actual maximum *IM* on a particular bridge is smaller than the proposed *IM* is between 95 and 99% for any specified RSC. The rationale for this choice was ensuring consistency with a safety coefficient approach, in which the nominal values of

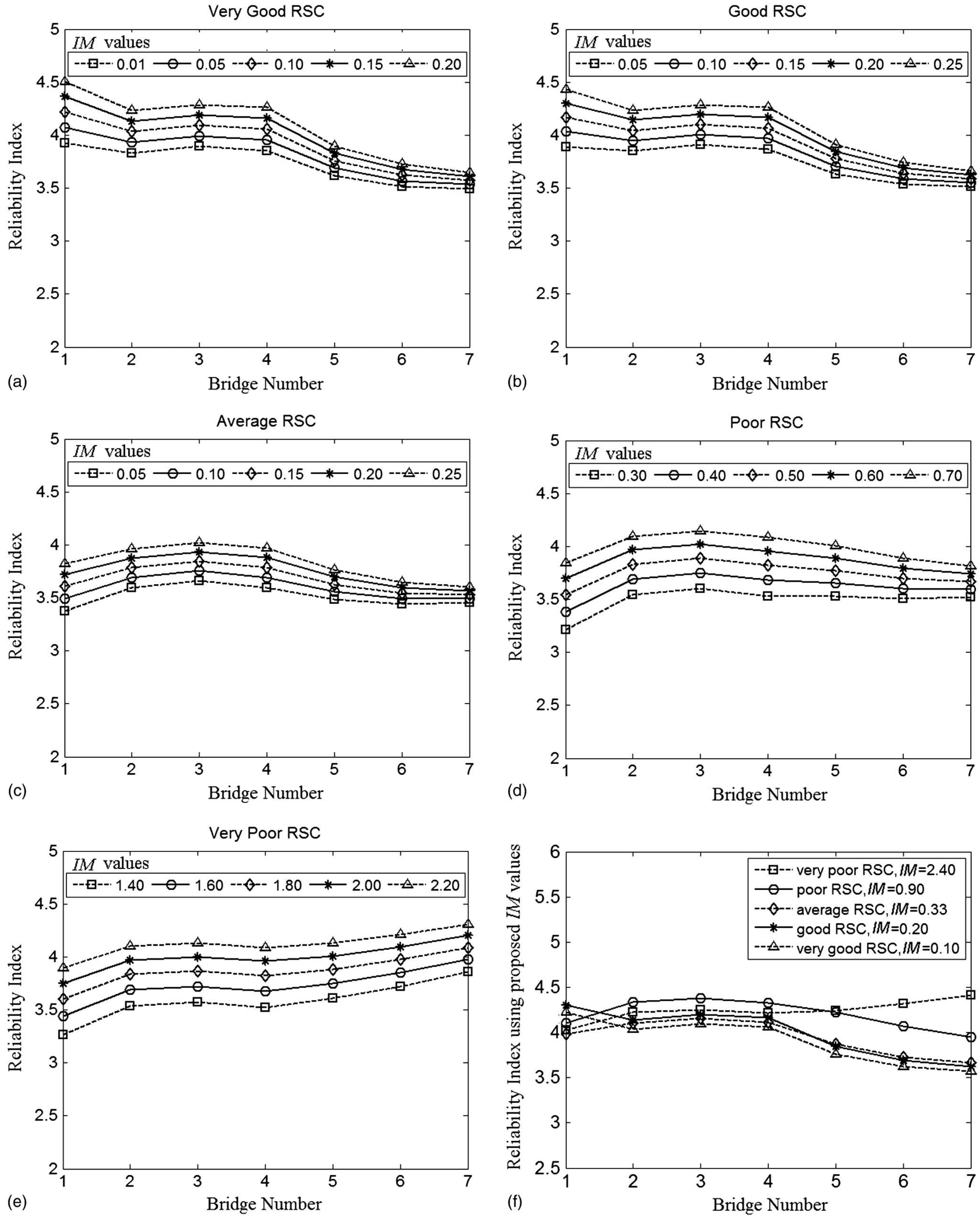

Fig. 6. Recalculated reliability indexes corresponding to the shear strength limit state for all seven bridges and all five road surface conditions

the design loads are usually chosen to lie between the 95th and 99th percentiles (Tsypin 1995; Lu and Lee 1996).

Bridge span length/natural frequency, road surface condition, and vehicle speed are three of the most important factors that affect the dynamic effect of vehicle loads (Huang et al. 1993; Yang et al. 1995; Shi et al. 2008). The statistical information derived in Deng and Cai (2010) already includes the effects on the dynamic impact factor caused by the span lengths and vehicle speed. Thus, the present study includes implicitly the effects of different span lengths and vehicle speeds. Three important motivations suggest

Table 4. Comparison of the Proposed Dynamic Load Allowance *IM* in the Present Study and Those by Deng and Cai (2010)

Road surface condition	Deng and Cai (2010) Proposed *IM*	Present study Theoretical minimum *IM*	Present study Proposed *IM*
Very poor	1.99	2.50	2.40
Poor	0.99	1.00	0.90
Average	0.50	0.50	0.33
Good	0.33	0.15	0.20
Very good	0.23	0.05	0.10

expressing the dynamic load allowance as a function of the road surface condition alone. First of all, the influence of the bridge span length and vehicle speed on the vehicle dynamic effect is not as obvious and straightforward as the influence of bridge surface condition, as demonstrated in a previous study (Deng and Cai 2010). A second important reason is that a constant dynamic load allowance must be used in practice, regardless of the varying vehicle speed. Finally, the format for the dynamic load allowance that can be used in practice for bridge rating not only must account rigorously for the accurate relationship between the primary factors and the dynamic impact factor, but also must be consistent with the current design code and as easy to use as possible for practicing engineers.

The proposed *IM*s have also been chosen with the aim of balancing three different needs: (1) obtaining reliability indexes consistent with the target reliability index of 3.5 for all bridges under any RSC, (2) ensuring as much consistency as possible with the AASHTO-LRFD code (2004), and (3) minimizing the modifications to the *IM* values suggested by the AASHTO-LRFR manual (2003). It is noteworthy that when the RSC is below average, the proposed *IM* values are greater than the value of 0.33 suggested by the current LRFD code. In particular, the *IM* value proposed for very poor RSC is significantly larger than 0.33 (i.e., more than seven times larger). Because a significant portion (i.e., more than 17%) of all bridge decks in the United States belong to the fair or worse RSC categories, it is concluded that the use of different *IM*s that account for the different RSCs is necessary for accurate performance evaluation and/or rating of existing bridges. When the RSCs are good or very good, the proposed *IM*s are the same as that suggested in the commentary of AASHTO-LRFR (2003), i.e., 0.2 and 0.1, respectively, despite the fact that the theoretical minimum values needed to reach the target reliability index of 3.5 are slightly smaller than the proposed values. The present study has generally confirmed the *IM* values suggested by LRFR for good and very good RSC cases.

In the case of average RSCs, which correspond to the design RSCs considered by the LRFD code, the required minimum *IM* value (i.e., 0.50) is significantly larger than the value $IM = 0.33$ prescribed by the LRFD code. The results reported in this paper seem to indicate that the *IM* value prescribed by the AASHTO-LRFD code may be insufficient to ensure the target reliability index of 3.5 in the case of prestressed concrete girder bridges. Further study is required to verify if a modification of the *IM* value prescribed by the AASHTO-LRFD code is needed because the data presented here are too limited to draw a general conclusion on this issue.

The proposed *IM*s can be used for rating of existing bridges if the RSC of the bridge to be rated is known. As a demonstration, the inventory load rating factors based on the newly proposed *IM*s are calculated for the seven bridges with different RSCs, and are shown in Fig. 7. Fig. 7 shows that for some RSCs, if the original bridge is designed perfectly (neither overdesigned nor underdesigned), the actual rating is lower than 1.0 due to the deterioration of road surface conditions. The rating (and correspondingly the reliability indexes) for different bridge types is more uniform for moment than for shear. When field dynamic tests are conducted, the actual measured *IM*s can be used in lieu of the proposed *IM*s.

Conclusions

This study derives, according to the methodology adopted in the AASHTO-LRFD code, the reliability indexes for a set of seven prestressed concrete girder bridges by modeling explicitly the variability of the dynamic load allowance (*IM*). By modeling the *IM* as a random variable with statistical properties obtained from a previous study, it is found that the newly calculated reliability indexes for the seven concrete girder bridges under study are well below the target level of 3.5 when the road surface condition is below average. Although road surface condition has been proven to be a significant factor for bridge dynamic loads by numerous studies in the literature, the current AASHTO codes employ, for all road surface conditions, a maximum *IM* of 0.33, which corresponds to an average road surface condition. The study presented in this paper clearly

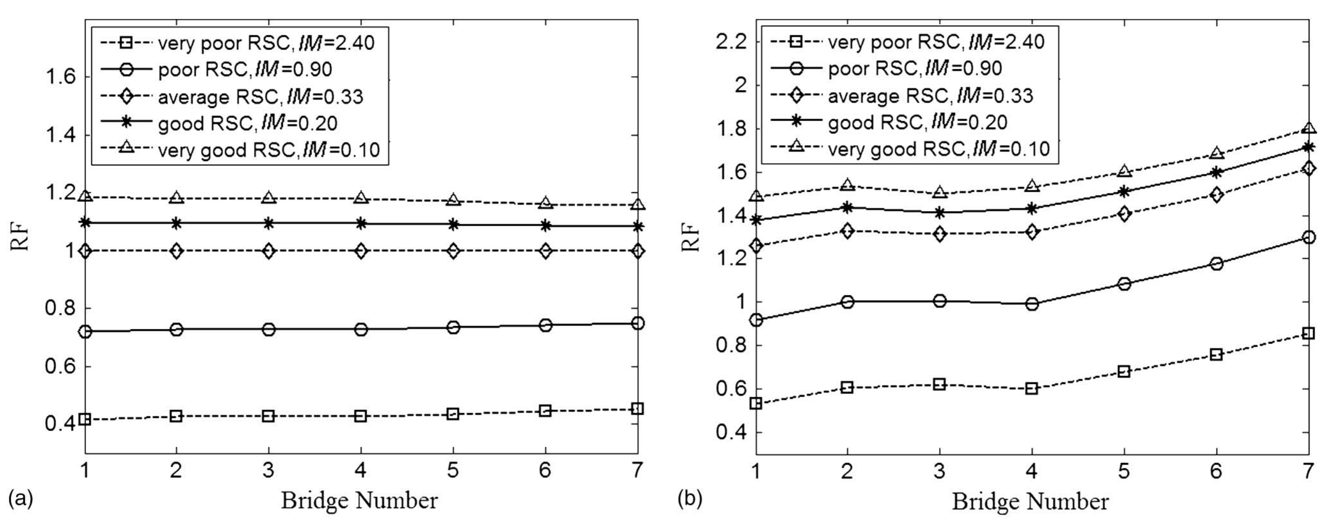

Fig. 7. Calculated load rating factors using the proposed *IM*: (a) moment; (b) shear strength limit states

indicates that a proper value of *IM*, often significantly different than the 0.33 value recommended in the AASHTO-LRFR manual, is needed for evaluation and rating of existing bridges. This appropriate value of *IM* strongly depends on the actual road surface condition of existing bridges.

In this study, appropriate values of *IM* are suggested for different road surface conditions. When the RSCs are average, good or very good, the proposed *IM*s are the same as that suggested in the commentary of AASHTO-LRFR (2003), i.e., 0.33, 0.2, and 0.1, respectively. When the RSC is poor or very poor, the proposed *IM* values (i.e., 0.9 and 2.4, respectively) are larger than the value of 0.33 suggested by the current LRFD code.

A dynamic load allowance that can be used for practical bridge rating must (1) rigorously account for the relations between the dynamic impact factor and the quantities that most affect it, (2) be consistent with the current design code, and (3) be as simple as possible for use by practicing engineers. In this paper, a dynamic load allowance is proposed by taking into account all of the above practical and theoretical considerations. The procedures and results from this study can be used to suggest an appropriate procedure for evaluation and rating of existing bridges that is consistent with the LRFD methodology adopted in the AASHTO codes.

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