Engineering Structures 32 (2010) 21-31

Contents lists available at ScienceDirect

**Engineering Structures** 

journal homepage: www.elsevier.com/locate/engstruct

# Development of dynamic impact factor for performance evaluation of existing multi-girder concrete bridges

# Lu Deng, C.S. Cai\*

Department of Civil and Environmental Engineering, Louisiana State University, Baton Rouge, LA 70803, United States

# ARTICLE INFO

Article history: Received 31 October 2008 Received in revised form 17 August 2009 Accepted 17 August 2009 Available online 4 September 2009

Keywords: Impact factor Vehicle-bridge coupled model Bridge span length Vehicle speed Road surface condition Distribution Confidence level

# ABSTRACT

The dynamic effect of moving vehicles on bridges is generally treated as a dynamic load allowance (or dynamic impact factor) in many design codes. Due to the road surface deterioration of existing bridges, studies have shown that the calculated impact factors from field measurements could be higher than the values specified in design codes that mainly target at new bridge designs. This paper develops a 3D vehicle–bridge coupled model to simulate the interaction between a bridge and vehicles and investigates the impact factor for multi-girder concrete bridges. The effects of bridge span length, vehicle speed, and road surface condition on the impact factor are examined. Chi-square tests are then performed on the impact factors and it is found that the impact factors obtained under the same road surface condition follow the Extreme-I type distribution. Finally, simple expressions for calculating the impact factors are suggested applicable to both new and existing bridges. Corresponding confidence levels with the proposed impact factors for the five studied bridges indicate that the proposed expressions can be used with considerable confidence. The proposed expressions for impact factor can be used as a modification of the AASHTO specifications when dealing with short bridges and old bridges with poor road surface condition for which the AASHTO specifications may underestimate the impact factor.

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# 1. Introduction

The dynamic effect of moving vehicles on bridges is generally treated as a dynamic load allowance (or dynamic impact factor) in many design codes. For example, a value of 0.33 is suggested for the dynamic impact factor by the AASHTO LRFD specifications [1]. In AASHTO standard specifications [2], it is expressed as a function of the bridge length. In other codes, like Canada's Ontario Bridge Design Code [3] and Australia's NAASRA Code [4], the impact factor is defined as a function of the first flexural frequency of the bridge. A review of various impact factors for highway bridges implemented by various countries around the world can be found in GangaRao [5].

In the past two decades, significant efforts have been made to investigate the dynamic effect caused by dynamic vehicle loads using different analytical bridge-vehicle models [6–13]. Field testing has also been carried out to verify the impact factors specified in the design codes [11,14–16]. However, it has been demonstrated through both analytical studies and field testing that the design codes may underestimate the impact factor under poor road surface conditions [15–17].

\* Corresponding author. E-mail address: CSCAI@LSU.EDU (C.S. Cai).

One of the reasons for the underestimation of the impact factor could be that design codes, like the AASHTO specifications, are mainly providing guidelines for designing new bridges with good road surface condition. Therefore, the code-specified impact factors may not be a problem for bridges with good surface condition. However, for a large majority of old bridges whose road surface conditions have deteriorated due to factors like aging, corrosion, increased gross vehicle weight and so on, caution should be taken when using the code-specified impact factor. As a matter of fact, the average age of bridges in the United States has reached 43 years according to a recent AASHTO report [18]. Therefore, for safety purposes more appropriate impact factors should be provided for performance evaluation of these old bridges. Chang and Lee [8] proposed a function of impact factor for simple-span girder bridges with respect to bridge span length, vehicle traveling speed, and maximum magnitude of surface roughness; however, their study was based on simplified bridge and vehicle models, and more theoretical support was also needed for the proposed impact factor functions.

In this paper a 3D vehicle-bridge coupled model is used to analyze the impact factor for multi-girder bridges. The relationship between three parameters, which include the bridge span length, road surface condition, and vehicle speed, and the impact factor is examined by numerical simulations. Chi-square tests are then performed to examine the distribution of the impact factors under the same road surface condition for all the five road surface conditions considered. Based on the results from this study, reasonable







Fig. 1. Typical cross section of bridges.

#### Table 1

Detailed properties of the five bridges.

-		*				
Bridge number	Span length (m)	Fundamental natural frequency (Hz)		Girder		Number of Intermediate Diaphragm
			AASHTO type	Cross-sectional area (m <sup>2</sup> )	Inertia moment of cross section $(10^{-2} \text{ m}^4)$	
1	9.14	15.51	II	0.238	2.122	0
2	16.76	6.58	II	0.238	2.122	1
3	24.38	4.60	III	0.361	5.219	1
4	32.00	3.20	IV	0.509	10.853	2
5	39.62	2.66	V	0.753	32.859	2



Fig. 2. A finite element model for Bridge 2.

expressions for calculating the impact factor are suggested applicable to both new and existing bridges. Corresponding confidence levels with the proposed impact factors for the five studied bridges are provided along with the determined distributions of the impact factors. The proposed expressions can be used as a modification of the AASHTO specifications when dealing with short bridges and old bridges with poor road surface condition for which the AASHTO specifications may underestimate the impact factor.

# 2. Analytical bridges

The bridges used in this study are good representatives of the majority of concrete slab-on-girder bridges in the United States. Five typical prestressed concrete girder bridges with a span length ranging from 9.14 m (30 ft) to 39.62 m (130 ft) were designed according to the AASHTO standard specifications [2]. All five bridges, consisting of five identical girders with a girder spacing of 2.13 m (7 ft), are simply supported 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 the bridges is shown in Fig. 1. Besides the end diaphragms, which are used for all five bridges, intermediate

#### Table 2

Major parameters of the vehicle under study (HS20).

Mass of truck body 1	2612 (kg)
Pitching moment of inertia of truck body 1	2022 (kg m <sup>2</sup> )
Rolling moment of inertia of tuck body 1	8544 (kg m <sup>2)</sup>
Mass of truck body 2	26 113 (kg)
Pitching moment of inertia of truck body 2	33153 (kg m <sup>2</sup> )
Rolling moment of inertia of tuck body 2	$181216(\text{kg}\text{m}^2)$
Mass of the first axle suspension	490 (kg)
Upper spring stiffness of the first axle	242 604 (N/m)
Upper damper coefficient of the first axle	2190 (N s/m)
Lower spring stiffness of the first axle	875 082 (N/m)
Lower damper coefficient of the first axle	2000 (N s/m)
Mass of the second axle suspension	808 (kg)
Upper spring stiffness of the second axle	1 903 172 (N/m)
Upper damper coefficient of the second axle	7882 (N s/m)
Lower spring stiffness of the second axle	3 503 307 (N/m)
Lower damper coefficient of the second axle	2000 (N s/m)
Mass of the third axle suspension	653 (kg)
Upper spring stiffness of the third axle	1 969 034 (N/m)
Upper damper coefficient of the third axle	7182 (N s/m)
Lower spring stiffness of the third axle	3 507 429 (N/m)
Lower damper coefficient of the third axle	2000 (N s/m)
L1	1.698 (m)
L2	2.569 (m)
L3	1.984 (m)
L4	2.283 (m)
L5	2.215 (m)
L6	2.338 (m)
В	1.1 (m)

diaphragms are also used to connect the five girders depending on their span lengths as shown in Table 1.

In the present study, the concrete bridges were modeled with the ANSYS<sup>©</sup> program using solid elements (with three translational DOFs at each node). Fig. 2 shows the finite element model of Bridge 2. A summary of the detailed properties and the fundamental frequencies of the five bridges obtained from the finite element analysis are shown in Table 1.

# 3. Analytical vehicle model

An AASHTO HS20-44 truck, which is a major design vehicle in the AASHTO bridge design specifications, was used for the vehicle loading for the five bridges. The analytical model for this truck is illustrated in Fig. 3, and the properties of the truck including the



Fig. 3. Analytical model of the HS20-44 truck.



Fig. 4. Deterioration of bridge road surface at (a) bridge deck [20] and (b) bridge joint [21].

geometry, mass distribution, damping, and stiffness of the tires and suspension systems are shown in Table 2 [16,19].

# 4. Vehicle-bridge coupled system

#### 4.1. Equation of motion of the vehicle

The equation of motion for a vehicle can be expressed as follows:

$$[M_v] \{ \ddot{d}_v \} + [C_v] \{ \dot{d}_v \} + [K_v] \{ d_v \} = \{ F_G \} + \{ F_v \}$$
(1)

where  $[M_v]$ ,  $[C_v]$ , and  $[K_v]$  = the mass, damping, and stiffness matrices of the vehicle, respectively;  $\{d_v\}$  = the displacement vector of the vehicle;  $\{F_G\}$  = gravity force vector of the vehicle; and  $\{F_v\}$  = vector of the wheel-road contact forces acting on the vehicle.

### 4.2. Equation of motion of the bridge

The equation of motion for a bridge under vehicle loading can be written as follows:

$$[M_b] \left\{ \ddot{d}_b \right\} + [C_b] \left\{ \dot{d}_b \right\} + [K_b] \left\{ d_b \right\} = \{F_b\}$$

$$(2)$$

where  $[M_b]$ ,  $[C_b]$ , and  $[K_b]$  = the mass, damping, and stiffness matrices of the bridge, respectively;  $\{d_b\}$  = the displacement vector of the bridge; and  $\{F_b\}$  = vector of the wheel-road contact forces acting on the bridge.

#### 4.3. Road surface condition

Road surface condition is a very important factor that affects the dynamic responses of both the bridge and vehicles. Deterioration of bridge road surfaces can occur at both the bridge deck and joints due to factors like aging, varying environmental conditions, corrosion, increased gross vehicle weight, etc. Fig. 4 shows two examples of degraded bridge road surface.

A road surface profile is usually assumed to be a zero-mean stationary Gaussian random process and can be generated through an inverse Fourier transformation based on a power spectral density (PSD) function [22] such as:

$$r(X) = \sum_{k=1}^{N} \sqrt{2\varphi(n_k)\Delta n} \cos(2\pi n_k X + \theta_k)$$
(3)

where  $\theta_k$  is the random phase angle uniformly distributed from 0 to  $2\pi$ ;  $\varphi()$  is the PSD function (m<sup>3</sup>/cycle) for the road surface elevation; and  $n_k$  is the wave number (cycle/m). In the present study, the following PSD function [23] was used:

$$\varphi(n) = \varphi(n_0) \left(\frac{n}{n_0}\right)^{-2} \quad (n_1 < n < n_2) \tag{4}$$

where *n* is the spatial frequency (cycle/m);  $n_0$  is the discontinuity frequency of  $1/2\pi$  (cycle/m);  $\varphi(n_0)$  is the roughness coefficient (m<sup>3</sup>/cycle) whose value is chosen depending on the road condition; and  $n_1$  and  $n_2$  are the lower and upper cut-off frequencies, respectively.



Fig. 5. Vehicle loading position.

The International Organization for Standardization (ISO) [24] has proposed a road roughness classification index from A (very good) to H (very poor) according to different values of  $\varphi(n_0)$ . In this paper the classification of road roughness based on the ISO [24] was used.

#### 4.4. Assembling the vehicle-bridge coupled system

Using the displacement relationship and the interaction force relationship at the contact points, the vehicle-bridge coupled system can be established by combining the equations of motion of both the bridge and vehicle, as shown below:

$$\begin{bmatrix} M_b \\ M_v \end{bmatrix} \begin{bmatrix} \ddot{d}_b \\ \ddot{d}_v \end{bmatrix} + \begin{bmatrix} C_b + C_{b-b} & C_{b-v} \\ C_{v-b} & C_v \end{bmatrix} \begin{bmatrix} \dot{d}_b \\ \dot{d}_v \end{bmatrix} + \begin{bmatrix} K_b + K_{b-b} & K_{b-v} \\ K_{v-b} & K_v \end{bmatrix} \begin{bmatrix} d_b \\ d_v \end{bmatrix} = \begin{cases} F_{b-r} \\ F_{b-r} + F_G \end{cases}$$
(5)

where  $C_{b-b}$ ,  $C_{b-v}$ ,  $C_{v-b}$ ,  $K_{b-b}$ ,  $K_{b-v}$ ,  $K_{v-b}$ ,  $F_{b-r}$ , and  $F_{b-r}$  are due to the wheel-road contact forces. When the vehicles move across the bridge, the positions of the contact points as well as the values of the contact forces change, indicating that all these terms listed above are time-dependent terms and will change as the vehicles move across the bridge.

To simplify the bridge model and therefore reduce the computation effort, the modal superposition technique can be used; the displacement vector of the bridge  $\{d_b\}$  in Eq. (5) can be expressed as:

$$\{d_b\} = \begin{bmatrix} \{\Phi_1\} & \{\Phi_2\} \dots \{\Phi_m\} \end{bmatrix} \begin{bmatrix} \xi_1 & \xi_2 \dots \xi_m \end{bmatrix}^{\mathsf{T}} = \begin{bmatrix} \Phi_b \end{bmatrix} \{\xi_b\} \quad (6)$$

where *m* is the total number of modes used for the bridge;  $\{\Phi_i\}$  and  $\xi_i$  are the *i*th mode shape of the bridge and the *i*th generalized modal coordinate, respectively. Each mode shape is normalized such that  $\{\Phi_i\}^T [M_b] \{\Phi_i\} = 1$  and  $\{\Phi_i\}^T [K_b] \{\Phi_i\} = \omega_i^2$ .

Assuming  $[C_b]$  in Eq. (2) to be equal to  $2\omega_i \eta_i [M_b]$ , where  $\eta_i$  is the percentage of the critical damping for the *i*th mode of the bridge, Eq. (5) can then be simplified into the following:

$$\begin{bmatrix} I \\ M_{v} \end{bmatrix} \begin{bmatrix} \dot{\xi}_{b} \\ \dot{d}_{v} \end{bmatrix} + \begin{bmatrix} 2\omega_{i}\eta_{i}I + \Phi_{b}^{T}C_{b-b}\Phi_{b} & \Phi_{b}^{T}C_{b-v} \\ C_{v-b}\Phi_{b} & C_{v} \end{bmatrix} \begin{bmatrix} \dot{\xi}_{b} \\ \dot{d}_{v} \end{bmatrix} + \begin{bmatrix} \omega_{i}^{2}I + \Phi_{b}^{T}K_{b-b}\Phi_{b} & \Phi_{b}^{T}K_{b-v} \\ K_{v-b}\Phi_{b} & K_{v} \end{bmatrix} \begin{bmatrix} \xi_{b} \\ d_{v} \end{bmatrix} = \begin{bmatrix} \Phi_{b}^{T}F_{b-r} \\ F_{v-r} + F_{G} \end{bmatrix}$$
(7)

The vehicle–bridge coupled system in Eq. (7) contains only the modal properties of the bridge and the physical parameters of the vehicles. As a result, the complexity of solving the vehicle–bridge coupling equations is greatly reduced. A Matlab program was developed to assemble the vehicle–bridge coupled system in Eq. (7) and solve it using the fourth-order Runge–Kutta method in the time domain. Readers are referred to Cai et al. [12] and Shi et al. [13] for more details.

#### 5. Numerical studies

In the literature, a number of parameters have been studied for their effects on the dynamic impact factor which includes the vehicle loading position, vehicle weight, vehicle traveling speed, number of loading lanes, girder spacing, road surface condition, road surface roughness correlation, etc. [8–10,23]. In the present study, for the purpose of developing simple code types of formulas only three main parameters commonly considered to have significant effect on the impact factor were investigated: namely the bridge span length, traveling speed of vehicle, and road surface condition.

The accuracy and reliability of the bridge-vehicle model is definitely crucial to the results on the dynamic impact factor. The reliability of the bridge numerical models and the bridge-vehicle model have been verified in other works by the authors [25,26], in which a series of field tests were conducted on an existing slabon-girder concrete bridge (a bridge very similar to the example bridges used in this study) in Louisiana, and the bridge responses, including deflections and strains at the mid-span of the girders, were measured and compared with the bridge responses obtained from the numerical simulations. The field measured results and the numerical results agree with each other very well.

The span lengths of the five bridges used in the present study are listed in Table 1. Seven vehicle speeds ranging from 30 km/h to 120 km/h with intervals of 15 km/h were considered, and five different road surface conditions according to the ISO [24] were studied: namely very good, good, average, poor, and very poor. Two loading cases were considered in the present study, and they were examined separately. Fig. 5 shows the vehicle positions for the two considered loading cases where the vehicles are traveling along the centerlines of the lanes. It should be noted that for Load Case II the road surface profiles along Lane 1 and Lane 2 were assumed to be exactly the same. In other words, the variation of road surface in the lateral direction was not considered.

To investigate the relationship between the three parameters and the impact factor for each specific case with a given bridge span length, vehicle speed, and road surface condition the vehicle-bridge interaction analysis was set to run 20 times with 20 sets of randomly generated road surface profiles under the given road surface condition, and the average value of the 20 impact factors was obtained. The coefficient of variation (COV) of the mean estimate is commonly used in statistics to verify the number of observations needed to estimate accurately the mean of variables. In this case such a COV was calculated to be less than 10% and the number of 20 was thus considered to be sufficient. The number of 20 simulations was also used by other researchers [10].

In this paper, the impact factor is defined as follows:

$$IM = \frac{R_d(x) - R_s(x)}{R_s(x)}$$
(8)

where  $R_d(x)$  and  $R_s(x)$  are the maximum dynamic and static responses of the bridge at location *x*, respectively. The deflection at



**Fig. 6.** Maximum static deflections at the mid-spans of the bridges under Load Case I.

the mid-span of the girder carrying the largest amount of load was selected as the bridge response for calculating the impact factor in the present study.

In the following parts of this section, the numerical studies will be presented as follows: two load cases will be examined separately; the average impact factor for each specific case with a given bridge span length, vehicle speed, and road surface condition will be obtained; Chi-square tests will then be performed to determine the distribution of impact factors under each road surface condition. Again, the determination of the distribution of impact factors will provide necessary base for deriving the expressions proposed for calculating the impact factors in the next section.

# 5.1. Load Case I

The maximum static deflections at the mid-spans of all five girders of each bridge under Load Case I are shown in Fig. 6. It can be easily observed from the figure that the largest deflection occurs at the mid-span of Girder 4 for both Bridges 1 and 2 while at Girder 5 for the other bridges. Therefore, the deflections from Girder 4 of Bridges 1 and 2 and Girder 5 of the other three bridges were used for calculating the impact factor for Load Case I. It should be noted that the reason why the deflection of Bridge 5 is smaller than that of Bridge 4 at every girder is that the girders of Bridge 5 have much larger moment of inertia than those of Bridge 4.

The average impact factors obtained from numerical simulations under Load Case I for each road surface condition (RSC) are plotted against the vehicle speed in Fig. 7 where plots for bridges with different span lengths are separated.

With the average impact factor varying from greater than 1.0 when the road surface condition is very poor to less than 0.15 when the road surface roughness is very good, it is evident from Fig. 7 that for all five bridges the road surface condition has a significant impact on the impact factor. However, an increase of vehicle speed does not necessarily guarantee an increase of the impact factor, as reported by many other researchers [10,27]. The effect of bridge span length on the impact factor is also unclear though most of the time Bridge 1 (the shortest bridge) seems to produce the largest impact factors among all five bridges.

Since Fig. 7 clearly shows that the AASHTO specifications have underestimated the impact factor when the bridge road surface condition is poor, an in-depth investigation on the distribution of the impact factors within each road surface condition would provide helpful information regarding the probability that the AASHTO specification may underestimate the impact factor. For this purpose, the impact factors generated previously were collected for each road surface condition, resulting in a total of 700 impact factors for each road surface condition (7 speeds  $\times$  5 bridges  $\times$  20 replicates). A Chi-square test was then performed on these 700 impact factors to determine their distributions. A known

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Chi-square test results on	the distribution of in	mpact factors for Load Case I
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Distribution type	Road surface condition				
	Very poor	Poor	Average	Good	Very good
Normal	37.60	27.74	46.06	21.04	27.37
Log-Normal	32.19	21.91	42.28	72.71	94.71
Extreme-I	5.02	6.27	9.02	23.39	17.35

distribution type is necessary for estimating the confidence level of the developed formulas later.

A Chi-square Test is a statistic test that can be used to test whether a set of data follows a certain type of distribution by comparing the Chi-square test value against a threshold value which can be determined by the number of intervals used in the histogram and the preset significance level for the test. The Chisquare test value can be calculated as follows:

$$\chi^{2} = \sum_{i=1}^{n} \frac{(O_{i} - E_{i})^{2}}{E_{i}}$$
(9)

where n = total number of intervals of histogram;  $O_i =$  exact number of data in the *i*th interval; and  $E_i =$  theoretical number of data in the *i*th interval of the assumed distribution type.

In the present study, the number of intervals of the histogram was set to be 10, and a significance level of 0.99 was used. As a result, the threshold value for the Chi-square test was set to be 18.48, which can be easily obtained from the CDF table of the Chi-Square Distribution from any statistics textbook. The collected impact factors for each road surface condition were tested against the Normal, Log-Normal, and Extreme-I type distributions, all of which are frequently used in the engineering field. The test results are shown in Table 3.

From Table 3 we can clearly see that the Chi-square test values for both Normal and Log-Normal distributions are all above the threshold value of 18.48 for each road surface condition indicating the collected impact factors do not fit these two distributions well. However, the Chi-square test values for the Extreme-I type distribution are all below 18.48 for all road surface conditions except the good road surface condition. This clearly indicates that the Extreme-I type distribution is the best distribution type that fits the data among the three distribution types.

To gain a better view of how the impact factors are distributed and how they are fitted to the Extreme-I type distribution, histograms showing the comparison between the theoretical Extreme-I type distribution and the real distribution of the impact factors for different road surface conditions are plotted in Fig. 8. From the figures it is clear that the impact factor data match the theoretical Extreme-I distribution very well, confirming the observations from Table 3.

# 5.2. Load Case II

The maximum static deflections at the mid-spans of all five girders of each bridge under Load Case II are shown in Fig. 9. It can be seen from the figure that the maximum static deflections occur at Girder 3 for all five bridges. Therefore, the deflections from Girder 3 were used for calculating the impact factors for all five bridges under Load Case II. Similar to Load Case I, the average impact factors obtained from numerical simulations under Load Case II are also plotted against the vehicle speed in Fig. 10.

Fig. 10 shows very similar results to those observed from Fig. 7: the average impact factors increase as the road surface condition becomes worse if the other two parameters remain unchanged; an increase in vehicle speed does not necessarily guarantee an increase of impact factor; the bridge with the shortest span length still produces the largest average impact factors. Chi-Square tests



Fig. 7. Variation of impact factors with change in vehicle speed and road surface condition for different bridges under Load Case I.

Table 4					
Chi-square test results on the distribution of impact factors for Load Case II.					
Distribution type	Road surface condition				

Distribution type	Road Sullace colluition					
	Very poor	Poor	Average	Good	Very good	
Normal Log-Normal Extreme-I	36.57 15.75 12.50	80.99 15.47 13.78	61.31 41.60 16.44	40.75 54.96 2.02	26.27 88.09 29.90	

were also performed on the impact factors obtained for Load Case II and the results are shown in Table 4.

From Table 4 it is clear that the Chi-square test values for both Normal and Log-Normal distributions are still above the preset threshold value of 18.48 most of the time. However, the Chisquare test values for the Extreme-I type distribution are all below 18.48 for all five cases except the case of very good road surface condition. These observations, which are similar to those observed from Table 3, confirm that the Extreme-I type distribution is the best distribution type that fits the impact factor data among the three distribution types, and it could be used as the true distribution of the impact factors with certain confidence.

Histograms are again used to show the comparison between the true distribution of the impact factors obtained under Load Case II and the theoretical Extreme-I type distribution in Fig. 11. Again, good matches can be observed from the histograms.

# 6. Suggested impact factors

The AASHTO LRFD specifications [1] use a dynamic impact factor of 0.33 for the design truck and tandem while a function of span length, as shown in Eq. (10) below, have also been used for many years in the AASHTO standard specifications [2].

$$IM = \frac{15.24}{L + 38.10} \tag{10}$$

where IM = dynamic impact factor; and L = bridge span length in meters.



Fig. 8. Histogram comparison between the real distribution of impact factors and theoretical Extreme-I type distribution for Load Case I.



Fig. 9. Maximum static deflection at the mid-span of the bridges under Load Case II.

To examine the individual effect of the three parameters on the impact factor more clearly and also compare the two different load cases, the averaged impact factors from both load cases are plotted against the three parameters separately in Fig. 12.

Fig. 12 confirms the conclusions observed in Figs. 7–10 regarding the effects of span length, vehicle speed, and road roughness. Moreover, from Fig. 12 it can be easily determined that while the road surface condition is of the level "Good", "Very Good", or "Average" condition the average impact factors are below the AASHTO-specified values. However, this conclusion does not hold when the road surface condition becomes worse than "Average". In fact, the impact factor could be much larger than the code-specified values if the road surface roughness is "very poor", a case commonly reported by many researchers [15–17]. This is easily understandable since the AASHTO specifications are mainly for guiding the design of new bridges with good road surface roughness. However, when it comes to the performance evaluation and maintenance of old bridges, the AASHTO specifications for the impact factor do not necessarily provide useful information for bridge engineers.

Since it was demonstrated in the previous section and previous research that the impact factor is highly dependent on the road surface condition, it would be very natural to propose the impact factor as a function of road surface condition. In the present study, the following expressions for calculating the dynamic impact factors are suggested based on a regression analysis of the numerical results and a consideration of present practice in AASHTO code specifications:

$$IM = RSI \times \begin{cases} 0.33 + 0.01 \times (16.76 - L) & L < 16.76 \text{ m} \\ 0.33 & L \ge 16.76 \text{ m} \end{cases}$$
(11)

where RSI = the road surface index, which takes the value of 0.7, 1, 1.5, 3, or 6 corresponding to very good, good, average, poor, or very poor road surface condition; and L = bridge span length.



Fig. 10. Variation of impact factors with change in vehicle speed and road surface condition for different bridges under Load Case II.

Compared with the single impact factor value of 0.33 provided by the AASHTO LRFD specifications [1], the proposed expressions in Eq. (11) are more reasonable in the sense that short bridges and different road surface conditions are considered with extra care based on the results from the numerical simulations. When the span length is larger than 16.76 m and the surface condition is good, the equation predicts the same impact factor as the AASHTO specifications [1]. This treatment can be justified with the observations from Fig. 12(a) where shows a significant decrease of the average impact factor as the bridge span length increases from 9.14 m to 16.76 m. However, as the bridge span length further increases. the average impact factor does not change significantly. The addition of a road surface index into the impact factor expression makes it not only suitable for new bridges with good surface conditions but also particularly useful for old bridges with different road surface conditions which could vary from very good to very poor.

The reason that the vehicle speed is not considered in this expression is that ideally vehicles can drive at speeds within a wide

range; second, as can be seen from Fig. 7, the average dynamic impact factors jump up and down as the vehicle speed increases from 30 km/h to 120 km/h; as a result, it would be difficult to describe the dynamic impact factor with respect to the vehicle speed. Therefore, as it is done in the codes, vehicle velocity is not treated as a variable in the proposed expressions for impact factor.

Based on the proposed expressions for the dynamic impact factor, the impact factor values for the five studied bridges are calculated in Table 5. Using the Chi-square tests it can be easily demonstrated that under the same road surface condition the impact factors for Bridge 1 and for Bridges 2–5 as a whole still follow the Extreme-I type distribution, respectively. Once the distribution is known, the probabilities that the generated impact factors would be less than the proposed impact factors can also be calculated as shown in Table 5. It should be noted that the probability values in Table 5 are the averaged one of the two loading cases.

As can be seen from Table 5, the confidence levels of all proposed impact factors for the five bridges are above 94%, with



Fig. 11. Histogram comparison between the real distribution of impact factors and theoretical Extreme-I type distribution for Load Case II.

# Table 5

Confidence levels of the proposed impact factors for the five bridges.

Bridge no.	Road surface condition				
	Very poor	Poor	Average	Good	Very good
1 2–5	2.44 <sup>a</sup>   95.4% <sup>b</sup> 1.98   99.0%	1.22   97.3% 0.99   99.5%	0.61   94.1% 0.50   98.5%	0.41   99.0% 0.33   99.5%	0.28   99.9% 0.23   99.4%

Proposed impact factor.

<sup>b</sup> Corresponding confidence level.

half of them even above 99%, indicating that these impact factor values can be used with considerable confidence in practice. The confidence levels with the proposed impact factors are also in good agreement with the criterion for determining design loads which is usually set to be between the 95 percentile to the 99 percentile points [28,29].

To check the credibility of the proposed dynamic impact factors, two other bridges (named Bridges 6 and 7) with different girder spacing and bridge width from the previous five bridges were created. These two girder bridges both have the same span length as Bridge 2 (16.76 m) with their configurations slightly modified from Bridge 2. Bridge 6 was modified from Bridge 2 by increasing the girder spacing from 2.13 m to 2.90 m, while Bridge 7 was modified by adding two more girders to Bridge 2. These modifications result in the two bridges having a width of 14.33 m (47 ft) each. Again, for each bridge the previously used five different road surface conditions and seven speeds were investigated; and for each case with the same road surface condition and vehicle speed, the program was set to run 20 times with randomly generated road surface profiles resulting in 140 impact factors under each road surface condition for each bridge. The numbers of obtained impact factors that exceed the proposed impact factors under different road surface conditions for the two bridges are shown in Table 6. These results show that the proposed impact factors are acceptable for these two bridges with small chances to be exceeded confirming that the proposed impact factors can be used with confidence for girder bridges with different girder spacing and bridge width.



Fig. 12. Variation of the average impact factor against each parameter individually.

#### Table 6

Number of impact factors that exceed the proposed impact factors for the two new bridges.

Bridge no.	Road surface condition				
	Very poor	Poor	Average	Good	Very good
6	1/140	0/140	0/140	0/140	0/140
7	0/140	0/140	1/140	0/140	0/140

# 7. Concluding remarks

A 3D vehicle-bridge coupled model was established, and numerical simulations were performed to study the impact factor for multi-girder concrete bridges. The effects of three parameters including the bridge span length, vehicle speed, and road surface condition were investigated. Simple and reasonable expressions for calculating the impact factor were suggested based on a study of the distribution of the impact factors. Corresponding confidence levels with the proposed impact factors for the five studied bridges were provided, indicating that the proposed expressions can be used with considerable confidence. The proposed impact factors were also checked using two other girder bridges, and results confirmed that the proposed impact factors are also appropriate for bridges with different girder spacing and bridge width.

The proposed expressions for the impact factor in this study can be used as a modification of the AASHTO specifications when dealing with short bridges and old bridges with poor road surface condition for which the AASHTO specification may underestimate the impact factor. Road surface condition has proven to be a significant factor for bridge dynamic loads by numerous studies in the literature; however, in the current AASHTO codes, the same impact factor is used for all road surface conditions. While this treatment is reasonable for new bridge design, evaluation of existing bridges with a possible deteriorated surface condition requires a separate treatment for different road surface conditions. For future researches, a reliability study may be conducted to check the corresponding reliability index of the proposed impact factor for each road surface condition.

It needs to be pointed out that many other factors, including truck suspension characteristics and traffic flow will affect the results and many previous studies have examined the effects of these factors. In the present study, one standard truck is used for the impact factor study. This is consistent with the AASHTO codes that are based on a single standard truck for the impact factor. Selection of suspension characteristics is also a difficult task since such information for many different trucks are not available. In this study, this selection is based on the best data available and the experience of other researchers. However, the same value is used for all road surface conditions and therefore, the relative effect of the road surface conditions on the impact factor is valid.

# Acknowledgements

The support of Economic Development Assistantship from Louisiana State University for the first author is greatly appreciated. Constructive inputs from Dr. Michelle Barbato at Louisiana State University are also acknowledged.

#### References

- [1] American Association of State Highway and Transportation Officials (AASHTO). LRFD bridge design specifications. Washington (DC); 2004.
- [2] American Association of State Highway and Transportation Officials (AASHTO). Standard specifications for highway bridges. Washington (DC); 2002.
- [3] Ontario Ministry of Transportation and Communications. Ontario highway bridge design code. Downsview (Ontario); 1983.
- [4] NAASRA bridge design specification. Sydney: National Association of Australian State Road Authorities; 1976.
- [5] GangaRao HVS. Impact factors for highway bridges. ASTM STP 1992;1164: 155–66.
- [6] Huang D, Wang TL, Shahawy M. Impact studies of multigirder concrete bridges. J Struct Eng ASCE 1992;119(8):2387–402.
- [7] Wang TL, Shahawy M, Huang DZ. Impact in highway prestressed concrete bridges. Comput. Struct. 1992;44(3):525–34.
- [8] Chang D, Lee H. Impact factors for simple-span highway girder bridges. J Struct Eng 1994;120(3):704–15.
- [9] Yang YB, Liao SS, Lin BH. Impact formulas for vehicles moving over simple and continuous beams. J Struct Eng ASCE 1995;121(11):1644–50.
- [10] Liu C, Huang D, Wang TL. Analytical dynamic impact study based on correlated road roughness. Comput Struct 2002;80:1639–50.
- [11] Park YS, Shin DK, Chung TJ. Influence of road surface roughness on dynamic impact factor of bridge by full-scale dynamic testing. Canad. J. Civil Eng. 2005; 32(5):825–9.
- [12] Cai CS, Shi XM, Araujo M, Chen SR. Effect of approach span condition on vehicle-induced dynamic response of slab-on-girder road bridges. Eng. Struct. 2007;29(12):3210–26.
- [13] Shi XM, Cai CS, Chen SR. Vehicle induced dynamic behavior of short span bridges considering effect of approach span condition. J Bridge Eng ASCE 2008; 13(1):83–92.
- [14] Green R. Dynamic response of bridge superstructures-Ontario observations. Supplementary Report SR 75, Transportation and Road Research Lab, Crowthorne, England, 40–48; 1997.
- [15] O'Connor C, Pritchard RW. Impact studies on small composite girder bridges. J Struct Eng ASCE 1985;111:641–53.
- [16] Shi XM. Structural performance of approach slab and its effect on vehicle induced bridge dynamic response. Ph.D. dissertation. Baton Rouge (LA): Louisiana State University; 2006.
- [17] Billing JR. Dynamic loading and testing of bridges in Ontario. Canad. J. Civil Eng. 1984;11:833–43.
- [18] American Association of State Highway and Transportation Officials (AASHTO). Bridge the Gap: Restoring and rebuilding the nation's bridges. Report. Washington (DC); 2008.

- [19] Wang TL, Liu CH. Influence of heavy trucks on highway bridges. Rep. No. FL/DOT/RMC/ 6672-379. Florida Department of Transportation. Tallahassee (FL); 2000.
- [20] Raina VK. Concrete bridge. New York (NY): McGraw-Hill, Inc.; 1996.
- [21] White D, Sritharan S, Suleiman M, Mekkawy M, Chetlur S. Identification of the best practices for design, construction, and repair of bridge approaches. Rep. No. CTRE Project 02-118. Ames (IA): Iowa Department of Transportation; 2005.
- [22] Dodds CJ, Robson JD. The description of road surface roughness. J Sound Vib 1973;31(2):175–83.
  [23] Huang DZ, Wang TL. Impact analysis of cable-stayed bridges. Comput Struct
- 1992;43(5):897–908.
- [24] International Organization for Standardization (ISO). Mechanical vibrationroad surface profiles-reporting of measured data. ISO 8068: 1995 (E), ISO. Geneva; 1995.
- [25] Deng L. System identification of bridge and vehicles based on their coupled vibration. Ph.D. dissertation, Baton Rouge (LA): Louisiana State University; 2009.
- [26] Deng L, Cai CS. Bridge model updating using response surface method and genetic algorithm. J Bridge Eng ASCE 2009 (in press).
- [27] Brady SP, O'Brien EJ, Znidaric A. Effect of vehicle velocity on the dynamic amplification of a vehicle crossing a simply supported bridge. J Bridge Eng ASCE 2006;11(2):241–9.
- [28] Tsypin VSh. Standardization of wave loads on river structures. Hydrotech. Construct. 1995;29(5):32–3.
- [29] Lu MW, Lee YL. Reliability based strength/fatigue design criterion. In: Proceedings of the annual reliability and maintainability symposium. Las Vegas (NV, USA). 1996. p. 263–9.