

## Research Article

# Development of Dynamic Impact Factor Expressions for Skewed Composite Concrete-Steel Slab-On-Girder Bridges

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In order to take into account the dynamic effects of moving vehicles, bridges are designed to carry static loads that are increased by dynamic impact (IFs) factors (or dynamic amplification factors) that are a function of either the span or the first flexural natural frequency of the bridge. However, this approach tends to produce very conservative designs as the IFs are calculated based on a relatively few general parameters, ignoring many significant bridge and truck dynamic characteristics. This paper presents a method for determining more realistic dynamic impact factors for skewed composite slab-on-girder bridges under AASHTO LRFD truck loading. An extensive parametric study of over 125 bridge prototypes examined key parameters, namely, the number of girders, number of lanes, skew angle, and span length. Based on the data generated by this analysis, appropriate expressions for dynamic impact factors for the longitudinal moment and deflection are proposed. In order to reduce the complexity of proposed expressions, the effects of road surface roughness on dynamic responses of bridge-vehicle interaction are considered in bridge modeling. The findings of this study are expected to help bridge engineers to design composite slab-on-girder bridges more reliably and economically and can also be used to reassess the safe live-load capacity of existing structures, potentially preventing the unnecessary posting or closing of busy highway bridges.

## 1. Introduction

Recent developments in industry have contributed to a major increase in the number and speed of vehicles, especially trucks, passing over bridges in many parts of the world. At the same time, the types of bridge superstructures are changing and becoming more diverse. Taken together, these trends have a significant impact on the dynamic responses of our bridges. Although the load-carrying capacity is considered the most significant of the design criteria during the bridge design process, the dynamic behavior is also assessed by applying a parameter known as an impact factor (IF) to represent the bridge's dynamic characteristics. The dynamic responses of a bridge under vehicle loading are a complex phenomenon because of the interaction between the bridge and the vehicle. This complexity makes it difficult to identify the significant parameters that govern the responses [1]. The factors affecting the dynamic increment

with respect to the static component have been recently investigated by several researchers [2, 3], and the surface roughness, vehicle characteristics, and the geometry and type of bridge were found to significantly affect the dynamic bridge-vehicle interactions. In spite of the substantial body of research devoted to investigating the dynamic interaction of bridges under moving loads [4–10], it has not been successful to express the dynamic behavior of bridges in a simple quantitative expression due to the nature of complexity. Earlier studies have investigated the vibration behavior of simply supported girder bridges with rough surface condition under truck loads by idealizing a bridge as a single-beam element and a truck as a single sprung-mass moving loads along the deck [11, 12]. Huang et al. [12] developed a theoretical method for bridge-vehicle interactions by idealizing the bridge as an isotropic or orthotropic plate. Nonlinear equations of motion are solved by discretizing the plate by finite elements as a function of

impact factors with respect to bridge span length and vehicle speed. Deng and Cai [13] developed a three-dimensional vehicle-bridge coupled model to simulate the bridge-vehicle interaction and determine the dynamic impact factors for composite multigirder bridges. The method relied on an iterative approach, in which the vehicle and bridge became totally uncoupled and were only linked by connect force history to predict the vehicle-induced responses of bridges [14].

Instead, current codes and specifications recommend the use of a dynamic impact factor that is expressed as a function of span length or natural frequency. For example, the AASHTO LRFD [15] specifications and the Korean Bridge Design specifications [16] both calculate this impact factor as a function of span length regardless of the type of bridge, while the Japanese road association's specifications for highway bridges [17] recommend different IF formulas for steel, reinforced concrete, and prestressed concrete bridges. An alternative approach used in highway bridge design codes in jurisdictions such as the Canadian province of Ontario [18] treats the impact factor as a function of the bridge's first fundamental frequency, where an appropriate value is developed based on analytical and experimental investigations of the relationship between a bridge's natural frequency and its dynamic response [4, 19]. Nevertheless, many researchers have found that these specifications all fail to compute reasonable value for IF to some extent.

Since the IFs computed from the various bridge analysis methods can differ significantly, further study is clearly needed if we are to obtain a deeper understanding of the relationship between the dynamic impact factors (IFs) for various bridge types and bridge behavior and to apply them more correctly in structural practice [20–25]. On the other hand, the distribution factor and dynamic impact factor equations provided in the current AASHTO LRFD specifications [15] included only a limited range of applicability; when these limitations are not satisfied, the specifications mandate a refined analysis. This restriction creates unacceptable inconsistencies when applied to the development and construction of new bridge types. To obtain satisfactory accuracy for the IFs and thus improve the ability of bridge engineers to model complex bridge structures and eliminate applicability limitations of the AASHTO LRFD specifications, a numerical study was therefore carried out on 125 skew composite concrete-steel slab-girder bridges. The effect of vehicle speed, loading position on traffic lane, number of lane loads, number of girders, and skew angle on the IFs were evaluated, and the accuracy of the current specifications for calculating IFs was investigated.

## 2. Finite Element Modeling

*2.1. Definition of Dynamic Load Allowance and Related Code Values.* The prototype bridge used in this study represents well the majority of concrete skew composite slab-on-girder bridges. Eighty-five typical bridges with a span length ranging from 30 m to 90 m are designed based on Canadian Highway Bridge Design Code [26] to overcome the limitations of current AASHTO LRFD. The number of boxes

TABLE 1: Impact factors specified by various codes.

Country	Impact factor (IF)	Code (specification)
US	$[15.24/(L + 38.10)] \leq 0.30$	AASHTO [28]
UK	0.25	BS5400 [29]
France	$[0.64/(1 + 0.2L)] \leq 0.30$ $[0.80/(1 + 0.2L)] \leq 0.30$	French cahier des prescriptions
Germany	$0.4 - 0.008L(m) - 0.1h$	DIN 1072-1976
Iran	$0.3 - 0.005L(m) - 0.15h \quad h \geq 0$	Iranian specification bridge code

varies from two to six which depends on width of bridges. All selected bridges are two-equal-span continuous, with bridge width ( $W$ ) of 9.14 m, 14.00 m, and 17.00 m.

$$\text{IF} = \frac{D_{\text{dyn}} - D_{\text{sta}}}{D_{\text{sta}}} = 1 + \frac{D_{\text{dyn}}}{D_{\text{sta}}}, \quad (1)$$

where  $D_{\text{sta}}$  and  $D_{\text{dyn}}$  are the maximum static and dynamic bridge actions, respectively [27]. Many codes specify expressions for the IF that are a function of either the span length or the bridge's first fundamental frequency. Table 1 lists the impact factors specified in the design codes for five different countries to show how the IF is typically calculated.

*2.2. Bridge Model.* One hundred twenty-five composite slab-on-girder prototype bridges with different configurations were designed and analyzed for this study. The bridge cross sections consisted of steel girders supported by concrete slabs. The prototype bridges, designed using the  $\alpha$ - $\theta$  method, covered all the possible ranges of applicability for the key parameters in AASHTO LRFD [15]. Only simply supported single-span bridges were considered. Solid steel diaphragms were incorporated at the abutments and midspan of the bridges. Figure 1 shows a cross section of a typical slab-on-girder bridge, and Table 2 presents the data for the prototype bridges modeled in this study. Five different span lengths of 30, 45, 60, 75, and 90 m were considered, representing a typical range for medium to long span bridges. The number of girders ranged from 4 to 7 depending on the bridge widths. The skew angle of the bridges ranged from  $0^\circ$  to  $60^\circ$  in  $15^\circ$  intervals. Three different bridge widths of 18, 21, and 24 m were used to represent four, five, and six lane bridges, respectively. Poisson's ratio was taken to be 0.2 for the concrete and 0.3 for the steel components. The densities of the concrete and steel were 2,450 and 7,850  $\text{kg/m}^3$ , respectively, when computing the dead load of the bridge due to self-weight.

*2.3. Numerical Modeling of Prototype Bridges.* Three-dimensional (3D) modeling of the bridge prototypes was conducted using the finite element (FE) technique and CSIBridge software V16. Figure 2 shows a typical FE mesh applied for the dynamic analyses of the bridges. 3D four-node shell elements, each with six degrees of freedom, were used to model the concrete deck, steel web, steel flanges, and solid end diaphragms. The following assumptions were applied for the finite element modeling of the

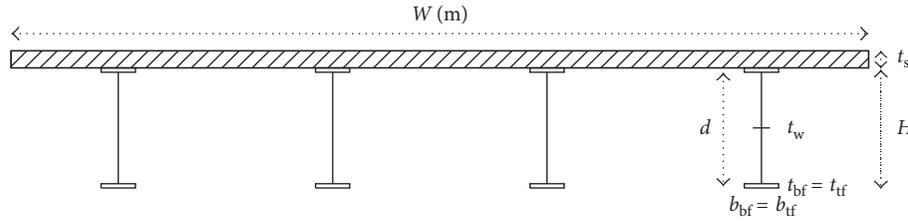


FIGURE 1: Cross-sectional view of a typical composite slab-on-girder bridge.

TABLE 2: Characteristics of the composite slab-on-girder bridges modeled in the study.

Number	$W_0$	$L$ (m)	$t_s$ (cm)	$W$ (m)	Ng	$t_w$ (cm)	$d_w$ (cm)	$t_{bf}$ (cm)	$t_{tf}$ (cm)	$b_{bf}$ (cm)	$b_{tf}$ (cm)	$d$ (cm)	$H$ (cm)
1	8.1	30	22.5	18	5	0.82	113.7	2.5	1.2	20.0	20.0	117.5	140
2	1.2	30	24.3	21	5	0.88	121.5	2.5	1.5	21.1	21.1	125.6	150
3	8.1	30	26.2	18	4	0.97	136.0	3.1	1.5	23.7	23.7	140.6	166
4	4.2	30	28.1	21	4	1.00	144.0	3.1	1.5	25.0	25.0	148.7	177
5	3.0	30	30.0	24	4	1.15	154.0	3.7	1.8	27.5	27.5	160.1	190
6	8.1	45	22.5	18	5	1.10	152.0	3.7	1.8	26.2	26.2	157.0	180
7	1.2	45	24.3	21	5	1.20	166.2	3.7	1.8	28.7	28.7	71.80	196
8	8.1	45	26.2	18	4	1.05	148.0	3.7	1.8	26.2	26.2	153.7	180
9	4.2	45	28.1	21	4	1.22	171.2	3.7	1.8	30.0	30.0	176.8	205
10	3.0	45	30.3	24	4	1.35	188.4	4.4	2.1	32.5	32.5	195.0	225
11	5.1	60	22.5	21	6	1.50	210.0	5.0	2.5	36.2	36.2	217.0	240
12	1.2	60	24.3	21	5	1.72	240.3	5.6	2.8	42.5	42.5	248.1	273
13	4.2	60	26.2	24	5	1.62	227.5	5.0	2.5	40.0	40.0	235.5	261
14	4.2	60	28.1	21	4	1.70	236.5	5.6	2.8	41.2	41.2	245.2	273
15	3.0	60	30.0	24	4	1.60	233.7	5.0	2.5	38.7	38.7	231.5	261
16	2.1	75	22.5	24	7	1.78	248.5	4.3	2.8	43.7	43.7	255.6	275
17	5.1	75	24.3	24	6	1.92	267.8	4.7	3.2	46.2	46.2	275.6	300
18	4.2	75	26.2	24	5	1.83	255.3	4.3	2.8	43.7	43.7	262.5	289
19	2.1	75	28.1	24	5	1.90	264.0	4.7	3.2	46.2	46.2	271.8	300
20	3.0	75	30.3	24	4	1.68	233.7	4.4	2.8	41.2	41.2	243.2	273
21	5.1	90	22.5	21	6	1.81	251.5	4.4	2.8	43.7	43.7	258.7	281
22	1.2	90	24.3	21	5	1.92	268.1	4.7	3.1	46.2	46.2	275.6	300
23	8.1	90	26.2	18	4	1.92	266.0	4.7	3.1	46.2	46.2	273.7	300
24	4.2	90	28.1	21	4	2.31	322.1	5.7	3.7	56.2	56.2	332.0	360
25	3.0	90	30.0	24	4	2.10	295.0	5.3	3.4	51.2	51.2	303.7	333

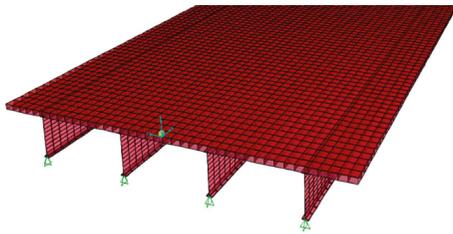


FIGURE 2: Finite element mesh used for modeling the composite slab-on-girder bridges with simple supported abutments.

bridge superstructures: (i) there is a composite interaction between the steel flanges of the girders and the reinforced slab; (ii) all materials were homogeneous and performing in the elastic region; and (iii) the reinforced slab was undamaged, with no cracks. The bridges were simply supported with a hinge at the right abutment at the bottom of each web that resisted both vertical and lateral displacement. Other supports were treated as supported on rollers at the bottom of each web that prevented only vertical translation [22, 24].

**2.4. Loading Conditions.** HL93, the LRFD-designated vehicular live load used in this study, consists of either a design truck plus design lane load or a design tandem plus lane load, as appropriate, to calculate the maximum bridge action (Figure 3). The CSIBridge software notes that certain features of the AASHTO specifications [15] for vehicular live loads apply only to certain types of bridge responses, such as negative and positive stresses or deflections along the span so factors of 1.00, 0.85, or 0.65, depending on the number of lane loads, should be applied. Applying the FEA to the three-dimensional bridges, the maximum bending moments and deflections were calculated by positioning the wheel loads at a distance of 61 cm from the curb edge of the bridge and then moving all live loads in 30 cm increments in a transverse direction. The live loads were applied according to the number of lanes, as shown in Table 1. The location of the critical live loading cases in the transverse direction across the bridges was evaluated from the preliminary studies and is shown in Figure 4. The wheel lines of trucks in adjacent lanes were taken to be 1.20 m apart.

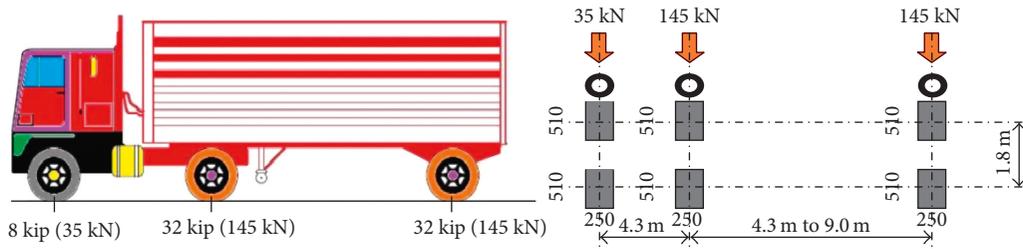


FIGURE 3: Truck characteristics based on HL93.

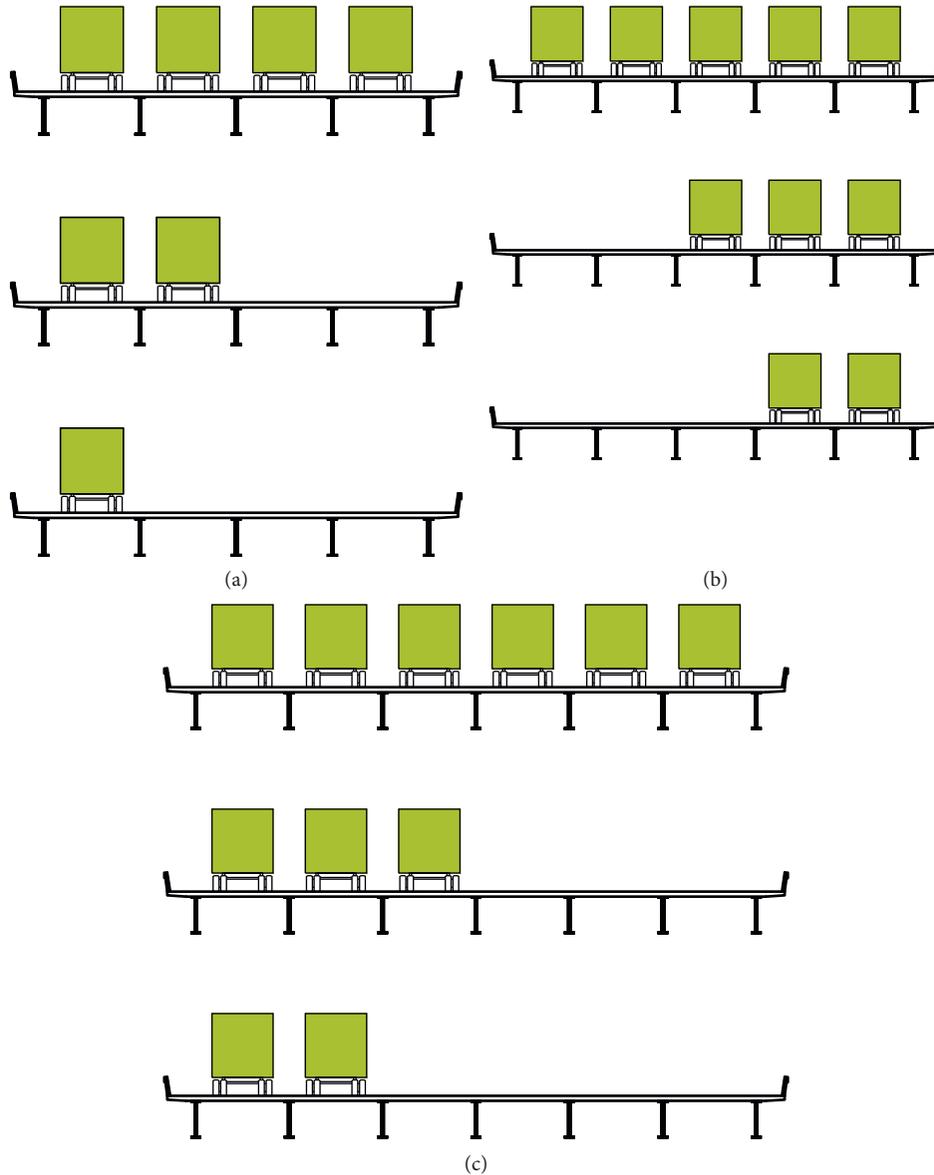


FIGURE 4: Cross-sectional view of truck arrangement on loaded bridges: (a) four lanes; (b) five lanes; (c) six lanes.

2.5. *Pavement Surface Roughness.* In the dynamic analysis of bridge under moving vehicle study, the effects of surface roughness should be taken into account, which is evaluated by the surface condition of the bridge. Many investigations have been evaluating the influence of asphalt roughness

condition. Ding et al. [7] concluded that roadway unevenness only results in a bounce in suspensions of vehicles as they enter the roadway bridge, and its effect can be ignored in bridge-vehicle interaction analysis. Hamidi and Danshjou [8] simulated the road profile as a random process

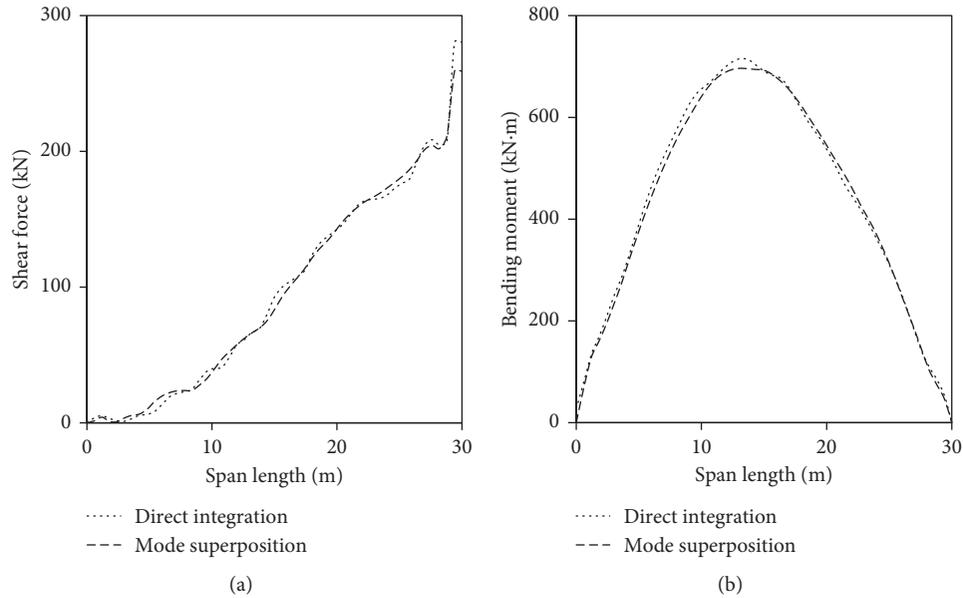


FIGURE 5: Comparison of different dynamic analysis methods for Bridge A-4. (a) Shear force; (b) bending moment.

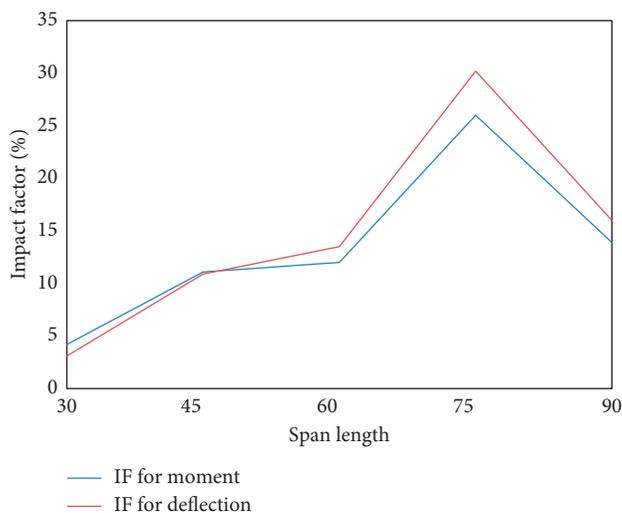


FIGURE 6: Effect of span length on the dynamic impact factors for Bridge A-4.

using a power spectral density (PSD) function based on exponential functions. Like most commercial finite element software, CSIBridge is unable to directly couple moving truck models in order to perform the dynamic bridge-vehicle interaction analyses under various surface roughness excitations. To cope with this issue, an equivalent nodal time-history excitation inputs of wheel loading from passing trucks are usually generated, and the multimode, reduced-order bridge dynamic models are used in the dynamic analysis of structures under vehicle loads. Next, modal analysis is carried out to determine the first couple critical modes for dynamic responses of structures. According to critical modes, a reduced-DOF bridge dynamic model is established to calculate the reasonable dynamic response for bridge by adopting only limited modes [30–32]. The road

surface roughness on the bridge superstructure can be simulated as a stationary Gaussian random process with a zero mean value. According to the ISO guideline (International Standard Organization), road roughness condition is classified based on the road roughness coefficient (RRC) into five major categories, which are Very Good, Good, Average, Poor, and Very Poor. In this study, it is assumed that the wearing surface of the bridge deck may be replaced before the surface condition would actually get into poor or very poor conditions. So only three roughness representative scenarios were used in this study: Very Good, Good, and Average with road roughness coefficient of  $5 \times 10^{-6} \text{ m}^3/\text{cycle}$ ,  $20 \times 10^{-6} \text{ m}^3/\text{cycle}$ , and  $80 \times 10^{-6} \text{ m}^3/\text{cycle}$ , respectively.

### 3. Results and Discussion

**3.1. Dynamic Analysis.** The general implicit method, which includes both the mode superposition method and the direct integration technique, is extensively used for time-history analyses and studies of dynamic bridge-vehicle interactions. As the mode superposition method tends to underestimate the dynamic responses for grid models due to modal clustering [20], the direct integration method utilizing CSIBridge (V20) software is used for dynamic analyses of bridges. The Wilson- $\theta$  and Hilber-Hughes-Taylor operations provide the time-history responses of bridge-vehicle interactions via a direct integration technique due to its inherent stability and economy [4]. Here, the Newmark method has been modified to introduce controllable numerical damping. A comparison between the dynamic responses (bending moment and shear force) for Bridge A-4 (Table 2) calculated using the superposition of modes method for 300 modes and direct integration reveals that direct integration produces slightly higher values than the mode superposition technique (Figure 5). In addition, given

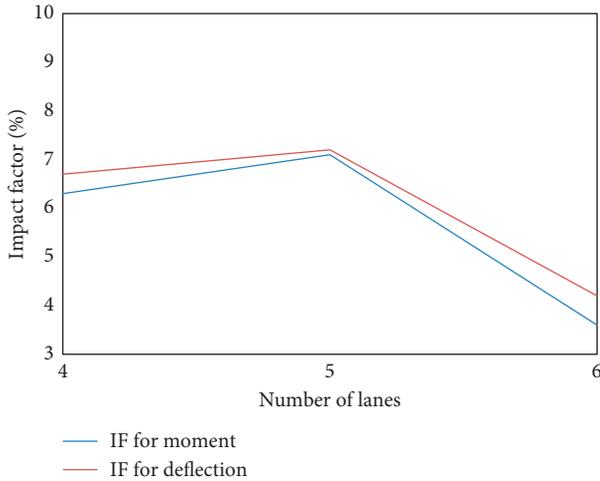


FIGURE 7: Effect of number of lane loads on the dynamic impact factor.

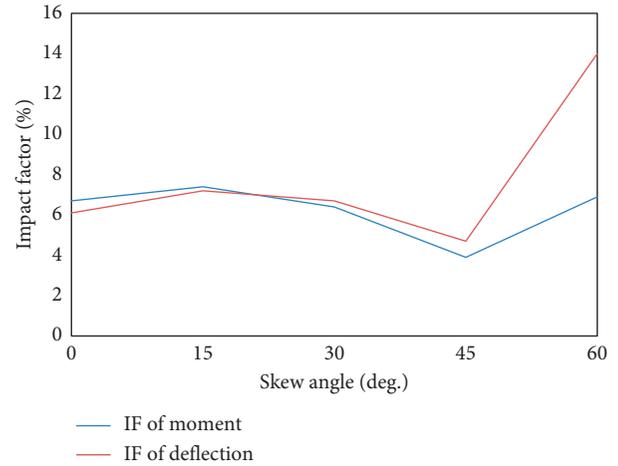


FIGURE 9: Effect of skew angle on the dynamic impact factors of Bridge A-6.

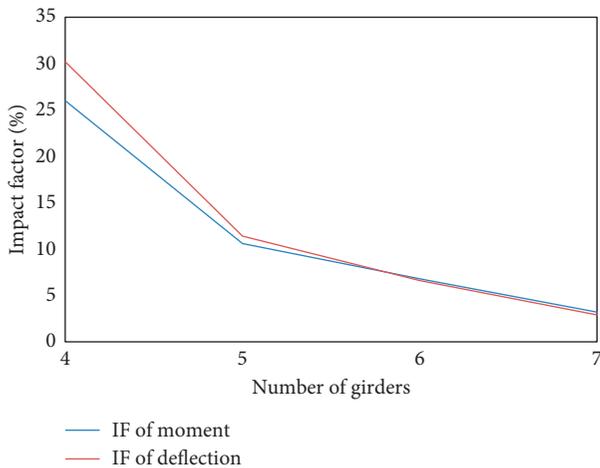


FIGURE 8: Number of girders versus the dynamic impact factor for Bridge A-10.

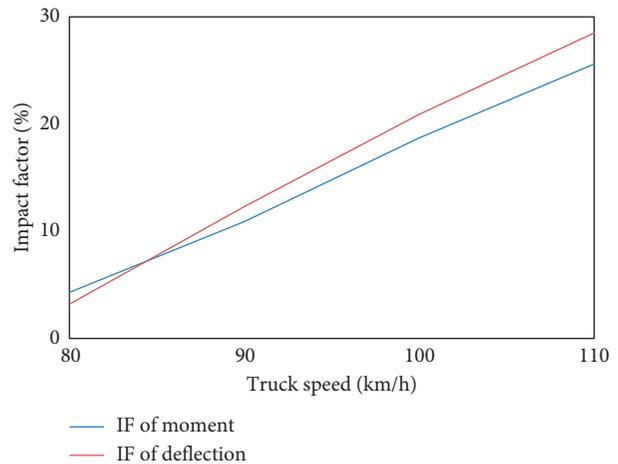


FIGURE 10: Relationship between truck speed and dynamic impact factor.

the necessity of performing a modal analysis to calculate the natural frequencies of the bridge and then clustering the frequencies obtained, the mode superposition technique is no faster than the direct integration method. Overall, the use of the direct integration method seems to be preferable for dynamic analyses of bridges.

**3.2. Parametric Study.** An extensive parametric study was carried out to consider the impact of various bridge and vehicle parameters, including the span length, number of girders, skew angle, and vehicle speed of skew, on the dynamic impact factor of composite slab-on-girder bridges.

**3.2.1. Effect of Span Length.** The span lengths of 30 m to 90 m cover the medium to long span bridges, and also, more than 90% of highway bridges are categorized in this range [33]. This study, therefore, focused on the bridges having medium to long span. It should be noted that the medium to long

span bridges would provide higher DIF compared to short span bridges. The relationship between the bridge span length and the dynamic impact factors for bending moment and deflection for Bridge A-10 (Table 2) with four girders and six lane loads are shown in Figure 6. The span length significantly affects the IF for both deflection and the bending moment, increasing as the span length changes from 30 to 75 m and then decreasing above that value. The same trends are observed for the other bridge prototypes both here and in the subsequent analyses, but due to space limitations, only a single example of each is provided here. This is the reason why the AASHTO LRFD specifications [15] limit the span length for this type of bridge to 73 m (240 ft). Although this is not easy to explain due to complex dynamic behavior of bridges, the span length must be considered a significant parameter affecting the IF.

**3.2.2. Effect of the Number of Lane Loads.** To consider the effect of the number of lanes on the dynamic responses of

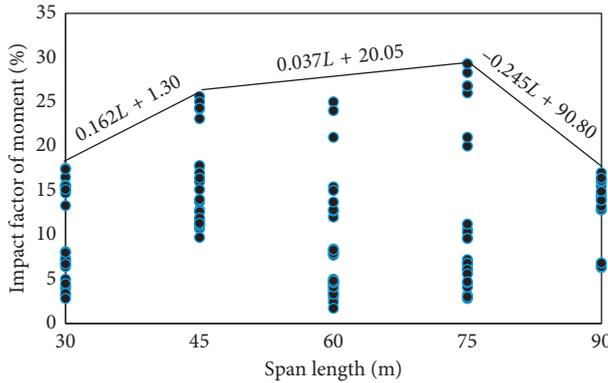


FIGURE 11: Bending moment dynamic impact factor versus span length.

prototype bridges with 4, 5, and 6 lanes, the simultaneous passage of multiple trucks traveling at a speed of 110 km/h over the bridge (fully loaded condition) was analyzed to determine the maximum dynamic responses. The dynamic IFs for bending moment and deflection versus the number of lanes for bridges with a span length of 75 m are shown in Figure 7. The IFs for bending moment and deflection follow similar trends; as the number of lanes rise from three to five, the IFs increase by 6.8%, and then as the number of lanes continue to increase, the IFs drop by 33%.

3.2.3. *Effect of the Number of Girders.* The impact of the number of girders on the dynamic responses of Bridge A-10 (Table 2) is shown in Figure 8. The IF decreases for both bending moment and deflection, with the rate of decrease increasing sharply for bridges with five or more girders, as the number of girders increase.

3.2.4. *Effect of Skew Angle.* Most bridge design codes and specifications derive IFs expressions for straight bridges, neglecting the effect of skewness. Research has revealed that the skew angle has a significant impact on a bridge’s live-load distribution, however. Figure 9 shows the effect of skew angle on the IFs for bending moment and deflection for Bridge A-6 (Table 2). Although low skewness does have a slight effect on the IFs (up to 10%), as the skew angle exceeds 45°, the IFs increase sharply. At acute angles of bridges with high skewness, significant torsional and flexural moments are induced by truck moving loads, substantially increasing the dynamic responses.

3.2.5. *Effect of Speed.* The effects of vehicle speed on the bending moment and deflection impact factors were examined for this study. A preliminary sensitive analysis was also performed to select a suitable range of vehicle speeds for composite girder bridges. As a result, it has been found that the DIF for a moment and deflection increases when the speed of the vehicle increases. The speed lower than 80 km/h, especially, had less effects on dynamic responses of bridges and induced lower DIFs for these types of bridges. In addition, North American codes AASHTO [28] and the

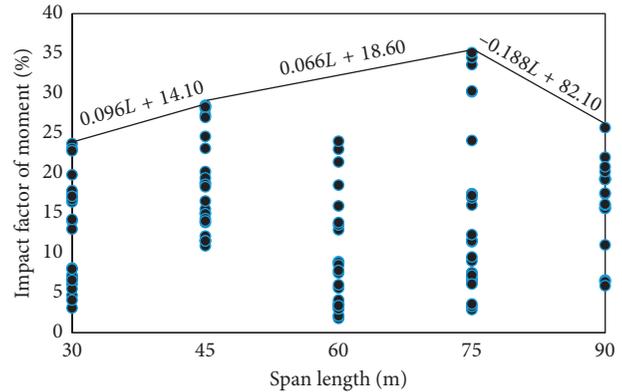


FIGURE 12: Shear force dynamic impact factor versus span length.

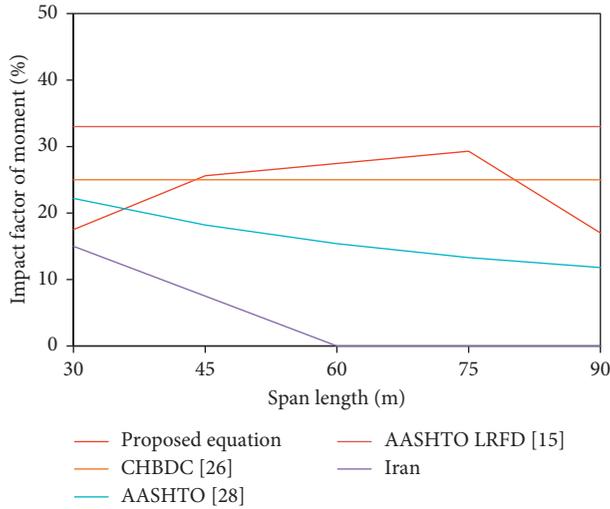
Geometric design standards for Ontario highways [18] allow trucks to travel at a maximum speed of 110 km/h on the national road networks. Hence, the range of vehicle speed was selected between 80 km/h and 110 km/h in this study.

Figure 10 presents the results for a fully loaded Bridge A-8 (Table 2). As expected, the IFs increased up to 60% as the truck speed increased from 80 to 110 km/h. A further parametric study to consider the effect of the number and position of trucks on the dynamic impact factor utilized the same maximum speed (results not shown). The impact factor was calculated for the loading case with trucks located in the center of the bridge, with the trucks being equidistant from the abutments on either side in the case of skew bridges, and traveling at the same speed in order to provide a higher bending moment and deflection.

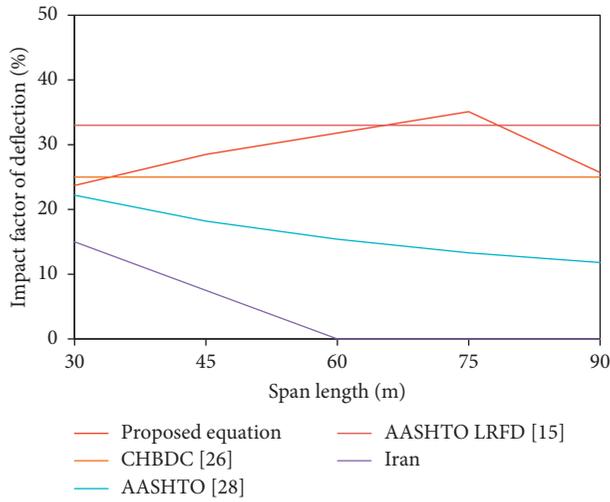
3.2.6. *Proposed Expressions for Dynamic Impact Factor.* The results of the parametric study indicated that the dynamic impact factors for bridges largely depend on the bridge geometry and truck configurations. However, it proved difficult to identify specific expressions for the function of key parameters due to the inherent complexity of the dynamic analysis and the unproven relationships between some of the influential parameters and the impact factor. Since one of the objectives of this study was to develop dynamic impact factors for bridges that do not suffer from the existing applicability limitations of the current AASHTO specification [15], a set of dynamic impact factor expressions were developed that are solely a function of span length. Realistic upper-bound envelopes were drawn to yield appropriate formulations of the proposed expressions for the dynamic impact factors for bending moment (Figure 11) and deflection (Figure 12) as follows:

- (i) Dynamic impact factor for bending moment:

$$IF_M = \begin{cases} 0.500L + 1.3 & 30 \leq L(m) \leq 45 \\ 0.013L + 20.05 & 45 \leq L(m) \leq 75 \\ 90.80 - 0.075L & 75 \leq L(m) \leq 90. \end{cases} \quad (2)$$



(a)



(b)

FIGURE 13: Comparison of proposed expressions with various codes. (a) Dynamic impact factor for bending moment; (b) dynamic impact factor for deflection.

(ii) Dynamic impact factor for deflection:

$$IF_D = \begin{cases} 0.030L + 14.1 & 30 \leq L(m) \leq 45 \\ 0.020L + 18.6 & 45 \leq L(m) \leq 75 \\ 82.1 - 0.570L & 75 \leq L(m) \leq 90. \end{cases} \quad (3)$$

The above upper-bound expressions for  $IF_D$  and  $IF_M$  are valid for  $30 \leq L(m) \leq 90$ . These expressions also accommodate variations in the factors to take into account the number of lanes and skew angle mentioned earlier. Using the proposed (2) and (3) to calculate the dynamic impact factor for simple supported bridges will automatically provide conservative results since these bridges have higher stiffness. It should be noted that the proposed expressions have taken into account the effects of road roughness, skewness, and vehicle speed.

Figure 13 demonstrates that the proposed expressions calculate dynamic impact factors for both moment and deflection that are significantly higher than those determined using the methods recommended by both AASHTO [28] and CHBDC [26] but are slightly smaller than AASHTO LRFD [15]. This is likely due to the high torsional and flexural stiffness and better load distribution characteristics associated with the LRFD bridge design principle used to design the bridge prototypes in this study. It should also be mentioned that the impact factors presented in the above specifications were based on results obtained from field testing a large number of bridge with different cross sections. In addition, these expressions consider only bending moments, which may explain the sometimes surprising differences between the impact factors suggested by the proposed expressions and those in the various codes.

#### 4. Conclusions

An extensive study was carried out to evaluate the influence of key parameters on the dynamic impact factors for skew composite concrete-steel slab-on-girder bridges. The resulting empirical expressions provide bending moment and deflection impact factors for the AASHTO truck-loading condition. Due to the scattered nature of the results obtained from dynamic analyses of bridges, upper-bound envelopes were applied to develop the proposed expressions, which are based on the bridges' span lengths in order to conform with those provided in the AASHTO [28] specifications. Based on the findings of this study, the following conclusions can be drawn:

- (i) A comparison of the proposed expressions for the IFs for bending moments and deflections with those provided in the current bridge specification indicates that apart from AASHTO LRFD, all the current specifications are unconservative.
- (ii) Truck speed has a significant influence on a bridge's dynamic responses, so any increase in truck speed will result in a significant increase in the bridge's dynamic impact factors.
- (iii) Effect of surface roughness is considered in FE models in order to reduce the complexity of proposed expressions.
- (iv) Dynamic responses of bridge that are outside the range of applicability of the AASHTO specifications require further attention.

It is expected that the findings of this study assist bridge engineers seeking to design composite slab-on-girder bridges more reliably and economically. They could also be applied to reassess the safe live-load capacity of existing structures, thus potentially preventing the unnecessary posting or closing of busy highway bridges.

#### Data Availability

The data sets generated/analyzed during the current study are available from the corresponding author on reasonable request.

## Conflicts of Interest

The authors declare that they have no conflicts of interest.

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## References

- [1] J. Humar and A. H. Kashif, "Dynamic response analysis of slab-type bridges," *Journal of Structural Engineering*, vol. 121, no. 1, pp. 48–62, 1995.
- [2] L. Deng, Y. Yu, Q. L. Zou, and C. S. Cai, "State-of-the-art review on dynamic impact factors of highway bridges," *Journal of Bridge Engineering*, vol. 20, no. 5, 2015.
- [3] Y. Yu, C. S. Cai, and L. Deng, "State-of-the-art review on bridge weigh-in-motion technology," *Advances in Structural Engineering*, vol. 19, no. 9, pp. 1514–1530, 2016.
- [4] R. Cantieni, "Dynamic load testing of highway bridges," *Transportation Research Record*, vol. 950, pp. 141–148, 1984.
- [5] T. H. T. Chan and C. O'Connor, "Wheel loads from highway bridge strains: field studies," *Engineering Structures*, vol. 116, no. 7, pp. 1751–1771, 1990.
- [6] K. M. Sennah, X. Zhang, and J. B. Kennedy, "Impact factors for horizontally curved composite box girder bridges," *Journal of Bridge Engineering*, vol. 9, no. 6, pp. 512–520, 2004.
- [7] L. Ding, H. Hao, and X. Zhu, "Evaluation of dynamic vehicle axle loads on bridges with different surface conditions," *Journal of Sound and Vibration*, vol. 323, no. 3–5, pp. 826–848, 2009.
- [8] S. A. Hamidi and F. Danshjo, "Determination of impact factor for steel railway bridges considering simultaneous effects of vehicle speed and axle distance to span length ratio," *Engineering Structure*, vol. 32, no. 5, pp. 1369–1376, 2010.
- [9] C. Caprani, "Lifetime highway bridge traffic load effect from a combination of traffic states allowing for dynamic amplification," *Journal of Bridge Engineering*, vol. 18, no. 9, pp. 901–909, 2012.
- [10] B. Wang and H. He, "Vehicle-bridge interaction analysis based on the ANCF quasi-conforming plate technique," *IOP Conference Series: Materials Science and Engineering*, vol. 216, pp. 1–5, 2017.
- [11] D. Chang and H. Lee, "Impact factors for simple-span highway girder bridges," *Journal of Structural Engineering*, vol. 120, no. 3, pp. 704–715, 1994.
- [12] D. Huang, T. L. Wang, and M. Shahawy, "Impact studies of multigirder concrete bridges," *Journal of Structural Engineering*, vol. 119, no. 8, pp. 2387–2402, 1992.
- [13] L. Deng and C. S. Cai, "Development of dynamic impact factor for performance evaluation of existing multi-girder concrete bridges," *Engineering Structures*, vol. 32, no. 1, pp. 21–31, 2010.
- [14] Z. Sun and Z. Zou, "Towards an efficient method of predicting vehicle-induced response of bridge," *Vehicle Induced Response of Bridge*, vol. 33, no. 7, pp. 2067–2089, 2016.
- [15] AASHTO-LRFD (American Association of State Highway and Transportation Officials), *AASHTO LRFD Bridge Design Specifications: Customary US Units*, AASHTO-LRFD, Washington, DC, USA, 7th edition, 2014.
- [16] KRTA (Korean Roadway Transportation Association), *Korean Bridge Design Specifications (KBDS). Roadway Standard Specification Codes*, Ministry of Land, Transport and Maritime Affairs, South Korea, 2005.
- [17] JRA (Japanese Road Association), *Japanese Road Association's Specifications (JRAS). Part 1: Common Specifications for Highway Bridges*, Japanese Road Association, Japan, 3rd edition, 2000.
- [18] OHBDC, *Ontario Highway Bridge Design Code*, Ontario Ministry of Transportation and Communications, Downsview, ON, Canada, 1991.
- [19] J. Billing, "Dynamic loading and testing of bridges in Ontario," *Canadian Journal of Civil Engineering*, vol. 1, no. 4, pp. 833–843, 1984.
- [20] M. Fafard, M. Laflamme, M. Savard, and M. Bennur, "Dynamic analysis of existing continuous bridge," *Journal of Bridge Engineering*, vol. 3, no. 1, pp. 28–37, 1998.
- [21] X. J. Zheng, J. J. Wu, and Y. Zhou, "Effect of spring non-linearity on dynamic stability of a controlled maglev vehicle and its guideway system," *Journal of Sound and Vibration*, vol. 279, no. 1-2, pp. 201–215, 2003.
- [22] I. Mohseni, A. R. Khalim, and E. Nikbakht, "Effectiveness of skewness on dynamic impact factor of concrete multicell box-girder bridges subjected to truck loads," *Arabian Journal of Science and Engineering*, vol. 39, no. 8, pp. 6083–6097, 2014.
- [23] D. B. Ashebo, T. H. T. Chan, and L. Yu, "Evaluation of dynamic loads on a skew box girder continuous bridge part I: field test and modal analysis," *Engineering Structures*, vol. 29, no. 6, pp. 1052–1063, 2007.
- [24] L. Deng, W. He, and Y. Shao, "Dynamic impact factors for shear and bending moment of simply supported and continuous concrete girder bridges," *Journal of Bridge Engineering*, vol. 20, no. 11, article 04015005, 2015.
- [25] X. Zhang, K. Sennah, and J. B. Kennedy, "Evaluation of impact factors for composite concrete-steel cellular straight bridges," *Engineering Structures*, vol. 25, no. 3, pp. 313–321, 2003.
- [26] CHBDC (Canadian Highway Bridge Design Code), *CAN/CSA-S6-00*, CSA International, Toronto, ON, Canada, 2006.
- [27] L. Kwasniewski, J. Wekezer, G. Roufa et al., "Experimental evaluation of dynamic effects for a selected highway bridge," *Journal of Performance of Constructed Facilities*, vol. 20, no. 3, pp. 253–260, 2006.
- [28] AASHTO (American Association of State Highway and Transportation Officials), *Standard Specification for Highway Bridges*, AASHTO, Washington, DC, USA, 17th edition, 2002.
- [29] BS EN 1991-2, *Eurocode 1: Actions on Structures-Part 2: Traffic Loads on Bridges*, British Standard Institution, London, UK, 2003.
- [30] Y. Xu and W. Guo, "Dynamic analysis of coupled road vehicle and cable-stayed bridge systems under turbulent wind," *Engineering Structures*, vol. 25, no. 4, pp. 473–486, 2003.
- [31] C. Cai and S. Chen, "Framework of vehicle-bridge-wind dynamic analysis," *Journal of Wind Engineering and Industrial Aerodynamics*, vol. 92, no. 7-8, pp. 579–607, 2004.
- [32] Y. Zhou and S. Chen, "Dynamic simulation of a long-span bridge-traffic system subjected to combined service and extreme loads," *Journal of Structural Engineering*, vol. 141, no. 9, article 04014215, 2014.
- [33] NCHRP (National Cooperative Highway Research Program), *Transportation Research Board 2016 Annual Report*, 2016.

