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Evaluation of existing prestressed concrete bridges considering the randomness of live load distribution factor due to random vehicle loading position

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Abstract

The live load distribution factor is a very important parameter in both the design of new bridges and the evaluation of in-service bridges. Studies have shown that there can be large discrepancy between the actual load distribution factors of field bridges and the load distribution factors predicted by bridge design codes. In addition, the load distribution factor is always treated as a constant in bridge assessment even though it is a random variable with certain statistical properties. In this study, the reliability indexes of 15 prestressed concrete girder bridges designed following the AASHTO LRFD code are calculated by considering the randomness of the load distribution factors induced by the random vehicle transverse position. It is found that there is a considerable increase in the calculated bridge reliability indexes, especially for short-span bridges, when the load distribution factor is modeled as a random variable with the statistical properties obtained from numerical simulations. This suggests that vehicle transverse position is one important factor that can be considered if a refined analysis is desirable when traditional evaluation methods predict unsatisfactory bridge assessment results. The findings in this article also highlight the importance of considering the actual vehicle transverse position in the evaluation of existing bridges.

Keywords

girder bridges, load distribution factor, prestressed concrete, reliability index, vehicle transverse position

Introduction

The lateral distribution of vehicular live load has received a substantial amount of attention from researchers in recent years. Adopting reasonable load distribution factors (LDFs) is of vital importance to both the design and the evaluation of bridges. However, most studies have focused on the LDFs related to bridge design while the LDFs for the performance evaluation of existing bridges, which are equally important, have received much less attention (Bae and Oliva, 2012; Chung et al., 2006; Gheitasi and Harris, 2015; Kim, 2012; Moses et al., 2006; Razaqpur et al., 2012).

The amount of live load distributed to a particular girder depends on the loading position of the vehicle and the load-distributing characteristics of the bridge. A few important parameters, including girder spacing, span length, deck thickness, and so on, have proven to have a significant impact on the LDFs (Harris, 2010). Some secondary elements, such as parapets and diaphragms, have also proven to have a remarkable effect on the LDFs (Chung et al., 2006; Conner and Huo, 2006). For curved bridges, some researchers found that the radius and the cross-frame spacing can influence the LDFs considerably based on studies on curved steel I-girder bridges (Kim et al., 2007; Nevling et al., 2006). In addition, the effect of truck configurations on the LDFs was also investigated by some researchers (Gheitasi and Harris, 2015; Seo et al., 2014a, 2014b; Seo and Hu, 2015), and the findings revealed that vehicles' characteristics had a considerable impact on the LDFs. However, the actual vehicle

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loading condition, including the vehicle transverse position which has a considerable effect on the LDFs of in-service bridges, was usually neglected. As a result, the LDF was usually treated as a constant even though it is a random variable with certain statistical properties (Kim and Nowak, 1997). Therefore, the accuracy of bridge evaluation results could be improved if more reasonable LDFs were considered.

In this article, the LDFs for 15 prestressed concrete bridges with American Association of State Highway and Transportation Officials - Prestressed Concrete Institute (AASHTO-PCI) girders were analyzed based on the finite element models. These bridges are good representatives of the simply supported concrete girder bridges in the United States. The vehicle transverse position, which was usually ignored in previous studies, was taken into account. The bridge reliability indexes were calculated by modeling the LDF as a constant and a random variable with obtained statistical properties, respectively, and the results were compared.

It should be noted that there are many factors that can lead to the conservativeness of bridge codes, such as the boundary conditions, composite action between the bridge deck and girders, material properties, and so on. This study attempts to explore the effect of one important factor, that is, the vehicle transverse loading position, rather than all the influencing factors, on the bridge evaluation results. The findings of this study highlight the importance of using the actual vehicle transverse loading position in the performance evaluation of existing bridges.

Properties of selected bridges

A group of slab-on-girder bridges were investigated in this study. All these bridges have a bridge width of 11.4 m and a deck thickness of 0.19 m. The span lengths of these bridges range from 10 to 50 m. Concrete diaphragms were used at both ends of the bridges while no intermediate diaphragms were used. Three different girder spacings were selected, namely, 2.2, 2.94, and 4.0 m. The girder spacings of 2.2 and 2.94 m and the slab thickness of 0.19 m are commonly used in the United States (Yousif and Hindi, 2007). As using fewer girders has become a favorable choice under many conditions, a larger girder spacing of 4 m, which was also used in Eamon and Nowak's (2005) study, was also investigated in this study. As a result, a total of 15 simply supported prestressed concrete bridges with AASHTO-PCI (types II–VI) girders were considered.

Figure 1 shows a typical cross section of the selected bridges. The widths of both the traffic lanes and the verges of the bridges were set equal to those of the tested bridge from which the vehicle transverse position data were collected (Getachew, 2003).

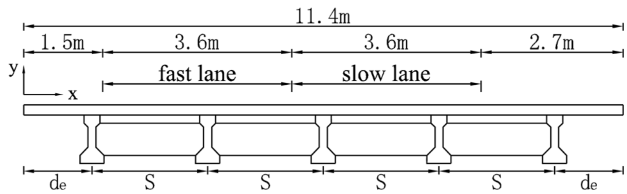

Figure 1. Typical cross section of the selected bridges.

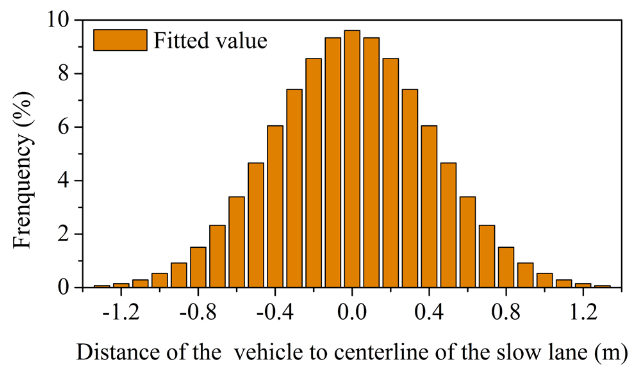

Figure 2. Distribution of vehicle transverse positions.

Transverse distribution of vehicles on bridges

The probability distribution of vehicle transverse position has been studied by researchers in different countries. Results from previous studies and bridge codes suggest that the vehicle transverse position can be described by normal distributions (BS 5400-10:1980, 1980; Bu et al., 2015; EN 1991-2:2003, 2003; Getachew, 2003). The probability distribution from Getachew (2003) was adopted in this study. It should be noted that using the distribution by Getachew (2003) was only for the purpose of illustration and was never meant to represent the actual distribution of the truck transverse loading positions. The tested bridge had two 3.6-m-wide traffic lanes in one direction and two verges. A total of 26,633 vehicles were recorded, 3011 of which were heavy vehicles. A total of 93% of the heavy vehicles were found to drive in the slow lane. More details about the field measurements can be found in Getachew (2003). In this study, only the heavy vehicles which cause considerable bridge responses were considered. A normal distribution can effectively describe the transverse positions of the heavy vehicles in the slow lane as shown in Figure 2.

Figure 2 also shows that drivers have a tendency to drive in the middle of the lane, which was also observed by Kim and Nowak (1997).

Finite element validation and analysis

In this study, the concrete bridges, including the girders, deck, and diaphragms, were modeled using three-dimensional solid elements with three translational degrees of freedom (DOFs) at each node. The element sizes were set to 0.1 m (width) × 0.2 m (length) × 0.1 m (depth). Full composite action was assumed between the bridge deck and the girders by sharing the same nodes at the interface. The bridge girders were simply supported on the two ends. The finite element program ANSYS 14.5 (n.d.) was used to create the bridge models. It should be noted that only the elastic range of bridge responses was considered in this study for the following reason: under routine traffic conditions (with regular heavy trucks), the behavior of bridges can be described by linear elastic models with sufficient accuracy, even with cracks present in the concrete (Eom and Nowak, 2001; Gheitasi and Harris, 2015). This is the reason why most researchers adopted linear elastic models for bridges when determining the LDFs for concrete bridges (Khaloo and Mirzabozorg, 2003; Mabsout et al., 2004; Song et al., 2003; Zokaie, 2000).

Before carrying out further analysis, the accuracy of the modeling method was verified. The benchmark selected for validation is the bridge documented by Hays et al. (1995), which was also used for verification in the studies by Chen and Aswad (1996) and Yousif and Hindi (2007). It was a simply supported bridge with a span length of 14.70 m and a slab thickness of 0.178 m. Six AASHTO-PCI type II concrete girders were spaced at 2.26 m, leaving a deck overhang of 0.80 m on each side. No intermediate diaphragms were used for the bridge. The compressive strength of concrete was 23.44 and 34.47 MPa for the slab and girders, respectively. A typical Poisson's ratio of 0.2 was adopted. Two HS-20 trucks with axle spacing of 4.27 m were adopted for vehicle loading. The comparison between the moment LDFs obtained by Yousif and Hindi (2007) and the results from this study is presented in Figure 3. Only slight differences are observed between the LDFs obtained from the two studies, indicating that the modeling method adopted in this study is reliable.

In Tabsh and Tabatabai's (2001) study, four oversize trucks with different configurations were examined when evaluating the LDF and the HS-20 truck was found to be the most critical truck configuration. Therefore, in this study, the HS-20 truck specified in the AASHTO LRFD (2012) code was adopted for vehicle loading. This truck has three axles, each weighing 35 (8), 145 (32), and 145 kN (32 kips), respectively, as shown in Figure 4. In the longitudinal direction, the truck position that produces the maximum bending moment at the bridge mid-span was adopted. In the transverse direction, the allowable range for the transverse truck position was defined, as shown in Figure 5. In the simulations, the truck can shift its position by a step of 0.1 m within the range. It was assumed that only one heavy truck will be present in the slow lane of the bridge at a time. This resulted in 27 loading cases with different truck transverse positions. The moment LDFs for different girders were then taken as the ratio of the bending moment in the particular girder to the total bending moment in all girders. Finally, a total of 27 moment LDFs were obtained for each girder of each bridge.

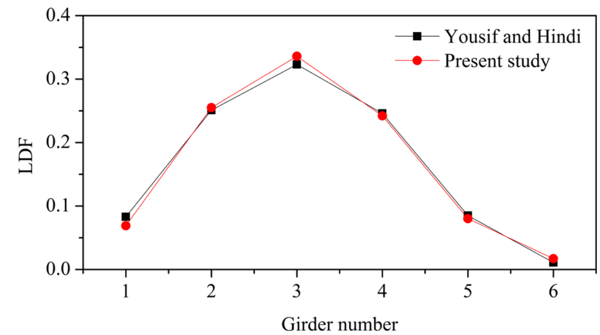

Figure 3. Comparison of moment LDFs between Yousif and Hindi (2007) and this study.

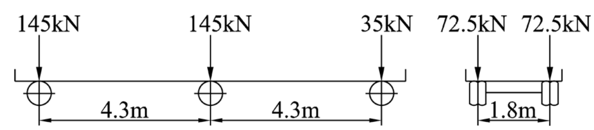

Figure 4. Characteristics of the live load model.

It should be noted that the LDF probability distribution curve adopted in this study was obtained by considering only the cases with one truck present at different transverse positions within one traffic lane. The cases with two trucks present side by side in two lanes were not considered for the following reasons. First, the side-by-side presence probability of two heavy trucks on the bridge is very low and will only cause slight changes to the LDF probability distribution obtained by considering only one lane. Fu and You (2009) found that the side-by-side presence probability of two trucks was only 1.5% and 3% (annual average daily truck traffic (ADTT) = 4000), respectively, for the two bridges investigated. Yet, these percentages did not exclude the cases with relatively light trucks in one

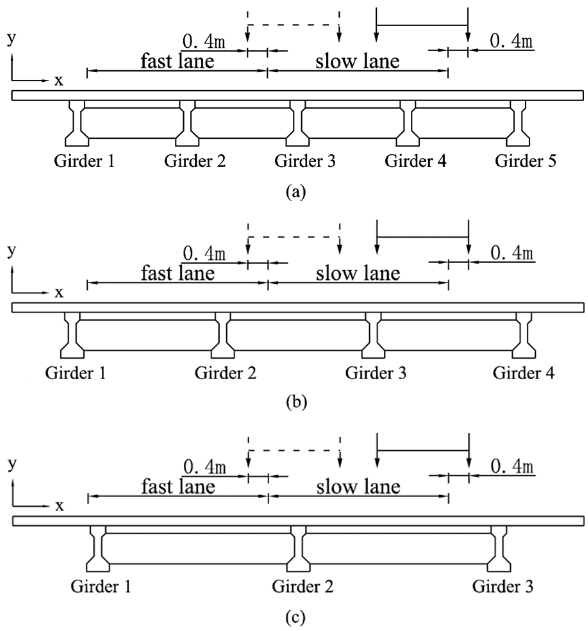

Figure 5. The range of allowable truck transverse positions in the slow lane: (a) girder spacing = 2.2 m, (b) girder spacing = 2.94 m, and (c) girder spacing = 4 m.

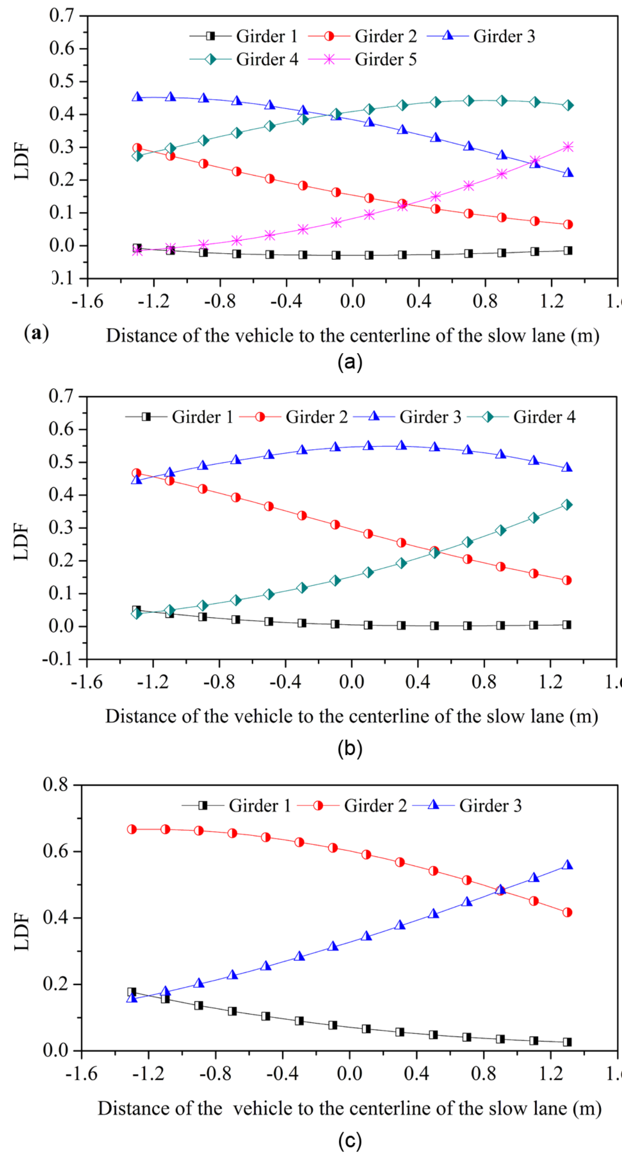

Figure 6. Load distribution factors of each girder (span = 20 m): (a) girder spacing = 2.2 m, (b) girder spacing = 2.94 m, and (c) girder spacing = 4 m.

lane or both lanes. Second, the combination of different transverse positions of two trucks side by side on a bridge is a very complicated problem. There is no statistics available that describes the probability distribution of the truck transverse positions, and it is very difficult to simulate such scenarios since the positions of the two trucks are mutually dependent due to the safety awareness of the two drivers.

Results

Probability distribution of LDFs

The bending moment LDFs for each girder under different truck loading conditions were obtained from finite element analysis. For the purpose of illustration, the moment LDFs of the three bridges with a span length of 20 m are plotted against the transverse truck position, as shown in Figure 6. In generally, it can be observed that the LDFs increase as the truck gets closer to the girder considered. Similar results were observed for bridges with other span lengths.

As shown in Figure 7, with the LDFs computed for different vehicle transverse positions, the probability distribution of the LDFs can then be obtained based on the LDFs calculated at each transverse loading position and the corresponding frequency of occurrence presented in Figure 2. Unlike the bridges with girder spacings of 2.2 and 4 m, for which the LDFs generally change monotonically when the truck moves from one side to the other transversely, for the bridges with girder spacing of 2.94 m, the LDFs first increase and then decrease, as illustrated by the LDFs of girder 3 (the control girder) in Figure 6. As a result, large LDFs occur when the truck is close to the center of the slow lane where the occurrence frequencies of truck are large, as shown in Figure 2. This leads to higher probabilities of large LDF values for the bridges with 2.94 m girder spacing, as shown in Figure 7(b), which is different from the results for the bridges with girder spacings of 2.2 and 4 m. It should be emphasized again that the probability distributions of the moment LDFs in Figure 7 were calculated based on the LDFs

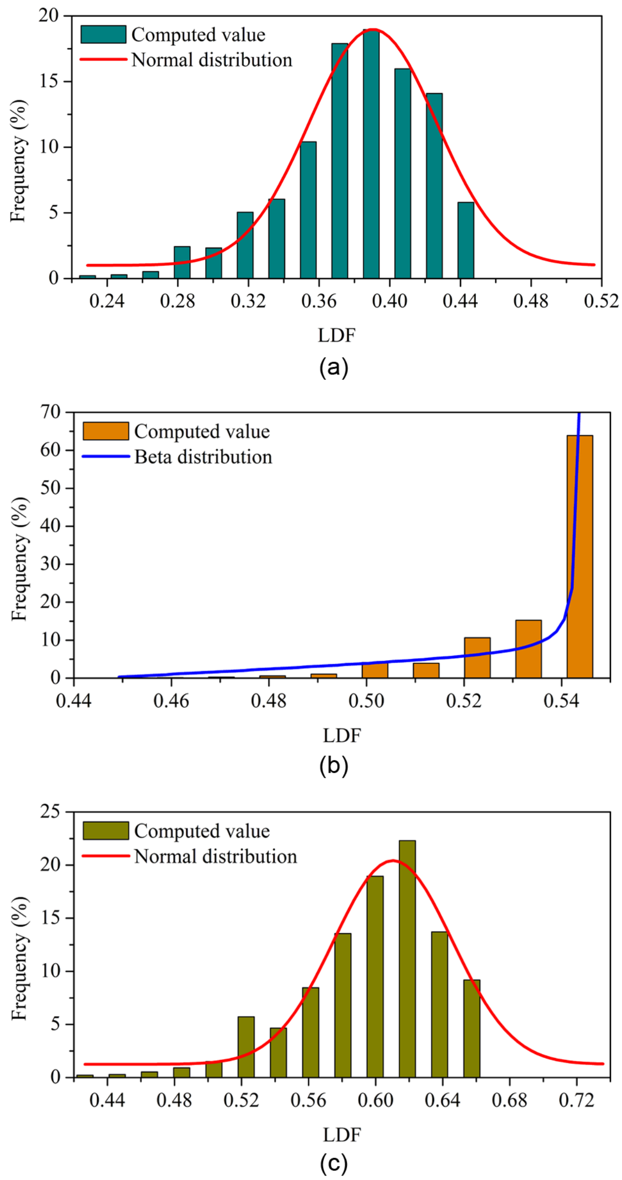

Figure 7. Probability distribution of the moment LDFs: (a) girder spacing = 2.2 m, (b) girder spacing = 2.94 m, and (c) girder spacing = 4.0 m.

calculated from finite element analysis (shown in Figure 6) and assumed normal distribution of vehicle transverse position based on field measurement results.

The distribution type of the LDFs was determined by performing a Kolmogorov–Smirnov (K-S) test, which is an exact method for determining distribution type by comparing the K-S test value against a threshold value which depends on the number of intervals and the significance level set for the test. The K-S test value is defined as follows

$$D_n = \max\{|F_n(X_i) - F_0(X_i)|\} \tag{1}$$

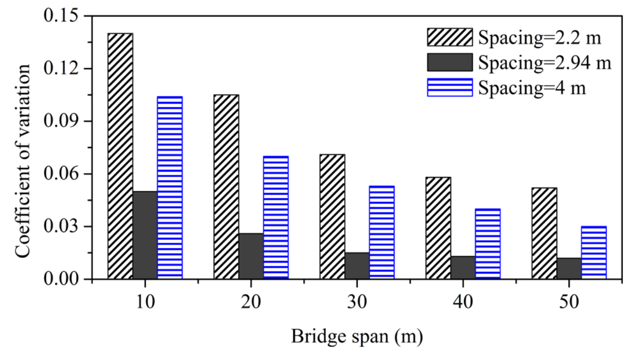

Figure 8. Coefficient of variation for each bridge.

where n is the total number of intervals of histogram, X_i is the upper limit value in the ith interval, F_0 is the empirical cumulative distribution function, and F_n is the theoretical cumulative distribution function.

The number of intervals of the histogram was set to be 13 for the bridges with girder spacings of 2.2 and 4 m and 10 for the bridges with girder spacing of 2.94 m, respectively. A significance level of 0.99 was used for all bridges. As a result, the threshold value for the K-S test was set to be 0.268 for the bridges with girder spacings of 2.2 and 4 m and 0.294 for the bridges with 2.94 m girder spacing, respectively. The computed LDFs for the bridges with girder spacings of 2.2 and 4 m were tested against normal, log-normal, and extreme-I distributions, which are frequently used in the engineering field. For bridges with girder spacing of 2.94 m, the partial beta, log-normal, and extreme-I distributions were used for the distribution test. Table 1 shows the test results for all bridges.

As can be seen from Table 1, the K-S test values are all below the threshold values except for the bridges with girder spacing of 2.94 m. This indicates that the empirical distribution does not reject the three tested distribution types for the bridges with girder spacing of 2.2 and 4 m while the Normal distribution fits the data best with a minimum test value. For the bridges with girder spacing of 2.94 m, only the partial beta distribution passed the test among the three tested distribution types. As a result, the normal distribution was used for the bridges with girder spacings of 2.2 and 4 m, and the partial beta distribution was adopted for the bridges with girder spacing of 2.94 m.

Figure 8 shows the coefficient of variation (COV) of the LDFs for the critical interior girders of every bridge. For bridges with the same girder spacing, the value of COV decreases with the increase in span length. The largest value of COV is observed in the bridges with a spacing of 2.2 m among all the bridges with the same span length. It can be inferred from Figure 8 that the span length and the girder spacing both have a significant impact on the COV of LDFs.

Table 1. Kolmogorov–Smirnov test results on the distribution of LDFs for all bridges.

Distribution type	Span length (m)				
	10	20	30	40	50
(a) Girder spacing = 2.2 m					
Normal	0.091	0.076	0.064	0.074	0.076
Log-normal	0.118	0.090	0.069	0.084	0.083
Extreme-I	0.119	0.111	0.140	0.150	0.151
(b) Girder spacing = 2.94 m					
Partial beta	0.211	0.281	0.229	0.185	0.261
Partial log-normal	0.286	0.289	0.293	0.307	0.291
Partial extreme-I	0.286	0.304	0.322	0.341	0.311
(c) Girder spacing = 4 m					
Normal	0.081	0.090	0.081	0.085	0.066
Log-normal	0.129	0.109	0.089	0.117	0.077
Extreme-I	0.168	0.167	0.111	0.162	0.129

LDF: load distribution factor.

Table 2. Statistical properties for load and resistance.

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	1.24–1.35	0.18	Extreme type I
Resistance	Moment	1.05	0.075	Lognormal

COV: coefficient of variation.

Reliability-based bridge evaluation

In the current practice of bridge assessment, the reliability of individual structural members is usually evaluated instead of the entire structural system. Therefore, only the girder with the maximum LDF was evaluated in this study. Figure 9 shows the maximum LDFs observed for all the loading cases. It is found that for bridges shorter than 40 m, the interior girders are consistently the most heavily loaded girders. However, for bridges with longer spans, the exterior girders close to the slow lane take more live load, as shown in Figure 9. In practice, the use of parapets has shown to reduce the LDF of exterior girders significantly (Conner and Huo, 2006). Therefore, only the critical interior girders were investigated in this study.

In this study, the limit state function shown in equation (2) was adopted in the reliability analysis

$$g = C - D_P - D_C - D_W - LDF(D_{Ll} + (1 + IM)D_{Lt}) \tag{2}$$

where C is the capacity of the girder; D_P and D_C are the dead load effects induced by the prestressed concrete and cast-in-place concrete bridge components, respectively; D_W is load effect induced by the wearing surface; D_{Ll} and D_{Lt} are the live load effects due to the lane load and truck load excluding the LDF; IM is the dynamic impact factor and is taken as 0.33 according to the AASHTO LRFD (2012) code; and LDF is the live-load distribution factor. As a result, the total load effect $D = D_p + D_c + D_w + LDF$ $(D_{Ll} + (1 + IM)D_{Lt})$. The statistical parameters for the load and resistance used by Nowak (1995) were employed in this study, as shown in Table 2.

The probability of failure P_f is defined as the probability that the total load effect exceeds the capacity of the structure. The relationship between the reliability index β and the failure probability P_f is described by the following equation

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

where Φ denotes the standard normal cumulative distribution function.

In this article, the Rackwitz–Fiessler algorithm (Rackwitz and Flessler, 1978) that involves an iterative calculation was employed to calculate the bridge reliability index. The Rackwitz–Fiessler algorithm can not only deal with parameters with normal distributions

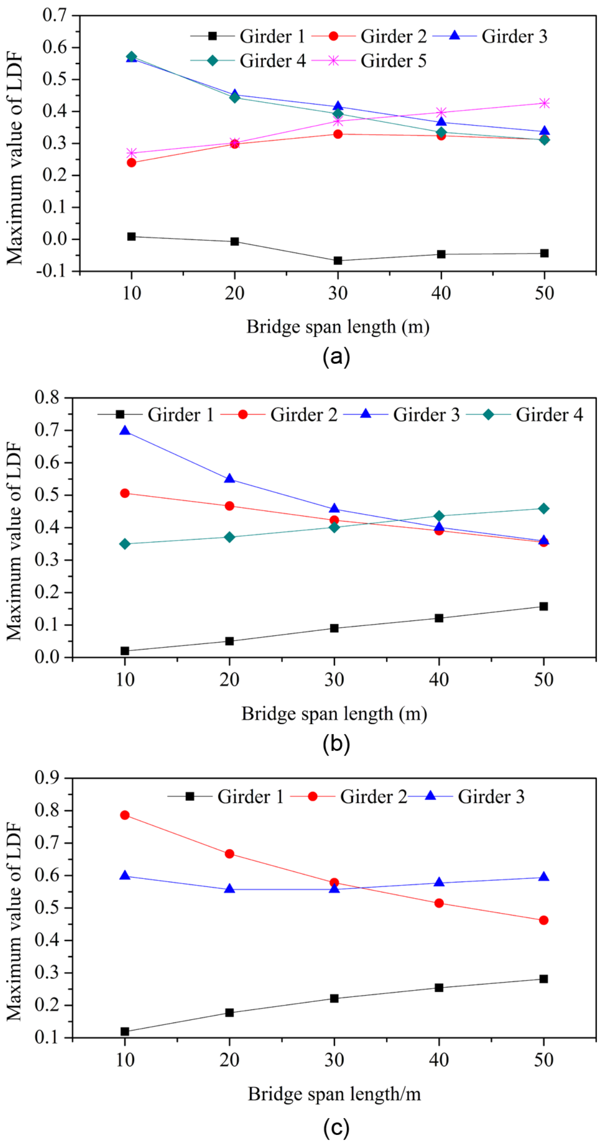

Figure 9. Maximum LDF values of all girders: (a) girder spacing = 2.2 m, (b) girder spacing = 2.94 m, and (c) girder spacing = 4.0 m.

but also parameters with non-normal distributions. In the latter case, equation (4) can be used to determine the reliability index β at the initial design point

$$\beta = \frac{g\left(x_1^*, x_2^*, \ldots, x_n^*\right) + \sum_{i=1}^{n} \frac{\partial g}{\partial X_i}\Big|_P \left(\mu_{X'_i} - x_i^*\right)}{\sqrt{\sum_{i=1}^{n} \left(\frac{\partial g}{\partial X_i}\Big|_P \sigma_{X'_i}\right)^2}} \tag{4}$$

where X_i is the variable in the limit state function; x_i^* is the value of variables at the design point; $\mu_{X'_i}$ and $\sigma_{X'_i}$ are the equivalent normal mean and standard deviation of variables; and $\partial g/\partial X_i$ is the partial derivative evaluated at the design point.

The relationship between the design point x_i^* and reliability index β can be defined as follows

$$x_i^* = \mu_{X'_i} + \alpha_{X'_i}\sigma_{X'_i}\beta \tag{5}$$

$$\alpha_{X'_i} = \cos\theta_{X'_i} = -\frac{\frac{\partial g}{\partial X_i}\Big|_P \sigma_{X'_i}}{\sqrt{\sum_{j=1}^{n} \left(\frac{\partial g}{\partial X_j}\Big|_P \sigma_{X'_j}\right)^2}} \tag{6}$$

Then, the above steps will be repeated until the design point x_i^* and reliability index β become converged. More details of the Rackwitz–Fiessler algorithm can be found in Nowak and Collins (2000).

Traditional bridge evaluation methods treat the LDF as a constant that equals to the maximum LDF obtained from structural analysis. The use of the maximum LDF in the evaluation may be too conservative and may lead to underestimated bridge reliability index (Enright and Frangopol, 1999; Tabsh and Nowak, 1991). In this article, the LDF was considered as a random variable with statistic properties obtained from numerical simulations as described in previous sections. Since the statistic properties of the LDFs were obtained based on the actual vehicle transverse positions and the corresponding frequency of occurrence collected from field tests, they represent the actual vehicle loading condition better than a single LDF value as adopted by the traditional methods.

For the purpose of comparison, the bridge reliability indexes were evaluated by treating the LDF as a constant and a random variable, respectively. Figure 10 shows the results of the critical interior girders for the 15 bridges investigated. It is observed from these three subfigures that the reliability indexes consistently increase when the actual statistic properties of LDFs are considered, especially for short-span bridges. As shown in Figure 10(a) and (c), the reliability indexes have increased by 0.4 for bridges with a span length of 10 m. It is also noticed that the influence of using the actual statistic properties of LDFs on the reliability indexes declines when the bridge span length increases. However, for bridges with a girder spacing of 2.94 m, considering the actual statistic properties of LDFs has a negligible impact on the reliability indexes. This is mainly because the COV of LDFs for bridges with 2.94 m girder spacing is much smaller than bridges with girder spacings of 2.2 and 4 m.

It is also observed that the reliability indexes of the 10 m-span bridges with girder spacings of 2.2 and 2.94 m are all below 3.5. This may be due to the reason that the LDFs obtained from the finite element analysis are all larger than the LDFs predicted by the AASHTO LRFD (2012) code for these two bridges and lead to relatively small reliability indexes. Similar

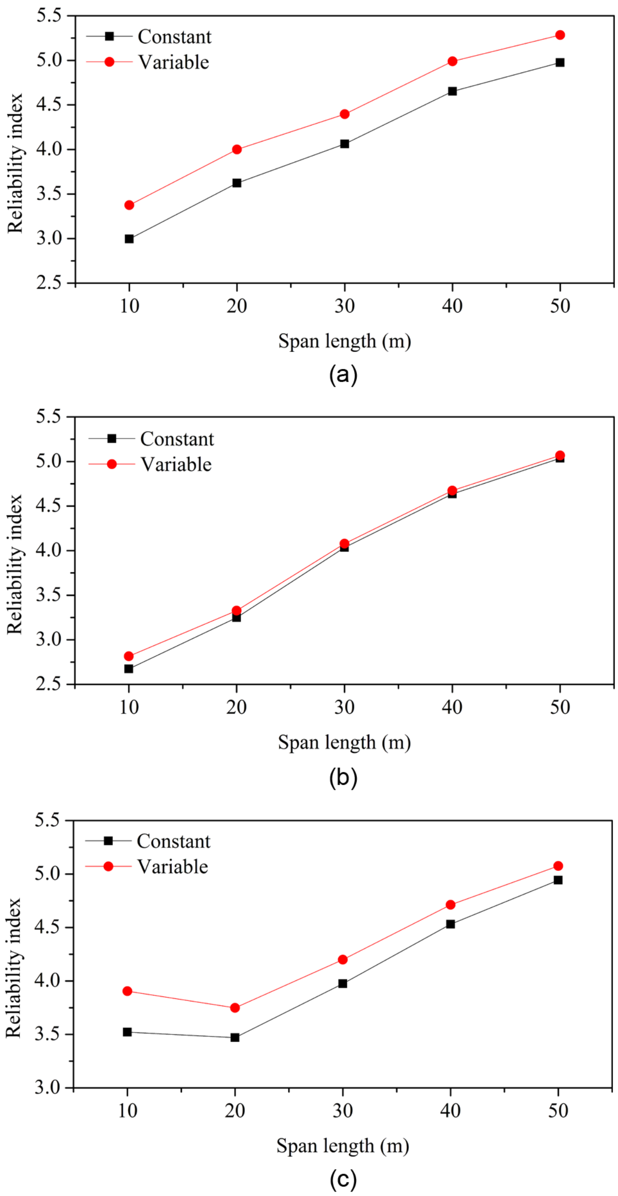

Figure 10. Reliability indexes computed by assuming LDF as a constant and a variable, respectively: (a) girder spacing = 2.2 m, (b) girder spacing = 2.94 m, and (c) girder spacing = 4.0 m.

results were also found in Yousif and Hindi's (2007) study.

All the bridges investigated in this study were simply supported bridges. However, the support conditions for in-service bridges were influenced by many factors, such as the corrosion of bearing, and may gradually change with time. Researchers found that the boundary conditions can have a considerable effect on the LDFs (Bakht and Jaeger, 1988; Eom and Nowak, 2001). Studies (Eom and Nowak, 2001; Harris, 2010) have shown that the actual boundary conditions for simply supported bridges are somewhere between the

Table 3. Statistical properties of LDFs for the bridges with 20 m span and 2.2 m girder spacing.

Boundary conditions	Mean	COV	Distribution type
Hinge–roller	0.379	0.108	Normal
Hinge–hinge	0.497	0.138	Normal

LDF: load distribution factor; COV: coefficient of variation.

Table 4. Reliability indexes for the 20-m span bridges with hinge–hinge supports.

LDF	Girder spacing (m)		
	2.2	2.94	4.0
Constant	4.66	4.35	4.68
Variable	4.99	4.43	4.97

LDF: load distribution factor.

simple support (with free longitudinal displacement) and the hinge–hinge support (horizontal movement is restrained). To investigate whether the change of boundary condition has a significant effect on the bridge reliability index, another scenario within which the bridge is hinge supported at both ends was investigated in this study. Bridges with 20 m span and 2.2 m girder spacing were used for the purpose of illustration. The statistical properties of the LDFs for these bridges are shown in Table 3. It can be observed that the mean values and coefficients of variation of the LDFs of the bridges hinge supported on both ends are larger than those of the simply supported bridges. Table 4 shows the calculated reliability indexes of the hinge–hinge-supported bridges with span length of 20 m by treating the LDF as a constant and a random variable, respectively. It can be seen that for the bridges hinge supported on both ends, treating the LDF as a constant in the reliability analysis still leads to underestimated reliability indexes as compared to using the actual statistical properties of the LDF, which is consistent with the findings for simply supported bridges.

Summary and conclusion

In this study, the statistic properties of the LDFs were obtained by considering the random vehicle transverse positions and their corresponding frequency of occurrence. By treating the LDF as a random variable with the obtained statistic properties, the reliability indexes of bridge girders were evaluated and compared to those predicted using the traditional method which treats the LDF as a constant. Based on the results of this study, it was found that by adopting the single maximum LDF value the traditional bridge evaluation methods may be too conservative and may lead to underestimated bridge reliability indexes under certain circumstances. This suggests that the actual statistical properties of the LDF should be considered when a refined analysis is desirable in case unsatisfactory assessment result is obtained by the traditional evaluation methods. By doing this, unnecessary bridge rehabilitation or demolishment may be avoided.

The findings in this article highlight the importance of considering the actual vehicle transverse position in the evaluation of exiting bridges. In addition, the results from this study also reveal that adopting a constant LDF value or simplifying the bridge structures via two-dimensional beams in the current practice of bridge evaluation may oversimplify the problem and lead to inaccurate bridge reliability indexes.

It should be emphasized again that there are many factors that can lead to the conservativeness of bridge codes, such as the boundary conditions, composite action between the bridge deck and girders, material properties, and so on. This study confirms that the boundary condition has a significant impact on the LDFs obtained. However, this study mainly aims to explore the effect of one important factor, that is, the vehicle transverse loading position, rather than all the influencing factors, on the bridge evaluation results. It should also be noted that the loading truck and transverse distribution of vehicles used in this study are only for the purpose of illustration, site-specific data should be used whenever available in order to obtain more accurate evaluation results of the bridge considered when performing a refined analysis.

Declaration of Conflicting Interests

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