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RELIABILITY ANALYSIS OF REINFORCED CONCRETE SLAB BRIDGES

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ABSTRACT: Empirical expressions for estimating the wheel load distribution and live-load bending moment are typically specified in highway bridge codes such as the AASHTO procedures. The objective of this paper is to assess the reliability levels that are inherent in concrete slab bridges that are designed based on the simplified empirical live load equations in the AASHTO LRFD procedures. To achieve this objective, typical one and two-lane straight bridges with different span lengths were modeled using finite-element analysis (FEA) subjected to HS20 truck loading, tandem loading, and standard lane loading per AASHTO LRFD procedures. The FEA results were compared with the AASHTO LRFD moments in order to quantify the biases that might result from the simplifying assumptions adopted in AASHTO. A reliability analysis was conducted to quantify the reliability index for bridges designed using AASHTO procedures. To reach a consistent level of safety for one lane and two lane bridges, the live load factor in the design equation proposed by AASHTO LRFD needs to be revised by increasing the live load factor to 2.07 for one lane and 1.8 for two lanes. The results will provide structural engineers with more consistent provisions to design concrete slab bridges or evaluate the load-carrying capacity of existing bridges.

Keywords: Concrete Slab Bridges, Finite-Element Analysis, Load-Carrying Capacity, Reliability Analysis.

1. INTRODUCTION

The design of highway bridges in the United States conforms to the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (Specs) or AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications [1]-[2]. The analysis and design of any highway bridge must consider live loads such as HS20 (truck or lane) or HL93 (combination of truck or tandem, and lane loading). To analyze and design reinforced concrete slab bridges, AASHTO specifies a distribution width for live loading that simplifies the two-way bending problem into a beam or one-way bending problem. Empirical expressions for estimating the wheel load distribution and live-load bending moment are typically specified in highway bridge codes such as the AASHTO standards. These equations do not take into account the many factors that govern the actual live load such as the transverse position of a truck or tandem on a specific lane, leading to either over-estimation or under-estimation of the live-load bending moment. The objective of this paper is to assess the AASHTO LRFD code provisions used for calculating the bending moment due to live loads. AASHTO provisions tend to either over-estimate or under-estimate the bending moment due to live loads when compared with the resulting maximum bending moment obtained using finite element analysis. In addition, finite element analyses show that by alternating the position of the truck loads transversely, the resulting bending moments tend to increase as the applied live loads come closer to the transverse edge of a bridge (Mabsout et al., 1997; Mabsout et al., 2004). [3]-[4]

Reliability analysis is an effective tool for developing and assessing new and existing design codes. AASHTO LRFD code was calibrated to create new load and resistance factors to reach a pre-selected safety target based on a reliability analysis using the basic design Eq. (1) (Nowak,1999) [5] :

$$\sum \gamma_i X_i < \phi R_n \qquad (1)$$

Where γ_i represents a set of load factors that are greater than one and that are applied to the different load effects X_i, while ϕ represents a resistance factor that is generally less than one and that is multiplied by the nominal resistance R_n.

In the first step of the analysis conducted in this paper, a finite element analysis was performed to evaluate numerically the maximum bending moments of single span, one and two lane bridges, with different span lengths and various slab thicknesses subjected to AASHTO LRFD live loads. Next, the bending moments were calculated using the simplified AASHTO LRFD provisions. The ratio of the FEA moments to the LRFD moments (α_{LL}) was then quantified for the bridge cases analyzed.

The second step involved defining the statistical characteristics of the different load effects and resistance as per Nowak (1995)[6]. This was followed by a reliability analysis that is aimed at quantifying the reliability levels that are inherent in the traditional LRFD design methodology as per the load and resistance factors that are recommended by AASHTO LRFD. The quantification of the reliability level was accomplished using Monte Carlo simulations whereby the reliability index of the bridge design was evaluated for the different bridges analyzed. The reliability analysis was then repeated while correcting the nominal LRFD live load moments to account for the more representative moments that were obtained from the finite element analysis.

The final step involves proposing modifications to the live load factors of the AASHTO LRFD equation to achieve a target reliability index of 3.5 for all the concrete slab bridges analyzed in this study.

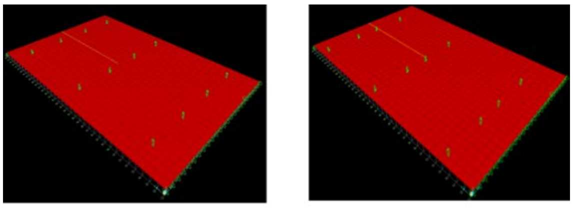
2. FINITE ELEMENT ANALYSIS

The finite element method was used to investigate the effects of live loads on concrete slab bridges. The bridges are modeled as simply supported slabs divided into shell elements. The size of each shell element is taken to be 1 ft. x 1 ft. The span lengths chosen in this study for each bridge are 24, 36, 46, and 52 ft. Lane widths are taken to be 14 ft for a single lane bridge (this takes into account a 1 ft offset on each side) and 24 ft for two-lane bridges. Slab thicknesses are calculated to take into account the deflections.

Live loads are simulated in this analysis as either a combination of HS20 trucks and lane loads, or tandems with lane loads. HS20 loads are taken to be 4 kip point load per tire for the front axles while the middle and rear axles are taken to be 16 kips point load per tire for each axle. The maximum moment developed by the HS20 truck or tandem loads are calculated based on several truck positions. The truck positions are assumed as either centered in each lane or located close to the edge of a lane with a 1 ft of separation distance between the edge of the bridge and the first truck while the separation distance for two side by side trucks are taken to be 4 ft. Figure 1 shows a typical arrangement for HS20 trucks for centered and edge cases, respectively.

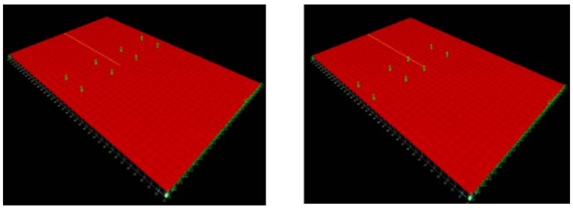
Tandem loads were assumed as 4 point loads with a transverse separation distance of 6 ft. and a longitudinal separation distance of 4 ft. Tandem load positions were assumed to be either centered on each lane, or near the edge of a lane with the same separation distances as the HS20 truck load case. Fig. 2 shows a typical setup for a tandem load for the centered and edge cases, respectively. Lane loads were assumed to be uniform loads centered in each lane with a magnitude of 640 pounds-ft/ft.

Results from the finite element analysis are presented in Table 1 for all the bridge cases analyzed. Results indicate that the maximum moment based on a combination of tandem loads and lane loads governs in short spans (24 and 36 ft) while the maximum moment found from the combined effect of the HS20 truck loads and lane loads governs in the longer spans. Results also show that the moments that were calculated for the edge loading case are generally larger than the moments calculated for a center loading case. This is applicable for the cases of shorter and longer spans, respectively.



(a) Centered Truck Loading (b) Edge Truck Loading

Fig. 1 Typical Concrete Slab Bridge with Truck Loading.



(a) Centered Tandem Loading (b) Edge Tandem Loading

Fig. 2 Typical Two Lane Concrete Slab Bridge with Tandem Loading.

Table 1. FEA Maximum Longitudinal Moment vs AASHTO LRFD Moment

Span length (ft)	Lane	FEA center Kip-ft/ft	FEA edge Kip-ft/ft	LRFD Kip-ft/ft	α_{LL} (FEA/LRFD)
24	One	23.65	24.21	34.95	0.69
36		38.45	39.1	49.44	0.79
46		54.49	56.91	60.68	0.94
54		69.35	72.91	69.84	1.04
24	Two	27.29	29.34	29.97	0.98
36		44.72	47.07	47.85	0.98
46		63.26	67.86	63.19	1.07
54		80.63	86.52	75.82	1.14

To allow for a one to one comparison between the FEA bending moments and the bending moments calculated from the simplified AASHTO LRFD method, the maximum bending moments that were obtained from the simplified method were divided by the equivalent width (E) for each span. The equivalent width could be determined from Eq. (2) and Eq. (3) such that:

$$E = \frac{10 + 5\sqrt{L_1 W_1}}{12} \quad (2)$$

$$E = \frac{84 + 1.44\sqrt{L_1 W_1}}{12} \quad (3)$$

Where:
L_1= span length in ft, the lesser of the actual span or 60 ft and W_1 = edge-to-edge width of bridge in ft taken to be the lesser of the actual width or 60 ft for multi-lane loading, or 30 ft for single-lane loading.

In this study and for the purpose of comparison, it was assumed that the largest value of the maximum longitudinal moment governs the span. The maximum bending moments that were calculated using the simplified AASHTO LRFD procedures are presented in Table 1. These results indicate that for a single lane, the moments calculated using the simplified LRFD procedure for shorter spans deviate from the FEA moments for both the centered and edge loading cases. For the case of one lane, the ratio between the FEA moments and the AASHTO moments (α_{LL}) ranges from 0.67 to 1.04. In the case of two lanes, the two moments tend to be closer to each other with α_{LL} range of 0.98 to 1.14.

3. DESIGN USING AASHTO LRFD

The design of the bridge slab using Eq. (1) (LRFD) requires knowledge about the nominal values of the bending moments due to dead load, live load, and impact load. The nominal bending moment due to dead loads includes the effects of the dead load coming from the slab's own weight (DC) and the weight of the wearing surface above it (DW). To determine the stress due to the own weight of the slab, the thickness of the slab was multiplied by the unit weight of concrete (0.145 kcf as per AASHTO LRFD, table 3.5.1-1). Similarly, the stress due to the wearing surface was calculated as the product of the thickness (0.25 ft) and the unit weight of 0.14 kcf. The nominal bending moment due to the components of the dead load was then determined based on the simply supported moment equation. Table 2 shows a summary of the bending moments determined based on the different dead load components.

The total nominal maximum live load moment (M_{LL+IL}) was calculated as the summation of the static live load moment (M_{LL}) and the dynamic/impact live load (M_{IL}). AASHTO LRFD defines the ratio of the dynamic load allowance as 33% of the static moment of the truck or tandem components of the static live load (M_{LL}) only. To calculate the impact load, the contribution of the truck/tandem load to the static live load was isolated and multiplied by a factor of 0.33. Table 3 shows the values of the static, impact, and total nominal live load moments.

Given the nominal dead load and live load moments, the AASHTO LRFD design Eq. (1) can be applied to calculate the nominal moment resistance (R_n) for each bridge such that:

$$R_n = \frac{(1.25M_{DC} + 1.5M_{DW} + 1.75(M_{LL} + M_{IL})}{0.9} \quad (4)$$

Table 2. Dead Load Moments due to Concrete (DC) and Wearing (WC)

Span length (ft)	Lane	Slab thickness (ft)	Moment DC (kip-ft/ft)	Wearing surface thickness(ft)	Moment DW (kip-ft/ft)
24	One lane	1.5	15.66	0.25	2.52
36		1.75	41.11	0.25	5.67
46		2	76.71	0.25	9.26
54		2.25	118.92	0.25	12.76
24	Two lanes	1.5	15.66	0.25	2.52
36		1.75	41.11	0.25	5.67
46		2	76.71	0.25	9.26
54		2.25	118.92	0.25	12.76

Table 3. Static (M_{LL}) and Dynamic (M_{IL}) Nominal Live Load Moments

Span length (ft)	Lane	Width (ft)	Governing M_{LL} source	M_{LL} (kip-ft/ft)	M_{IL} (kip-ft/ft)	$M_{(LL+IL)}$ (kip-ft/ft)
24	One lane	14	Tandem	34.95	9.84	44.79
36		14	Tandem	49.44	13.13	62.57
46		14	Truck	60.68	18.93	79.61
54		14	Truck	69.84	18.77	88.61
24	Two lanes	24	Tandem	29.97	7.69	37.66
36		24	Tandem	47.85	10.23	58.08
46		24	Truck	63.19	14.71	77.90
54		24	Truck	75.82	14.57	90.39

4. RELIABILITY ANALYSIS

Monte Carlo simulations were utilized to conduct a reliability analysis for concrete slab bridges that are designed based on the AASHTO LRFD design equation. Failure was defined using the performance function shown in Eq. (5).

$$g = R - DC + DW + (LL + IL) \quad (5)$$

Where R, DC, DW, and (LL+IL) were assumed to be random variables. The probability of failure (P_f) was determined from the Monte Carlo simulations by counting the realizations with ($g < 0$) and dividing them by the total number of simulations (1,000,000 simulations). The reliability index β, which is a measure of structural safety, was then calculated as:

$$\beta = -\phi^{-1}(P_f) \quad (6)$$

Where Φ^{-1} constitutes the inverse of the standard normal cumulative distribution function. The probabilistic models and the statistical parameters (mean and standard deviation) describing the uncertainty in the different design variables are discussed in the following sections.

4.1 Statistical Load and Capacity Models

The statistical parameters (bias λ and coefficient of variation V) for the bending moments due to slab weight and wearing surface were adopted from (Nowak, 1995) as λ_{DC} = 1.05 and V_{DC} = 0.1 and λ_{DW} = 1.0 and V_{DW} = 0.25, respectively. The bias factor is defined as the ratio of the mean of a given parameter to the nominal value of that parameter. As a result, the mean values for DC and DW for all bridges considered can be defined from λ_{DC} and λ_{DW} together with the nominal values shown in Table 2.

The bias factors λ_{LL} of the static live load moments (M_{LL}) were presented by Nowak (1995)[6] and are dependent on the number of lanes and span lengths, while the coefficient of variation of (M_{LL}) has been found to be constant with a value of 0.12. Table 4 shows the bias factor of the live load moment for each span and the corresponding mean values of the total live load.

Table 4. Live Load Statistical Parameters

No. of lanes	Span (ft)	λ_{LL}	$V(M_{LL})$	Mean of $M_{(LL+IL)}$	Standard Deviation $M_{(LL+IL)}$	V $M_{(LL+IL)}$
1	24	1.38	0.12	61.80	9.77	0.158
	36	1.39	0.12	86.97	13.36	0.154
	46	1.37	0.12	109.01	18.14	0.166
	54	1.36	0.12	120.5	18.85	0.156
2	24	1.16	0.12	43.68	7.43	0.170
	36	1.19	0.12	69.11	10.66	0.154
	46	1.19	0.12	92.70	14.83	0.160
	54	1.18	0.12	106.65	15.85	0.149

The mean and standard deviation of the total live load $M_{(LL+IL)}$ are determined according to Eq. (7) and Eq. (8) by combining the statistics of the static live load and the impact load. In Eq. (8), the values of 0.12 and 0.8 represent the coefficients of variation of the static live load and the dynamic impact load, respectively. The resulting means and standard deviations of the total live load for the cases analyzed in this study are presented in Table 4 together with the corresponding estimates of the coefficient of variation of the $M_{(LL+IL)}$ [7].

$$\text{Mean of } M_{(LL+IL)} = \lambda_{LL} * M_{(LL+IL)_{nominal}} \quad (7)$$

$$\text{St. Dev. } M_{(LL+IL)} = \sqrt{(0.12\lambda_{LL} M_{LL})^2 + (0.8 M_{IL})^2} \quad (8)$$

Finally the statistical parameters for the moment capacity for reinforced concrete slab bridges were adopted from Kulicki et al. (2007) [7] based on a bias factor λ_R of 1.14 and a V_R of 0.13.

In the reliability analysis, the moments due to slab weight, wearing surface, and total live load were assumed to be normally distributed as per the recommendations of Kulicki et al. (2007)[7]. Along the same lines, the moment capacity was taken to be lognormally distributed.

4.2 Results of the Reliability Analysis

The first set of reliability analyses were conducted to assess the reliability levels that are inherent in concrete slab bridges that are designed in accordance with the current LRFD design equation which is based on a live load factor of 1.75. The results of this set of analyses are presented in Fig. 3a and indicate that the reliability index β ranges from 2.6 to 3.0 for the cases involving single lane bridges and is slightly below 3.5 for the cases involving two-lane slab bridges. The results of the single lane concrete bridges reflect reliability levels that fall short of the target reliability index of 3.5 that was set by AASHTO LRFD[6]. On the other hand, the results of the two-lane bridges are closer to the target reliability level.

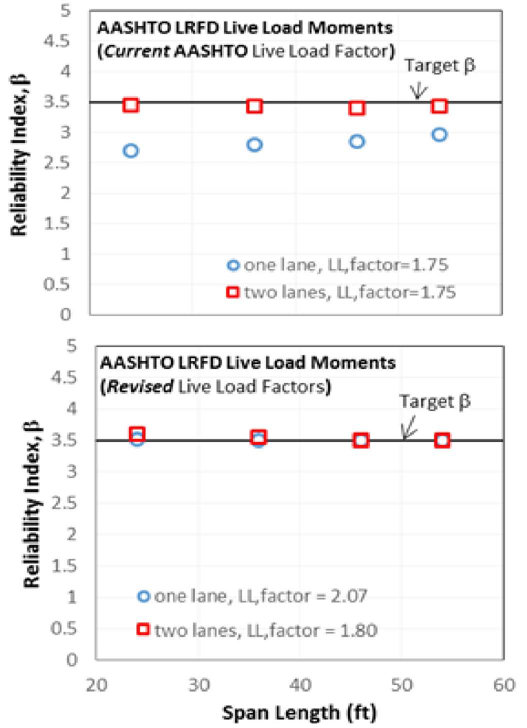


Fig. 3 Reliability Indices Using Simplified AASHTO LRFD Moments for (a) AASHTO Live Load Factors and (b) Revised Live Load Factors.

Results on Fig. 3b point to the need for revising the AASHTO LRFD live load factors if a reliability index as high as 3.5 is to be targeted. This is particularly important for the case involving single lane bridges. As a result, the reliability analysis was repeated assuming different live load factors in an attempt to identify the factors that would ensure the desired level of reliability in the design. Results indicated that for single lane bridges, a live load factor that is as high as 2.07 is required to ensure that bridges with all span lengths would achieve a target reliability index of 3.5. For the two lane loading case, the LRFD load factor needs to be increased slightly from 1.75 to 1.8 to achieve the target reliability level. The revised LRFD design equations for the single and double lane scenarios are presented in Eq. (9) and Eq. (10), respectively. The resulting reliability levels for the revised cases are presented in Fig. 3b.

For single lane bridges:

$$\phi R_n = 1.25M_{DC} + 1.5M_{DW} + 2.07(M_{LL} + M_{IL}) \quad (9)$$

For two lane bridges:

$$\phi R_n = 1.25M_{DC} + 1.5M_{DW} + 1.8(M_{LL} + M_{IL}) \quad (10)$$

The results presented in Fig. 3 pertain to bridge designs that are based on the live load moments M_{LL} that are calculated using the simplified procedure recommended by AASHTO LRFD. Results presented in Table 1 indicate that these M_{LL} values could deviate from the more representative finite element values, particularly for cases involving single lane bridges with relatively shorter spans (24ft and 36ft). To account for this discrepancy in the value of M_{LL}, the reliability analyses were repeated such that the simplified AASHTO live load moments were corrected by multiplying these moments by the ratio α_{LL} (see Table 1). For the cases involving single-lane bridges with shorter spans (24, 36 ft) where the tandem/lane combination governed, an α_{LL} value of 0.74 (average of the two α_{LL} values for the two span lengths) was adopted. For a single lane with longer spans (46, 54 ft) where the HS-20 truck/lane combination governed, an α_{LL} value of 0.99 was adopted. For the two lane bridge cases, the α_{LL} ratios of 0.98 and 1.11 were adopted for the shorter and longer spans, respectively. To incorporate the ratio α_{LL} in the reliability analysis, the static live load moment that is based on the simplified AASHTO LRFD procedure was multiplied by α_{LL} as reflected in the modified performance function in Eq. (11):

$$g = R - (DC + DW + \alpha_{LL}(LL) + IL) \quad (11)$$

The results of the reliability analysis that was conducted using the revised performance function that is presented in Eq. (11) are shown in Fig. 4. Results pertain to the conventional live load factor of 1.75 that is recommended by AASHTO LRFD. As expected, the calculated reliability indices for the single lane bridges with the shorter span lengths of 24 and 36 ft increased significantly compared with the earlier results (Fig. 3a). This increase in the reliability index (up to values of 3.8) is directly correlated to the smaller α_{LL} ratio (average of 0.74) which indicates that the simplified AASHTO LRFD procedure overestimated the maximum live load moments on the bridge. For the single lane bridges with the longer spans, the reliability indices were found to be still less than the target reliability index since the α_{LL} ratio for these cases was close to 1.0.

For the two lane bridges, results in Fig. 4a indicate that the target reliability index was achieved for the shorter spans, but fell short of achieving a target reliability index of 3.5 for the longer spans for the case where the conventional AASHTO LRFD load factor of 1.75 was adopted.

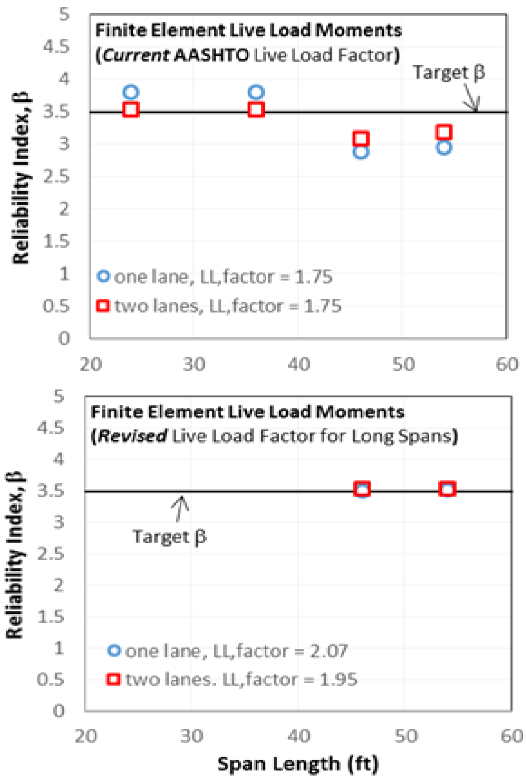


Fig. 4 Reliability Indices for Cases where the AASHTO LRFD Live Load Moments are Corrected using the FEA Results for (a) AASHTO Live Load Factors and (b) Revised Live Load Factors.

To ensure a target reliability index of 3.5 for the longer spans, the LRFD live load factors need to be revised for the single lane and the two-lane bridge cases. Results from the reliability analysis indicated that for the one-lane case with longer spans, the live load factor has to be increased from 1.75 to 2.07, even if the live load moments are corrected based on the FEA results. As for the two lane bridge cases, the target reliability levels for the longer spans cases could be ensured with a revised live load factor of 1.95 as shown in Fig. 4b. Thus, it is recommended that Eq. (12) and Eq. (13) be used in the design of single and two lane reinforced concrete bridges with longer spans, respectively.

For single lane bridges:

$$\phi R_n = 1.25 M_{DC} + 1.5 M_{DW} + 2.07(M_{LL} + M_{IL}) \quad (12)$$

For two lane bridges:

$$\phi R_n = 1.25 M_{DC} + 1.5 M_{DW} + 1.95(M_{LL} + M_{IL}) \quad (13)$$

5. SUMMARY AND CONCLUSIONS

The method used to calculate the bending moment in AASHTO LRFD tends to overestimate the live load moments for shorter spans in one and two lane bridges when compared to the moment obtained from the finite element analysis. For longer spans, the bending moment obtained from AASHTO LRFD provisions tends to slightly underestimate the moment when compared with the FEA moment for one and two lane reinforced concrete bridges.

The reliability analysis performed in this study is used to check the level of safety for the reinforced concrete bridges that are designed with the AASHTO LRFD provisions. The results of the reliability analysis showed that the reliability index is slightly lower than the target reliability index for two lane bridges. The reliability indices for one lane reinforced concrete bridges were considerably lower than the target reliability index. To reach a consistent level of safety for one lane and two lane bridges, the live load factor in the design equation proposed by AASHTO LRFD needs to be revised by increasing the live load factor to 2.07 for one lane and 1.8 for two lanes.

When the difference between the moments obtained from AASHTO LRFD and FEA is incorporated in the reliability analysis, the results showed acceptable target reliability levels for shorter span bridges and relatively inferior reliability indices for longer spans. To achieve the target reliability levels for these cases, the load factors in the AASHTO LRFD provisions needed to be increased to 2.07 for a single lane with longer spans and to 1.95 for two lanes with longer spans.

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