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INFLUENCE OF RAILING STIFFNESS ON WHEEL LOAD DISTRIBUTION IN TWO-SPAN CONCRETE SLAB BRIDGES

SARAH JABER[1], MOUNIR MABSOUT[1], and KASSIM TARHINI[2]

[1]*Dept of Civil and Environmental Engineering, Amer. Univ. of Beirut, Beirut, Lebanon*
[2]*Dept of Civil Engineering U. S. Coast Guard Academy, New London, USA*

The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications or LRFD do not account for the presence of railings in the analysis and design of concrete slab bridges. This paper presents a parametric investigation of the influence of railing stiffness on the wheel load distribution in simply-supported, two-equal-span, and one- and two-lane reinforced concrete slab bridges using the finite-element analysis (FEA). A total of 160 bridge cases were modeled and bridge parameters such as span lengths and slab widths were varied within practical ranges. Various railing stiffness were investigated by assuming railings built integrally with the bridge deck and placed on both edges of the bridge. The FEA wheel load distribution and longitudinal bending moments were compared with reference bridge slabs without railings as well as to the AASHTO design procedures. Accordingly, the presence of railings reduced the FEA negative moments by a range of 54% to 72% and the FEA positive moments by a range of 40% to 61% depending on the railing stiffness. This reduction in slab moments due to the presence of railings could be considered an increase in the bridges load carrying capacity. The results of this investigation will assist bridge engineers in better designing and/or evaluating concrete slab bridges in the presence of railings. This could also be considered an alternative for strengthening existing concrete slab bridges.

Keywords: Concrete slab bridges, Railing stiffness, Finite-element analysis, AASHTO procedures, Load-carrying capacity.

1 INTRODUCTION

A significant number of highway bridges are short-span reinforced concrete slabs that are owned and maintained by local and state governments. The design of highway bridges in the United States conforms to the American Association of State Highway and Transportation Officials (AASHTO) Standard (2002) or Load and Resistance Factor Design (LRFD) Bridge Design Specifications (2012). The current AASHTO procedures do not consider the effect of railings that are built integrally with bridge deck in the analysis and design or evaluating the load-carrying capacity of concrete slab bridges.

A published parametric study by Mabsout *et al.* (2004) investigated single-span, simply-supported reinforced concrete slab bridges using finite-element analysis (FEA). Results indicated that AASHTO Standard Specifications moments overestimate the FEA moments for short spans and one-lane bridges but agreed with FEA moments for short-span bridges with two or more lanes. It was also found that AASHTO Standard Specifications (2002) underestimated the FEA moments for longer spans. However, for AASHTO LRFD (2012) procedure, it overestimated the FEA moments for all bridge cases. In addition, several published studies were conducted by investigating the influence of sidewalks and railings on wheel load distribution in steel and prestressed girder bridges which was shown to increase the stiffness of the superstructure and increase the load-carrying capacity of these bridges (Akinci *et al.* 2008, Chung *et al.* 2006, Conner and Huo 2006, Eamon and Nowak 2002, Mabsout *et al.* 1997). Recently, a parametric investigation by Fawaz *et al.* (2017) studying the influence of one standard railing size on straight concrete slab bridges was performed. The results indicated that by placing two railings on straight bridges, AASHTO Standard Specifications (2002) procedures overestimated the FEA moments by about 100% for one-lane bridges, and by about 20% for bridges with two-lanes. AASHTO LRFD (2012) overestimated the FEA moments in all bridge cases by about 150% for one-lane, and about 70% for two-lanes when placing two railings on slab bridges. This paper builds on the recent study to investigate the influence of railings stiffness on the increase in load carrying capacity in two-span continuous reinforced concrete slab bridges.

2 AASHTO BENDING MOMENTS

For simply-supported concrete slab bridges, AASHTO Standard Specifications (2002) suggest three approaches in determining the live-load bending moment but only one procedure is used in this study that was compared with the finite-element analysis results.

$$M = 13{,}500S \;\; for\ S \le 15m \tag{1}$$

or

$$M = 1{,}000(19.5S - 90) \;\; forS > 15m \tag{2}$$

where S is the span length (m) and M is the longitudinal moment per unit width (N-m/m).

AASHTO LRFD Section 4.6.2.3 (2012) provides an equivalent strip width procedure to design reinforced concrete slab bridges that is comparable to procedures specified in the Standard Specifications. However, the AASHTO LRFD Section 3.6.1.2 requires the use of HL93 live loading (addition of HS20 Truck plus lane loading). This approach is to divide the total bending moment from a line of wheel and lane load by an equivalent width defined by AASHTO LRFD (2012) to obtain a statically design moment per unit width.

3 DESCRIPTION OF BRIDGE CASES

Typical simply-supported two-equal-spans, one- and two-lane reinforced concrete slab bridge cases were analyzed in this investigation. Four span lengths were considered in this parametric study: 7.2, 10.8, 13.8, and 16.2 m (24, 36, 46, and 54 ft) with corresponding slab thicknesses of 450, 525, 600, and 675 mm (18, 21, 24, and 27 inches), respectively, to control deflection. The overall slab widths were assumed to be 4.2 m (14 ft) for one lane and 7.2 m (24 ft) for two lanes.

The base case for the standard railings size was adopted from previous research by assuming 200 mm (8 in) wide and 760 mm (30 in) high railing above slab, as reported by Fawaz *et al.* (2017). Five stiffness factors are considered including X0, X1, X2, X3, X4, and X0.5, assuming X0 as the reference case with no railings and X1 to be the base case with the standard railings. The railing moment of inertia (I) was computed at the bottom of the railing section as per Eq. (3).

$$I_{(bottom)} = I_{(center)} + Ad^2 = \frac{bh^3}{12} + bh(\frac{h}{2})^2 = \frac{bh^3}{3} \qquad \therefore I_{(bottom)} = 4I_{(center)} \tag{3}$$

Therefore, the railings stiffnesses considered have the following moment of inertia with X0 (I = 0), X1 (I = 4Ic), X2 (I = 8Ic), X3 (I = 12Ic), X4 (I = 16Ic), and X0.5 (I = 2Ic). The various railings sizes corresponding to (X0.5 – X4) are shown in Figure 1. Also, Figure 2 (right) shows typical plan views and cross-section of 10.8 m (36 ft) per each span, two-lane bridge case with railings (base case, X1), with HS20 trucks loading placed longitudinally and transversely to cause maximum longitudinal positive moments, respectively, as explained in the following section.

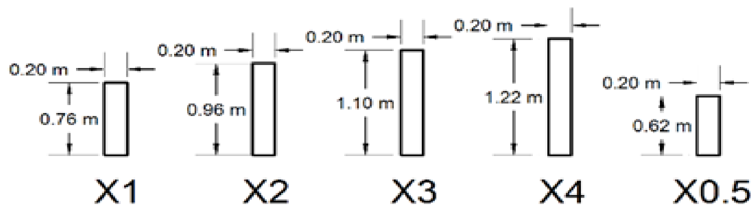

Figure 1. Various railing sizes corresponding to stiffnesses (X1, X2, X3, X4, X0.5).

4 BRIDGE LOADING

The bridge cases considered in this study were subjected to AASHTO HS20 standard design trucks. The AASHTO HS20 truck, with two line of wheel spaced at 6 ft (1.8 m) consisting of loads of 4, 16, 16 Kips spaced at 14 ft (or 18, 72, 72 KN spaced at 4.2 m), were placed longitudinally on the bridge deck to produce maximum positive or negative bending moments in the slab. Transversally, as per Fawaz *et al.* (2017), Edge loading condition was adopted in this study, which consists of positioning the center of the left wheel of the left most truck at 0.3 m (1 ft) from the left edge of the slab, and the other trucks placed side-by-side with a distance 1.2 m (4 ft) between the adjacent trucks. Figure 2 (left) illustrates the Edge loading conditions for the 10.8 m (36 ft) span, two-lane bridge cases for maximum positive moments.

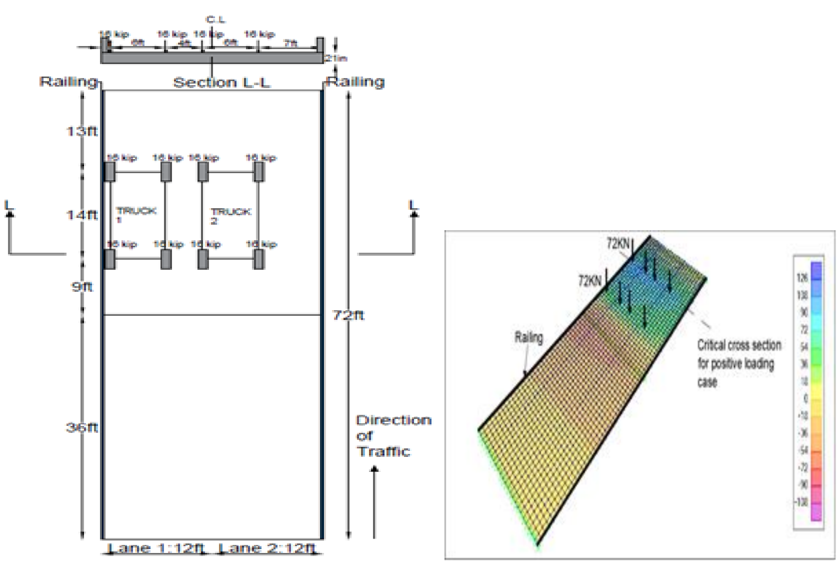

Figure 2. (Left) Typical plan view and cross-section for a 10.8 m (36 ft) per span, two-lane bridge, subjected to positive Edge loading condition with base case railings (X1); and (Right) FEA corresponding longitudinal positive bending moments (KN-m/m).

5 FINITE ELEMENT MODELING

A total of 160 concrete slab bridge cases were investigated in this study using the FEA. The computer program SAP2000 (version 19) was used to discretize the bridge into four-node shell elements with six degrees of freedom at each node. The simple eccentric beam element was adopted in this investigation to model the railings, as per Fawaz *et al.* (2017). Figure 2 (right) illustrates typical finite element models with the corresponding positive moment contours, for a 10.8 m (36 ft) span, two-lane bridges, subject to HS20 Edge loading condition, and in the presence of two standard base case railings.

6 FINITE-ELEMENT ANALYSIS RESULTS

The FEA results are reported in terms of the maximum longitudinal bending moments at critical section in the concrete slab bridges. The FEA results for bridges with railings of different stiffness factors were compared with reference bridge cases without railings, and with AASHTO Standard Specifications (2002) as per Eqs. (1) or (2), and with LRFD procedures (2012).

6.1 FEA Results vs. AASHTO

Figure 3 (left) shows sample plots of the FEA longitudinal positive moment plots at the critical sections for all the two-lane bridge cases in combination with the four span lengths (S) with base case railings (X1). Figure 3 (right) shows the positive moment plots for all the two-lane bridges with 10.8 m (36 ft) span length, with different railing stiffness configurations, along with the AASHTO Standard Specifications (2002) and LRFD procedures (2012) moments. The maximum FEA moments for the concrete slabs were defined as the first peak value occurring after the maximum value at the leftmost edge; the latter "edge" moment is assumed to be resisted by an edge beam. Similar figures were obtained for the negative moments**.**

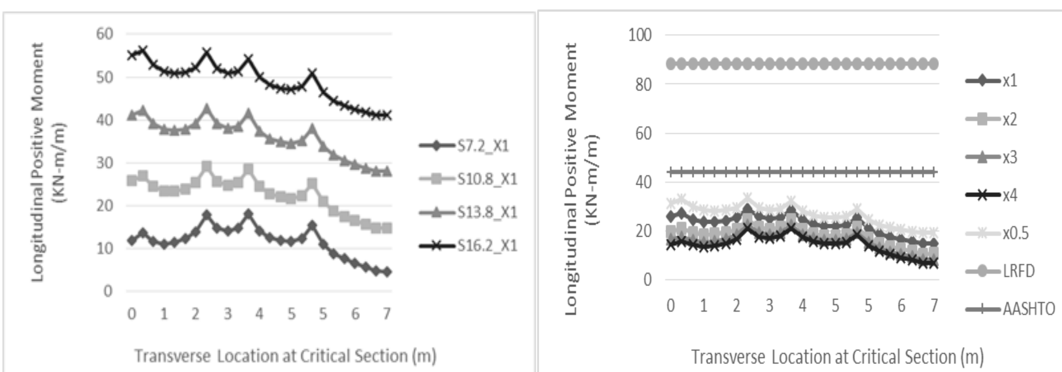

Figure 3. FEA positive moments for two-lane bridges with base case railings (X1) for various spans (left); and for a 10.8 m (36 ft) span with various railings sizes compared with AASHTO moments (right).

The maximum FEA positive and negative moments were summarized and compared with the AASHTO Standard Specifications (2002) and LRFD (2012) moments for all the bridge cases. It was observed that, for bridge cases with no railings (X0), AASHTO Standard Specifications (2002) generally tends to give similar results to the FEA negative moments, with the exception of one-lane with spans less than 12 m (40 ft) where the AASHTO overestimates the FEA negative moments by about 30%. This is more pronounced with more lanes and longer spans, where AASHTO underestimation of the FEA moments reaches up to 26% for two lanes with spans greater than 12 m (40 ft). When base case railings (X1) are present in a concrete slab, the FEA slab negative moments decrease significantly and AASHTO overestimates or gives similar moments in almost all cases, reaching about 68% for the one-lane bridges with spans less than 12 m (40 ft), and gives similar moments for two-lane bridges with spans longer than 12 m (40 ft). Also, as the stiffness factor of railings increases, the FEA negative moments decrease and a more significant AASHTO overestimation is observed. This overestimation reaches about 81% for one-lane bridges and 72% for two-lane bridges with (X4) stiffer railings. It can be observed that, for bridge cases with no railings (X0), AASHTO Standard Specifications generally tends to overestimate the results of the FEA slab positive moments, with the AASHTO overestimation reaching about 35%. As the number of lanes and span length increase, this overestimation decreases. When base case railings (X1) are present in a concrete slab, the FEA slab positive moments decrease significantly and AASHTO overestimates moments in all cases, reaching about 55% for the one-lane bridges with spans less than 12 m (40 ft), and this overestimation decrease to about 18% for two-lane bridges with spans longer than 12 m (40 ft). Also, as the stiffness factor of railings increases, the FEA positive moments decrease and a more significant AASHTO overestimation is observed. This overestimation reaches about 70% for one-lane bridges and 53% for two-lane bridges with (X4) stiffer railings.

AASHTO LRFD (2012) overestimates the FEA slab negative moments and positive moments in almost all bridge cases with or without railings. AASHTO LRFD overestimates the FEA slab negative moments by about 30% and positive moments by about 40% for one-lane bridges. For two-lane bridges, this overestimation increases to about 48% for the negative moment and about 55% for the positive moment. This overestimation decreases with the increase in span length. When base case railings (X1) are present, the AASHTO LRFD overestimation of the FEA slab moments becomes more significant reaching an average high of 66% for the negative moment and 61% for the positive moment in one-lane bridges, and 67% for the negative moment and 70% for the positive moment in two-lane bridges. This overestimation is further increased as the railings stiffness factor increases where it reaches about 81% for the negative moment and 74% for the positive moment for one-lane bridges and about 82% for the negative moment and 77% for the positive moment for two-lane bridges with (X4) stiffer railings.

Table 1. Comparison of FEA positive moment with railings to reference case without railings.

Number of Lanes	Span Length (m)	Ratio of FEA Maximum Longitudinal Moment with Railings to Reference Case without Railings						Reference Moment X0
		Stiffness Factor						
		X0		X1		X2		
1	7.2	19.1	1.00	15.6	0.82	14.0	0.73	19.1
	10.8	32.9	1.00	19.8	0.60	15.9	0.48	32.9
	13.8	46.6	1.00	30.1	0.65	23.7	0.51	46.6
	16.2	57.8	1.00	40.5	0.70	32.4	0.56	57.8
2	7.2	23.7	1.00	18.0	0.76	16.9	0.71	23.7
	10.8	40.5	1.00	29.2	0.72	24.7	0.61	40.5
	13.8	56.4	1.00	42.7	0.76	35.8	0.63	56.4
	16.2	69.4	1.00	55.7	0.80	47.5	0.68	69.4
		X3		X4		X0.5		
1	7.2	12.9	0.68	12.3	0.64	17.1	0.90	19.1
	10.8	14.1	0.43	13.0	0.40	23.9	0.73	32.9
	13.8	20.2	0.43	18.0	0.39	36.1	0.77	46.6
	16.2	27.5	0.48	24.4	0.42	47.3	0.82	57.8
2	7.2	16.3	0.69	16.1	0.68	19.4	0.82	23.7
	10.8	22.2	0.55	20.7	0.51	33.2	0.82	40.5
	13.8	31.6	0.56	28.7	0.51	48.1	0.85	56.4
	16.2	42.0	0.61	38.1	0.55	61.6	0.89	69.4

6.2 FEA Results: Railings vs. No Railings

The maximum slab positive bending moments are summarized in Table 1 for all bridge cases in terms of ratios of FEA results for cases with various railings stiffness factors as compared to the reference cases X0 without railings; a similar table was obtained for negative moments. It can be observed that the presence of railings reduces the maximum moments, and this becomes significant as the railings stiffness factor increases, and more for one lane vs. two lanes. For one-lane bridges, the maximum longitudinal moment was reduced by about 40 to 55% when adding railing with stiffness factor (X1) and reduced by about 60 to 70% with (X4) railing stiffness factor. As for two-lane-bridges, the slab moment was reduced by about 30 to 40% with (X1) railing stiffness factor, and a higher reduction was observed for (X4) stiffer railing of about 50 to 65%.

7 SUMMARY AND CONCLUSIONS

AASHTO Standard Specifications (2002) and AASHTO LRFD (2012) procedures do not account for the presence of railings as integral parts of a bridge slab. Based on this study, it is evident that these integral elements increase the capacity of the concrete slab bridges. This reduction in the slab moment decreases with the increase in the number of lanes and increases with the change in the railing stiffness. These railings can be used as an alternative strengthening technique to upgrade or rehabilitate existing bridges and allow permit vehicles on the bridge.

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