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INFLUENCE OF RAILINGS STIFFNESS ON WHEEL LOAD DISTRIBUTION IN THREE- AND FOUR-LANE CONCRETE SLAB BRIDGES

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ABSTRACT: The American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO LRFD) do not account for the presence of railings in the analysis or design of highway bridges. This paper presents a follow-up parametric investigation of the influence of railing stiffness on the wheel load distribution in simply-supported, one-span, three- and four-lane reinforced concrete slab bridges using the finite-element analysis (FEA). A total of 48 bridge cases are modeled using refined 3D FEA and bridge parameters such as span lengths, slab widths, and railings that were varied within practical ranges. Various railings stiffness were considered to be built integrally with the bridge deck and placed on both edges of the concrete slabs. The FEA wheel load distribution and bending moments are compared with reference bridge slabs without railings as well as to the AASHTO design procedures. According to the FEA results, the presence of railings reduces the longitudinal bending moment in the concrete slabs by 25% to 60% depending on the stiffness of the railings in one- and two-lane reinforced concrete bridges. However, when considering three- and four-lane bridges, the presence of railings reduced the longitudinal bending moment in the concrete slab by a range of 10% to 32% depending on the stiffness of railings. The results of this investigation will assist bridge engineers in better evaluating the load carrying capacities of multi-lane concrete slab bridges using 3D FEA and account for the contribution of railings. The presence of railings can also be considered a possible alternative for strengthening existing concrete slab bridges.

Keywords: Concrete Slab Bridges, Multi-lane, Railings Stiffness, Finite-Element Analysis, AASHTO Procedures, Load-Carrying Capacity.

1. INTRODUCTION

According to the U.S.A. Federal Highway Administration’s 2016 National Bridge Inventory data, approximately 22% of the nation's 603,620 bridges are structurally deficient or functionally obsolete as reported in Better Roads Magazine (November 2016). Highway Bridges that are either built using cast-in-place concrete or precast concrete panels form about 69% of all bridges (423,216). Single span reinforced concrete bridges represent about 150,000 and assuming 22% (33,000) of those bridges to be structurally deficient or functionally obsolete. It is also known that the average bridge age in the U.S.A. is 43 years old while most of the bridges were designed for a lifespan of 50 years. Therefore, an increasing number of bridges will soon need either major rehabilitation or replacement. However, this bridge inventory accounts for structures with span lengths greater than 6 m (20 ft), where the majority of the structurally deficient bridges are short spans, averaging less than 15 m (50 ft) in length. These deficient bridges are being recommended for weight-limit posting, rehabilitation, or replacement. Thousands of such structures especially with span length are less than 6 m (20 ft) in every state and municipality may be ignored, not inspected, or not replaced on regular basis due to the lack of funding. This task is left up to each local government to maintain such structures that span less than 6 m (20 ft) without federal support.

Therefore, a significant number of highway bridges are short-span reinforced concrete slabs that are owned and maintained by local and state governments. The main advantage of concrete slab bridges is the ease of construction and the ability to field adjustment of the roadway profile during construction. The design of highway bridges in the United States conforms to the American Association of State Highway and Transportation Officials (AASHTO) Standard [1] or Load and Resistance Factor Design (LRFD) Bridge Design Specifications [2]. The current AASHTO procedures do not consider the effect of railings that are built integrally with bridge deck in the evaluation of the load-carrying capacity of bridges. Therefore, this study investigates the effect of railings and its stiffness in resisting highway loading and by increasing the load-carrying capacity of reinforced concrete slab bridges.

A parametric study investigated straight, single-span, simply-supported reinforced concrete slab bridges using finite-element analysis (FEA) [10]. Results indicated that AASHTO Specifications overestimate the FEA bending moments for short spans, one-lane bridges and agreed with FEA moments for short spans in a combination of two or more lanes. In addition, AASHTO Standard Specifications underestimates the FEA moments for longer spans. As for AASHTO LRFD procedure, it overestimates the FEA bending moments for all bridge cases. Several studies were conducted and reported in the literature that investigated 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 improve the load-carrying capacity of these bridges [4]–[5]–[6]–[7]–[9].

Recently, a published investigation studying the influence of one standard railings size on concrete slab bridges was performed [8]. The results indicated that placing two railings on straight concrete slab bridges, AASHTO Standard Specifications procedures overestimated the FEA moments by 100% for one-lane bridges, and by 20% for bridges with two or more lanes. AASHTO LRFD overestimated the FEA moments in all bridge cases by 150% for one-lane, 70% for two-lanes, and a 30% for three- and four-lanes when placing two railings on slab bridges. It is worth noting that the AASHTO procedures, which overestimated the FEA moments, did not consider the presence of side railings and the effect of their stiffness. Another study investigated the influence of railing stiffness in one- and two-lane reinforced concrete slab bridges [3]. The study showed that the presence of railings reduces the longitudinal bending moment in slabs or increase the load carrying capacity by a range of 25% to 60% depending on the stiffness of the railings.

This paper builds on the 3D FEA investigation studying the influence of railing stiffness in one- and two-lane bridges reported in [3]. The paper presents the results of a parametric study investigating the influence of railings stiffness in simply supported one-span, three- and four-lane bridges and how it will increase the load-carrying capacity of 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.

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

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

Where:

S = span length (m)

M = longitudinal bending moment per unit width (N-m/m)

AASHTO LRFD Section 4.6.2.3 (2014) 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 (addition of HS20 Truck plus lane loading) live loading. This approach is to divide the total bending moment by an equivalent width to obtain a statically design moment per unit width. The equivalent width "E" of longitudinal strips per lane for both shear and moment is determined using the following formulas:

The width for one lane loaded is

$$E = 250 + 0.42\sqrt{L1 \times W1} \tag{3}$$

while the width for multi-lanes loaded is:

$$E = 2{,}100 + 0.12\sqrt{L1 \times W1} \tag{4}$$

Where:

E = equivalent width of longitudinal strips per lane, "mm"

L1 = span length in "mm", the lesser of the actual span or 18,000 mm

W1 = edge-to-edge width of the bridge in "mm" taken to be the lesser of the actual width or 18,000 mm for multi-lane loading, or 9,000 mm for single-lane loading.

The AASHTO Standard Specifications and AASHTO LRFD equations above do not take into account the influence of side railing when analyzing or designing concrete slab bridges. These equations will be used in calculating the AASHTO bending moments that will be compared with 3D FEA results.

3. DESCRIPTION OF BRIDGE CASES

Building on the published investigations [3]-[8], typical simply-supported one-span, three-lane, and four-lane reinforced concrete slab bridge cases were selected and 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. The concrete slab thicknesses were calculated using the AASHTO equations to control deflection. The overall slab widths were assumed to be: 10.8 m (36 ft) for three-lanes, and 14.4 m (48 ft) for four-lanes.

The base case for the standard railings size adopted from previous research was 200 mm (8 in) wide and 760 mm (30 in) high above slab [8]. Another parameter considered in this study was varying the railings stiffness, which is represented by the moment of inertia of the railing (I) computed at the bottom of the railing section.

$$I_{(bottom)} = I_{(center)} + Ad^2 = \frac{bh^3}{12} + bh\left(\frac{h}{2}\right)^2 = \frac{bh^3}{3}$$

$$\therefore I_{(bottom)} = 4I_{(center)}$$

Five stiffness factors are considered including X1, X2, X3, X4, and X0.5, along with X0 (reference case with no railings). Figure 1 shows the various railing sizes considered in this investigation.

Where:

X0 No railing, reference case moment of inertia = 0
X0.5 Half the base case moment of inertia = 2Ic
X1 Moment of inertia of base case = 4Ic
X2 Twice the base case moment of inertia = 8Ic
X3 Triple the base case moment of inertia = 12Ic
X4 Four times the base case moment of inertia = 16Ic

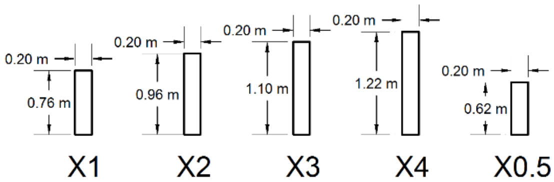

Fig. 1 Various railing sizes (X1, X2, X3, X4, X0.5).

Figure 2 shows a typical cross-section and plan-view of three-lane bridge cases with/without railings (base case, X1), with HS20 trucks placed transversely close to one edge of the slab deck with minimum spacing between trucks (Edge loading condition).

4. BRIDGE LOADING

The bridge cases considered in this study were subjected to AASHTO HS20 design trucks assuming to be traveling in the same direction when considering multiple lanes. AASHTO HS20 design trucks were placed longitudinally and transversely to produce maximum bending moments. The results of a previous study indicated that the Edge loading condition is more critical than the Centered loading condition where trucks are centered transversally each in its own lane [8]. Therefore, only the Edge loading condition was adopted in this study. Figure 2 shows the Edge loading condition for the three-lane bridge case where the first design truck was placed close to one edge of the slab, such that the center of the left wheel of the leftmost truck is positioned at 0.3 m (1 ft) from the left edge of the slab, and the other trucks were placed side-by-side with a distance 1.2 m (4 ft) between the adjacent trucks in order to produce the worst live loading condition on the bridge.

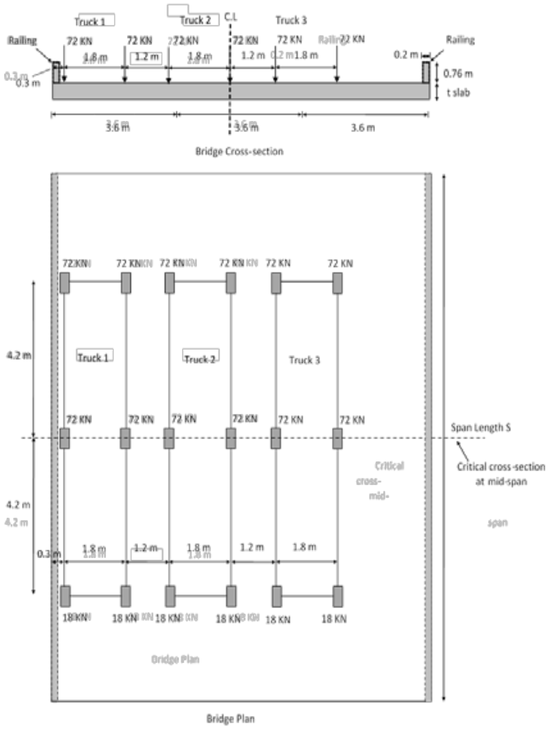

Fig. 2 Typical cross-section and plan-view of a three-lane bridge with base case railings (X1).

5. FINITE ELEMENT MODELING

A total of 48 slab bridge cases were investigated using the 3D FEA. The computer program SAP2000 (version 17) was used to discretize the bridge geometry into a convenient number of square four-node shell elements with six degrees of freedom at each node [11]. Railings could be modeled as shell elements placed orthogonally on top and along the edges of each slab, which represent a realistic but complex geometric model. A previous study showed that railings modeled as beam elements placed "eccentrically" along the slab edges with the moment of inertia calculated about its base, as described in Section 3, gave similar results for longitudinal moments than when modeled with shell elements [8]. Therefore, the simpler eccentric beam element was adopted to model the railings in this study. Figure 3 illustrates a typical 3D finite element model with the corresponding longitudinal bending moment contours for a 10.8 m (36 ft) span, three-lane bridge, in the presence of two railings, and subject to HS20 Edge loading condition.

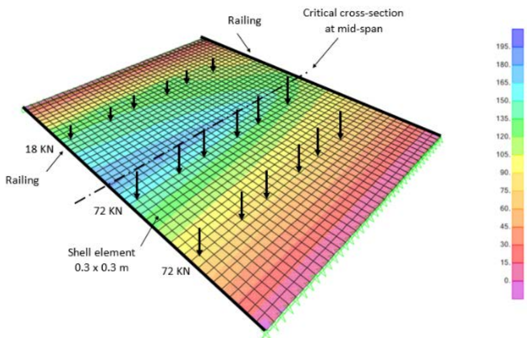

Fig. 3 3D-FEA model and longitudinal bending moments (KN-m/m) for a 10.8 m (36 ft), three-lane bridge, with base case railings (X1) on both edges of the slab.

6. FINITE-ELEMENT ANALYSIS RESULTS

The 3D-FEA results are reported in terms of the maximum longitudinal bending moments at critical locations in the concrete slab bridges. The FEA results for bridges with railings of different stiffness factors were compared with reference bridge cases without railings as well as with AASHTO Standard Specifications and LRFD procedures.

6.1. FEA Results vs. AASHTO Procedures

Figure 4 shows typical 3D-FEA longitudinal bending moments at the critical sections for all the three-lane bridge cases in combination with the four span lengths (S) and base case railings (X1). Figure 5 shows the bending moment variations at the critical section for all the three-lane bridges of 10.8 m (36 ft) span length, considering different railing stiffness (X0, X0.5, X1, X2, X3, X4), as compared with the AASHTO Standard Specifications and LRFD moments. The maximum 3D-FEA longitudinal moments in Figure 5 for the concrete slabs was defined as the first peak value occurring after the maximum value at the leftmost edge, the latter moment assumed to be resisted by the edge beam.

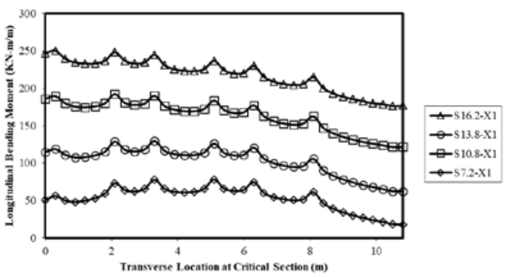

Fig. 4 3D-FEA longitudinal bending moments for three-lane bridges with all span lengths (S and base case railings (X1).

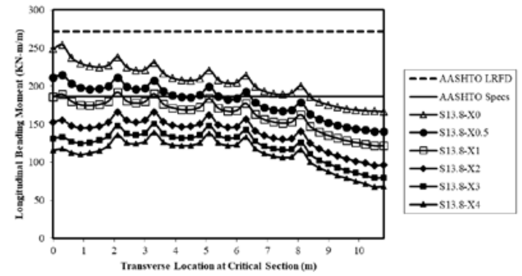

Fig. 5 3D-FEA longitudinal bending moments for 10.8 m (36 ft) span, three-lane bridge, and various railings stiffness.

Table 1 summarizes the increase or decrease in predicting bending moments in the concrete slabs when comparing the maximum FEA with the AASHTO Standard Specifications moments (Eq. 1 or 2) for all the bridge cases. Using Table 1, it can be observed that, for bridge cases with no railings (X0), for three and four lanes, AASHTO Standard Specifications generally tends to give similar moments for short-span bridges, or lower results (by about 20%) as compared to the FEA slab moments for spans longer than 10.8 m and either three- or four-lanes. When base case railings (X1) are present in a concrete slab, the FEA slab moments were off by about 10% of AASHTO moments in three- and four-lane bridges. In addition, as the stiffness factor of railings increases, the FEA moments decrease and a more significant AASHTO overestimation was observed to be about 10%.

Table 1. Comparison of FEA Maximum Slab Longitudinal Bending Moments and AASHTO Standard Specifications Moments

Number of Lanes	Span Length (m)	FEA Maximum Longitudinal Moments (KN-m/m) and Percent Difference with AASHTO Specs						AASHTO Specs Moments (KN-m/m)
		Stiffness Factor						
		X0		X1		X2		
3	7.2	97.7	-1%	78.1	24%	75.0	30%	97.2
	10.8	168.7	-14%	129.4	13%	114.1	28%	145.8
	13.8	238.1	-22%	192.2	-3%	166.0	12%	186.3
	16.2	292.6	-23%	248.6	-9%	219.0	3%	225.9
4	7.2	100.3	-3%	86.3	13%	84.4	15%	97.2
	10.8	175.9	-17%	142.1	3%	131.5	11%	145.8
	13.8	248.5	-25%	207.4	-10%	185.6	0%	186.3
	16.2	304.7	-26%	266.5	-15%	239.5	-6%	225.9
		X3		X4		X0.5		
3	7.2	73.5	32%	72.7	34%	84.7	15%	97.2
	10.8	106.1	37%	101.4	44%	143.9	1%	145.8
	13.8	150.2	24%	139.0	34%	211.6	-12%	186.3
	16.2	197.6	14%	182.3	24%	268.2	-16%	225.9
4	7.2	83.6	16%	83.1	17%	88.7	10%	97.2
	10.8	126.0	16%	122.6	19%	152.7	-5%	145.8
	13.8	173.3	8%	164.4	13%	225.1	-17%	186.3
	16.2	221.3	2%	208.4	8%	283.7	-20%	225.9

With reference to Table 2, AASHTO LRFD overestimates the FEA slab moments in almost all bridge cases with or without railings. Assuming no railings on the bridge, AASHTO LRFD overestimates the FEA slab moments by about 15% for three-lane bridges, and by about 10% for four-lane bridges. This overestimation increases with the increase in railing stiffness. When base case railings (X1) are present, the AASHTO LRFD overestimation of the FEA slab moments reaching an average high of 40% in three-lane bridges or 30% in four-lane bridges. This overestimation is further increased as the railings stiffness factor increases where it reaches 90% for three-lane bridges and around 60% for four-lane bridges with (X4) railings stiffness factor.

Table 2. Comparison of FEA Maximum Slab Longitudinal Bending Moments and AASHTO LRFD Moments

Number of Lanes	Span Length (m)	FEA Maximum Longitudinal Moments (KN-m/m) and Percent Difference with AASHTO LRFD						AASHTO LRFD Moments (KN-m/m)
		Stiffness Factor						
		X0		X1		X2		
3	7.2	97.7	4%	78.1	30%	75.0	36%	101.7
	10.8	168.7	13%	129.4	47%	114.1	67%	190.4
	13.8	238.1	14%	192.2	41%	166.0	64%	271.8
	16.2	292.6	19%	248.6	40%	219.0	58%	347.0
4	7.2	100.3	-4%	86.3	12%	84.4	15%	96.8
	10.8	175.9	2%	142.1	27%	131.5	37%	180.0
	13.8	248.5	8%	207.4	30%	185.6	45%	269.1
	16.2	304.7	14%	266.5	30%	239.5	45%	347.0
		X3		X4		X0.5		
3	7.2	73.5	38%	72.7	40%	84.7	20%	101.7
	10.8	106.1	79%	101.4	88%	143.9	32%	190.4
	13.8	150.2	81%	139.0	96%	211.6	28%	271.8
	16.2	197.6	76%	182.3	90%	268.2	29%	347.0
4	7.2	83.6	16%	83.1	16%	88.7	9%	96.8
	10.8	126.0	43%	122.6	47%	152.7	18%	180.0
	13.8	173.3	55%	164.4	64%	225.1	20%	269.1
	16.2	221.3	57%	208.4	66%	283.7	22%	347.0

6.2. FEA Results With Railings vs. With No Railing

The maximum slab bending moments are summarized in Table 3 for all bridge cases in terms of ratios of FEA results for cases with various railing stiffness factors as compared to the corresponding cases without railings (reference case, X0). Table 3 shows that the presence of railings reduces the maximum longitudinal slab moment and this decrease is evidently more pronounced as the railings stiffness factor increases. For three-lane bridges, the slab moments reduced by about 20% when adding railing with stiffness factor (X1) as compared to the same bridge which was reduced by 40% for (X4) railing stiffness factor. As for four-lane bridges, the slab moments reduced by about 15% with X1 railing stiffness factor, compared to about 30% with X4 railing stiffness factor. It is worth mentioning that the reduction in moments is higher in three-lane bridges than in the wide four-lane bridges; also the rate of the increase of the reduction decreases as the railing stiffness factor increases.

Table 3. Comparison of FEA Results of Railing Stiffness to Reference Case without Railing

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		
3	7.2	97.7	1.00	78.1	0.80	75.0	0.77	97.7
	10.8	168.7	1.00	129.4	0.77	114.1	0.68	168.7
	13.8	238.1	1.00	192.2	0.81	166.0	0.70	238.1
	16.2	292.6	1.00	248.6	0.85	219.0	0.75	292.6
4	7.2	100.3	1.00	86.3	0.86	84.4	0.84	100.3
	10.8	175.9	1.00	142.1	0.81	131.5	0.75	175.9
	13.8	248.5	1.00	207.4	0.83	185.6	0.75	248.5
	16.2	304.7	1.00	266.5	0.87	239.5	0.79	304.7
		X3		X4		X0.5		
3	7.2	73.5	0.75	72.7	0.74	84.7	0.87	97.7
	10.8	106.1	0.63	101.4	0.60	143.9	0.85	168.7
	13.8	150.2	0.63	139.0	0.58	211.6	0.89	238.1
	16.2	197.6	0.68	182.3	0.62	268.2	0.92	292.6
4	7.2	83.6	0.83	83.1	0.83	88.7	0.89	100.3
	10.8	126.0	0.72	122.6	0.70	152.7	0.87	175.9
	13.8	173.3	0.70	164.4	0.66	225.1	0.91	248.5
	16.2	221.3	0.73	208.4	0.68	283.7	0.93	304.7

7. SUMMARY AND CONCLUSIONS

AASHTO Standard Specifications and AASHTO LRFD empirical equations do not account for the presence of railings as integral parts of a bridge slab, and these elements are neglected during the design stage. Based on the 3D finite-element analysis, it is evident that the presence of railings increases the load-carrying capacity of the concrete slab bridges if they are modeled and constructed as integral parts of the slab. It was also found that the maximum longitudinal bending moment, due to the presence of standard railings placed integrally on both edges of the slab deck, reduced by about 40% for one-lane, 30% for two-lanes, 20% for three-lanes, and 15% for four-lane bridges. This reduction in the slab moment gets smaller with the increase in the number of lanes and gets larger with the increase in the railing stiffness. Therefore, the presence of side railings increases the load-carrying capacity of reinforced concrete slab bridges and the addition of stiffer railings can be used as an alternative strengthening technique to upgrade existing bridges that require rehabilitation or to allow permit vehicles on the bridge.

8. ACKNOWLEDGMENT

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