International Journal of GEOMATE, May, 2017, Vol. 12, Issue 33, pp. 134-138
Geotec., Const. Mat. & Env., ISSN:2186-2990, Japan, DOI: http://dx.doi.org/10.21660/2017.33.2795

INFLUENCE OF RAILINGS STIFFNESS ON WHEEL LOAD DISTRIBUTION IN ONE- AND TWO-LANE CONCRETE SLAB BRIDGES

Mohammad Abou Nouh[1], Ghassan Fawaz[1], Mounir Mabsout[1*], and Kassim Tarhin[2]

[1]Dept. of Civil and Environmental Engineering, Amer. Univ. of Beirut, Lebanon.

[4]Dept. of Civil Engineering, U.S. Coast Guard Academy, New London, CT 06320, USA.

*Corresponding Author, Received: 2 July 2016, Revised: 22 Aug. 2016, Accepted: 20 Dec. 2016

ABSTRACT: The American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (AASHTO LRFD) do not account for the presence of railings as integral parts of highway bridges. This paper presents the parametric investigation of the influence of railings stiffness on the wheel load distribution in simply-supported, one-span, one- and two-lane reinforced concrete slab bridges using the finite-element analysis (FEA). A total of 80 bridge cases are modeled and bridge parameters such as span lengths and slab widths were varied within practical ranges. Various railings built integrally with the bridge deck are 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 slabs by 25% to 60% depending on the stiffness of the railings. The results of this investigation will assist structural and bridge engineers in better designing or evaluating concrete slab bridges in the presence of railings. This can also be considered to be a possible alternative for strengthening existing concrete slab bridges.

Keywords: Concrete Slab Bridges, Railings 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 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 in resisting highway loading and 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) [9]. Results indicated that AASHTO Specifications moments overestimate the FEA moments for short spans, one lane bridges and agreed with FEA moments for short spans in combination of two or more lanes. Also, AASHTO Specifications underestimates the FEA moments for longer spans. As for AASHTO LRFD procedure, it overestimates FEA moments for all bridge cases. Several studies were conducted to investigate 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 [3]–[4]–[5]–[6]–[8].

Recently, a parametric investigation studying the influence of one standard railings size on straight concrete slab bridges was performed [7]. The results indicated that placing two railings on straight bridges, AASHTO Standard Specifications procedures overestimated the FEA moments by 100% for one-lane bridges, and by 20% for bridges with two lanes. AASHTO LRFD overestimated the FEA moments in all bridge cases by 150% for one-lane, and 70% for two-lanes when placing two railings on slab bridges. It is worth noting that the AASHTO Procedures which overestimated the FEA results above do not consider the stiffness or the effect of side railings.

This paper presents the results of a parametric study investigating the influence of railings stiffness on the increase in load carrying capacity in reinforced concrete slab bridges.

2. AASHTO BENDING MOMENTS AND SLAB THICKNESS

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.

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

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

Where:
S = span length (m)
M = longitudinal bending moment per unit width (N-m/m)

AASHTO LRFD Section 4.6.2.3 (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 (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:

Width for one lane loaded is:

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

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 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.
AASHTO Specifications and AASHTO LRFD do not take into account the influence of side railings on concrete slab bridges.

3. DESCRIPTION OF BRIDGE CASES

Typical simply-supported one-span, one-lane, and two-lane reinforced concrete slab bridge cases were analyzed in this investigation. Four single 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 reported in earlier sections. 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 adopted from previous research was 200 mm (8 in) wide and 760 mm (30 in) high above slab [7]. 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.

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

Five stiffness factors are considered including X0, X1, X2, X3, X4, and X0.5, along with X0 (reference case with no railings).
Where:
X0 No Railings, Reference case = 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

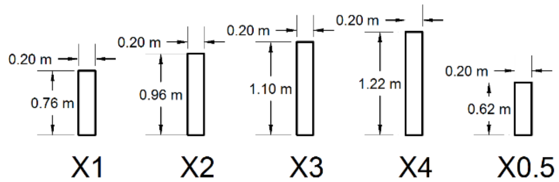

Fig. 1 Various railing sizes (X0, X1, X2, X3, X4, X0.5).

Various railings sizes are shown in Fig.1. Figure 2 shows a typical cross-section and plan-view of two-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).

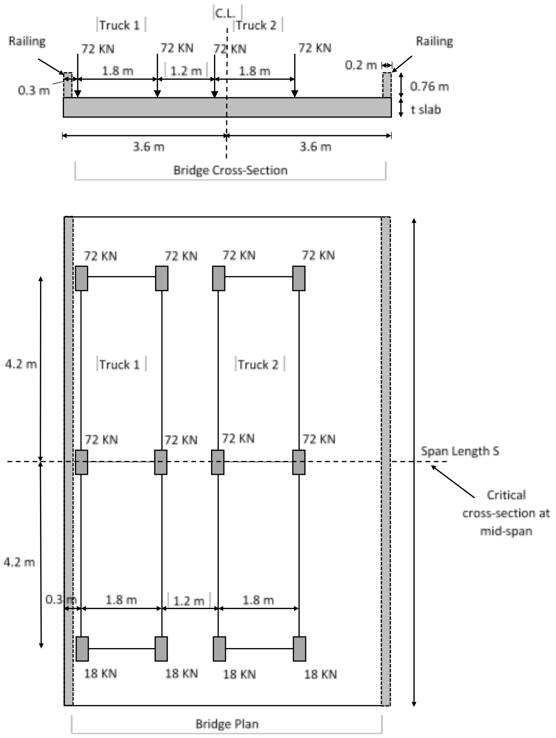

Fig. 2 Typical cross-section and layout for a two-lane bridge subject to Edge loading condition with base case railings (X1).

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 [7]. Therefore, only the Edge loading condition was adopted in this study. Figure 2 shows the Edge loading condition for the two-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 left most 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.

5. FINITE ELEMENT MODELING

A total of 80 slab bridge cases were investigated using the FEA. The computer program SAP2000 (version 17) was used to discretize the bridge into a convenient number of square four-node shell elements with six degrees of freedom at each node [10]. A previous study which investigated the influence of railings, showed that railings modeled as beam elements placed "eccentrically" along the slab edges with the second moment of area calculated about its base, gave similar results for longitudinal moments for models where railings were modeled as shell elements placed orthogonally on top and along the edges of each slab which represent a realistic geometric model [7]. Therefore, the simpler eccentric beam element was adopted to model the railings in this study. Figure 3 illustrates a typical finite element model with the corresponding longitudinal bending moment contours for a 10.8 m (36 ft) span, two-lane bridge, in the presence of two railings, and subject to HS20 Edge loading condition.

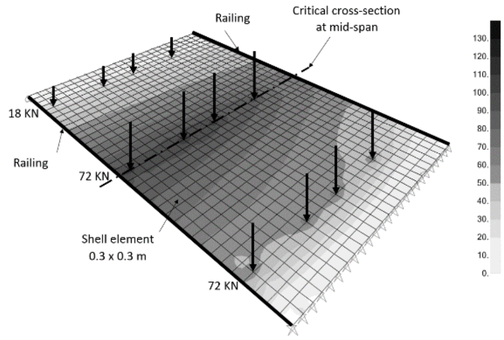

Fig. 3 FEA longitudinal bending moments (KN-m/m) for two-lane Bridge with base case railings (X1).

6. FINITE-ELEMENT ANALYSIS RESULTS

The 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

Figure 4 shows sample plots of the FEA longitudinal bending moment 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 5 shows the bending moment plots for all the two-lane bridges with 10.8 m (36 ft) span length, with different railing configurations (X0, X0.5, X1, X2, X3, X4), along with the AASHTO moments.

The maximum 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 which is assumed to be resisted by the edge beam.

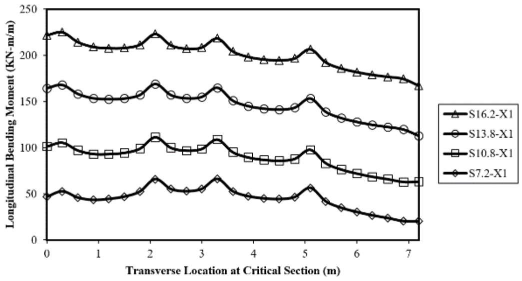

Fig. 4 FEA longitudinal bending moments for two-lane bridges with base case railings (X1).

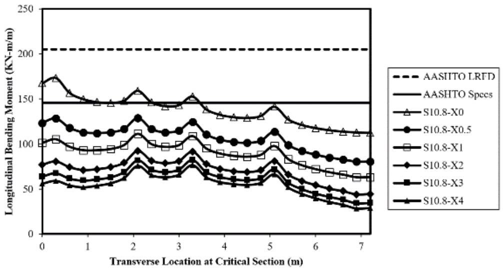

Fig. 5 FEA longitudinal bending moments for a 10.8 m (36 ft) span, two-lane bridge, with various railings sizes (X0, X0.5, X1, X2, X3, X4), with AASHTO Specs and LRFD.

Table 1 summarizes the increase or decrease in predicting bending moments in the concrete slabs when comparing the maximum FEA with the AASHTO Specs moments for all the bridge cases. Using Table 1, it can be observed that, for bridge cases with no railings (X0), AASHTO Standard Specifications generally tends to give similar results to the FEA slab moments, with the exception of one-lane with spans less than 12 m (40 ft) where the AASHTO overestimates FEA moments by about 20%. This is more pronounced with more lanes and longer spans, where AASHTO underestimates FEA moments reaching up to 20% 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 moments decrease significantly and AASHTO overestimates or gives similar moments in almost all cases, reaching 100% 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 as the FEA moments decrease and a more significant AASHTO overestimation is observed. This overestimation reaches 170% for one-lane bridges and is around 70% for two-lane bridges with (X4) railings stiffness factor.

Table 1 Comparison of FEA Maximum Slab Longitudinal Bending Moments and AASHTO 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		
1	7.2	74.5	30%	44.8	117%	39.0	149%	97.2
	10.8	131.0	11%	75.9	92%	59.6	145%	145.8
	13.8	188.8	-1%	119.0	57%	91.8	103%	186.3
	16.2	235.1	-4%	162.8	39%	128.1	76%	225.9
2	7.2	92.3	5%	65.9	47%	60.5	61%	97.2
	10.8	159.0	-8%	111.3	31%	92.4	58%	145.8
	13.8	226.6	-18%	168.8	10%	139.4	34%	186.3
	16.2	280.6	-19%	222.9	1%	188.3	20%	225.9
		X3		X4		X0.5		
1	7.2	36.5	166%	35.2	176%	52.2	86%	97.2
	10.8	51.8	181%	47.3	209%	93.1	57%	145.8
	13.8	77.3	141%	68.2	173%	144.1	29%	186.3
	16.2	107.8	110%	94.4	139%	191.2	18%	225.9
2	7.2	58.0	68%	56.5	72%	73.4	33%	97.2
	10.8	82.2	77%	76.4	91%	128.6	13%	145.8
	13.8	121.4	53%	109.3	70%	192.0	-3%	186.3
	16.2	165.2	37%	148.5	52%	247.6	-9%	225.9

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		
1	7.2	74.5	70%	44.8	182%	39.0	224%	126.5
	10.8	131.0	62%	75.9	180%	59.6	256%	212.4
	13.8	188.8	50%	119.0	138%	91.8	208%	283.1
	16.2	235.1	44%	162.8	108%	128.1	164%	338.9
2	7.2	92.3	18%	65.9	65%	60.5	79%	108.5
	10.8	159.0	29%	111.3	84%	92.4	122%	205.2
	13.8	226.6	30%	168.8	74%	139.4	111%	293.9
	16.2	280.6	31%	222.9	65%	188.3	95%	367.7
		X3		X4		X0.5		
1	7.2	36.5	246%	35.2	259%	52.2	142%	126.5
	10.8	51.8	310%	47.3	350%	93.1	128%	212.4
	13.8	77.3	266%	68.2	315%	144.1	96%	283.1
	16.2	107.8	214%	94.4	259%	191.2	77%	338.9
2	7.2	58.0	87%	56.5	92%	73.4	48%	108.5
	10.8	82.2	150%	76.4	169%	128.6	60%	205.2
	13.8	121.4	142%	109.3	169%	192.0	53%	293.9
	16.2	165.2	123%	148.5	148%	247.6	48%	367.7

With reference to Table 2, AASHTO LRFD overestimates the FEA slab moments in almost all bridge cases with or without railings. AASHTO LRFD overestimates the FEA slab moments by about 50% for one-lane bridges and about 30% for two-lane bridges. 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 150% in one-lane bridges or 70% in two-lane bridges. This overestimation is further increased as the railings stiffness factor increases where it reaches 250% for one-lane bridges and around 150% for two-lane bridges with (X4) railings stiffness factor.

6.3. FEA RESULTS Railings vs. 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 railings stiffness factors 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 increase is more pronounced as the railings stiffness factor increases. For one-lane bridges, maximum longitudinal moment reduces by 40% when adding railing with stiffness factor (X1) and it reduces by 60% with (X4) railing stiffness factor. As for two-lane-bridges, the slab moment reduces by 25% with X1 railing stiffness factor and by about 50% with X4 railing stiffness factor. Worth mentioning that the rate of the increase of the reduction decreases as the railing stiffness factor increases.

Table 3. Comparison of FEA Results 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	74.5	1.00	44.8	0.60	39.0	0.52	74.5
	10.8	131.0	1.00	75.9	0.58	59.6	0.46	131.0
	13.8	188.8	1.00	119.0	0.63	91.8	0.49	188.8
	16.2	235.1	1.00	162.8	0.69	128.1	0.54	235.1
2	7.2	92.3	1.00	65.9	0.71	60.5	0.66	92.3
	10.8	159.0	1.00	111.3	0.70	92.4	0.58	159.0
	13.8	226.6	1.00	168.8	0.74	139.4	0.62	226.6
	16.2	280.6	1.00	222.9	0.79	188.3	0.67	280.6
		X3		X4		X0.5		
1	7.2	36.5	0.49	35.2	0.47	52.2	0.70	74.5
	10.8	51.8	0.40	47.3	0.36	93.1	0.71	131.0
	13.8	77.3	0.41	68.2	0.36	144.1	0.76	188.8
	16.2	107.8	0.46	94.4	0.40	191.2	0.81	235.1
2	7.2	58.0	0.63	56.5	0.61	73.4	0.80	92.3
	10.8	82.2	0.52	76.4	0.48	128.6	0.81	159.0
	13.8	121.4	0.54	109.3	0.48	192.0	0.85	226.6
	16.2	165.2	0.59	148.5	0.53	247.6	0.88	280.6

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 finite-element analysis, it is clearly evident that these elements increase the capacity of the bridges if they are modeled as integral parts of the slab. It was found that the maximum slab moment was reduced due to the presence of two railings. This reduction in the slab moment decreases with the increase in the number of lanes, and increases with the increases in the railing stiffness. These railings can be used as one alternative strengthening technique to upgrade existing bridges that require rehabilitation or to allow permit vehicles on the bridge.

8. ACKNOWLEDGMENT

This research was supported by a grant from the University Research Board (URB) at the American University of Beirut to whom the authors are indebted and thankful.

9. REFERENCES

[1] AASHTO (2002). *Standard specifications for highway bridges*, 17th Ed., American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

[2] AASHTO (2012). *LRFD bridge design specifications*, 5th Ed., American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

[3] Akinci, N.O., Liu, J., and Bowman, M.D. (2008). "Parapet strength and contribution to live load response for super load passages." *J. Bridge Eng.*, 13(1), 55-63.

[4] Chung, W., Liu, J., and Sotelino, E.D. (2006). "Influence of secondary elements and deck cracking on the lateral load distribution of steel girder bridges." *J. Bridge Eng.*, 11(2), 178-187.

[5] Conner, S., and Huo, X.S. (2006). "Influence of parapets and aspect ratio on live-load distribution." *J. Bridge Eng.*, 11(2), 188-196.

[6] Eamon, C., and Nowak, A. (2002). "Effects of edge-stiffening elements and diaphragms on bridge resistance and load distribution." *J. Bridge Eng.*, 7(5), 258-266.

[7] Fawaz, G., Waked, M., Mabsout, M., and Tarhini, K. (2016). "Influence of railings on load carrying capacity of one- and two-lane concrete slab bridges." Sixth Annual International Conference in Civil Engineering, Athens, Greece.

[8] Mabsout, M., Tarhini, K., Frederick, G., and Kobrosly M. (1997). "Influence of sidewalks and railings on wheel load distribution in steel girder highway bridges." *J. Bridge Eng.*, 2(3), 88-96.

[9] Mabsout, M., Tarhini, K., Jabakhanji, R., and Awwad, E. (2004). "Wheel load distribution in simply supported concrete slab bridges." *J. Bridge Eng.*, 9(2), 147-155.

[10] SAP2000 (version 17). Computers and Structures Inc., Berkeley, California.