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WHEEL LOAD DISTRIBUTION IN STRAIGHT AND SKEWED CONCRETE SLAB BRIDGES STIFFENED WITH RAILINGS

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ABSTRACT: This paper presents the parametric investigation of the influence of railings on the wheel load distribution in simply-supported, one-span, three- and four-lane straight and skewed reinforced concrete slab bridges using the finite element method. A total of 96 bridge cases were modeled using finite-element analysis (FEA) and bridge parameters such as span length, slab width, and skew angle are varied within practical ranges. Typical railings built integrally with the bridge were placed on both edges of the deck slabs. AASHTO HS20 truck loadings were positioned transversely and longitudinally to produce maximum longitudinal live load bending moments in the slabs. The FEA wheel load distribution and bending moments were compared with reference straight bridges without railings as well as to the AASHTO Standard Specifications for Highway Bridges and the AASHTO LRFD Bridge Design Specifications. AASHTO overestimates FEA moments for almost all bridge cases and this overestimation increases with the increase in the skew angle, and it is more significant in the presence of two railings. Also, it was found that the reduction in slab moment due to skewness and railings is cumulative. The presence of railings can be considered to be a possible method for strengthening and rehabilitating straight and skewed concrete slab bridges.

Keywords: Concrete slab bridges, Multi-lane, Skew angle, Railings or parapets, 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.

Skewed bridges are often encountered in highway design when the geometry cannot accommodate straight bridges. The skew angle can be defined as the angle between the normal to the centerline of the bridge and the centerline of the abutment or pier cap as described in Fig. 1.

The design of highway bridges in the United States conforms to the American Association of State Highway and Transportation Officials (AASHTO) procedures, either to the Standard Specifications for Highway Bridges (2002) prior to 2007 – thereafter referred to as AASHTO Standard [1], or to the current Load and Resistance Factor Design (LRFD) Specifications (2014) – thereafter referred to as AASHTO LRFD [2]. The current AASHTO procedures do not consider the effect of railings that are built integrally with the bridge deck, and only AASHTO LRFD accounts for skewness in the evaluation of the load-carrying capacity of bridges. Therefore, this study investigates the combined effect of railings and skew angle in resisting highway loading and increasing the load-carrying capacity of reinforced concrete slab bridges.

Several studies were conducted on 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 [3-7].

A parametric study of straight, single-span, multi-lane (1 to 4 lanes) simply-supported reinforced concrete slab bridges, using finite-element analysis (FEA) was also reported [8]. Results indicated that AASHTO Standard 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 Standard underestimates the FEA moments for longer spans. As for AASHTO LRFD procedure, it overestimates FEA moments for all bridge cases. This study was extended to study the influence of skew angle on the same concrete slab bridges [9]. The ratio between the FEA longitudinal moments for skewed and straight bridges was almost one for bridges with skew angle less than 20 degrees. This ratio decreased to about 0.75 for bridges with skew angles between 30 degrees and 40 degrees, and further decreased to about 0.50 skew angle of the bridge increased to 50 degrees.

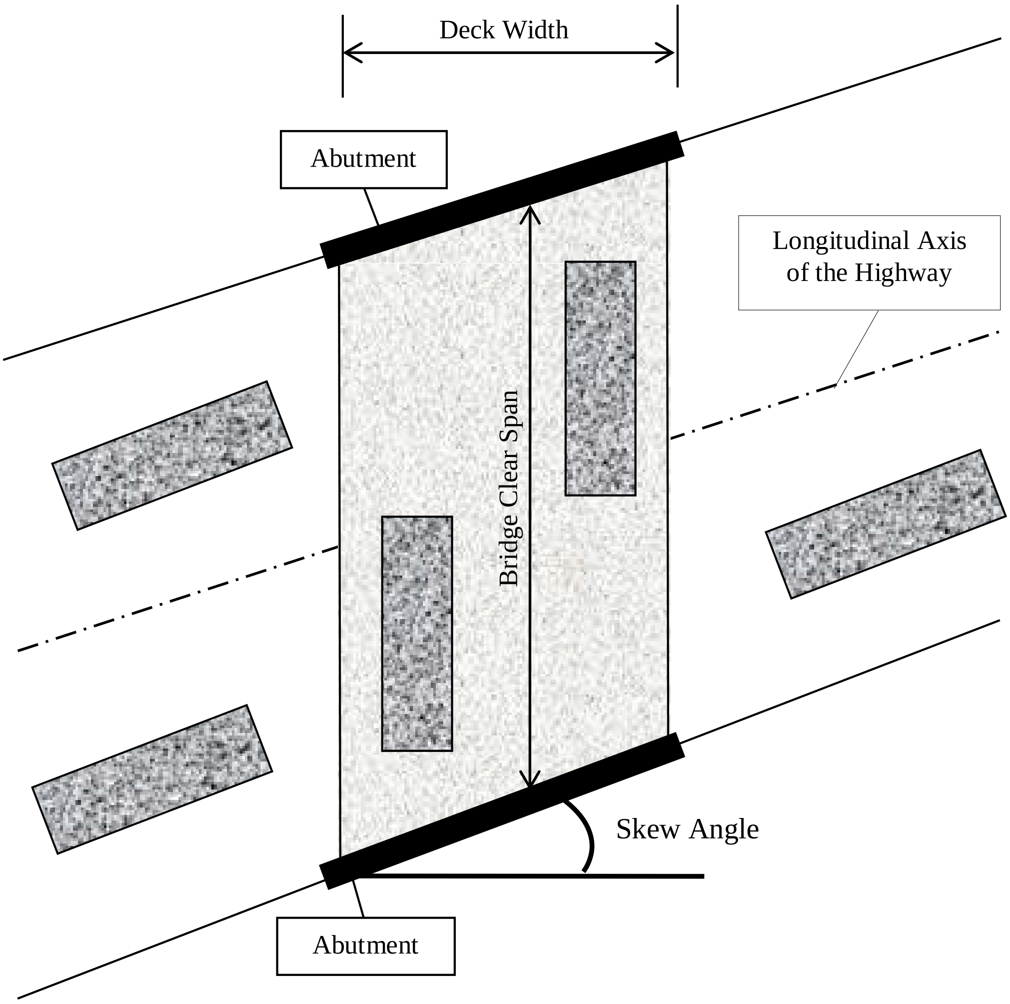

Fig. 1 Description of skewed bridge

Further, a study investigated the influence of one standard railing built integrally on either one or both edges of the slab deck, for the same straight bridges [10]. The results indicated that placing two railings on the bridge slab, AASHTO Standard overestimated the FEA moments by 100% for one-lane bridges, and by 20% for bridges with two or more lanes, while 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. Finally, a preliminary study considered the combination of railings and skewness on the concrete slab bridges previously analyzed for the cases of one and two lanes [11]. With no railing, AASHTO Standard generally tends to give similar results to the FEA slab moments for skew angle up to 20 degrees. As skew angle increases, AASHTO overestimation increases till it reaches 100% for bridges with skew angle of 50 degrees. Also, generally AASHTO overestimation is higher for one-lane bridges as compared to two-lane bridges and it decreases with the increase in span length. Adding two railings, AASHTO overestimates FEA moments for all bridge cases and this overestimation increases with the increase in skew angle reaching 140% for a skew angle of 50 degrees. Further, AASHTO LRFD overestimates the FEA slab moments in almost all bridge cases with or without railings. This overestimation increases with the increase in the skew angle and it is most significant for bridges with two railings and a skew angle of 50 degrees. For straight bridges with no railings, AASHTO LRFD overestimates the FEA slab moments by about 50% for one-lane bridges and about 30% for two-lane bridges. When the skew angle increases to 50 degrees, AASHTO LRFD overestimates the FEA slab moments by about 125% for one-lane bridges and about 100% for two-lane bridges. When two railings are present, and for straight bridges, 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. As for bridges of skew angle of 50 degrees this overestimation reaches its maximum values and is 190% for one-lane bridges and 140% for two-lane bridges.

This paper builds on the previously published research, namely in [8-11], by performing a parametric study investigating the influence of railings and skweness on wheel load distribution in simply-supported, one-span, three- and four-lane concrete slab bridges. The FEA slab moments will be assessed with AASHTO Standard and LRFD, as well as with the reference straight bridge cases without railing.

2. AASHTO STANDARD AND LRFD PROCEDURES

For simply-supported concrete slab bridges, AASHTO Standard [1] 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.

$$M = 13{,}500S \ \ for\ S \leq 15m \tag{1a}$$

or

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

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

AASHTO Standard Section 3.2.6 suggests that slab bridges with a skew angle less than 30 degrees be designed as a typical slab at right angles, or as straight bridges, with no modifications. However, if the skew angle exceeds 30 degrees, AASHTO suggests the use of an alternate superstructure configuration.

AASHTO LRFD [2] Section 4.6.2.3 provides an equivalent strip width procedure to design reinforced concrete slab bridges that is comparable to procedures specified in the AAHSTO Standard. 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:

$$E = 250 + 0.42\sqrt{L1 \times W1} \tag{2}$$

while the width for multi-lanes loaded is:

$$E = 2{,}100 + 0.12\sqrt{L1 \times W1} \tag{3}$$

where:
M = longitudinal bending moment per unit width (N-m/m)
E = equivalent width of longitudinal strips per lane (mm)
$L1$ = span length (mm), the lesser of the actual span or 18,000 mm
$W1$ = edge-to-edge width of the bridge (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.

For skewed bridges, AASHTO LRFD 4.6.2.3-3 reduces longitudinal force effects by a factor "r" which is a function of the skew angle:

$$r = 1.05 - 0.25\tan\theta \leq 1\ldots \tag{4}$$

where θ is the skew angles in degrees.

The current AASHTO procedures (Standard or LRFD) do not consider the influence of railings that are built integrally with the bridge deck on the increase of the bridge stiffness and its load-carrying capacity.

3. BRIDGE CASES AND LOADING

Typical simply-supported one-span, three-lane and four-lane, straight and skewed reinforced concrete slab bridge cases were analyzed in this study, without and in the presence of integral railings. Four span lengths were considered, 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 lane width was assumed to be 3.6 m (12 ft) and therefore the overall slab widths were taken as 10.8 m (36 ft) for three lanes and 14.4 m (48 ft) for four lanes. Six skew angles varying between 0° and 50° by increments of 10° were considered. A straight bridge is defined as having a 0° skew angle. Standard railings, 200 mm (8 inches) wide and 760 mn (30 inches) high above roadway, were placed integrally on both sides of the slab edges (labeled as R2). Straight bridges without railings were first analyzed and considered as the reference cases (labeled as R0). Figure 2 shows a typical cross-section and plan layout of a 13.8 m (46 ft) span, three-lane bridge case, with 30° skew angle and railings placed on both edges of the slab deck.

The bridge cases considered were subjected to AASHTO HS20 design trucks assumed to be traveling in the same direction placed longitudinally and transversally to produce maximum moments. For HS20 loading, the line wheel loads are 18 KN (4 Kips), 72 KN (16 Kips), and 72 KN (16 Kips) with axle spacing of 4.2 m (14 ft). The longitudinal location of HS20 axle loads that produce maximum positive moment in one-span loading condition bridges was placed such that the centerline of bridge aligned with the location of centerline halfway between the resultant load of the truck and the middle

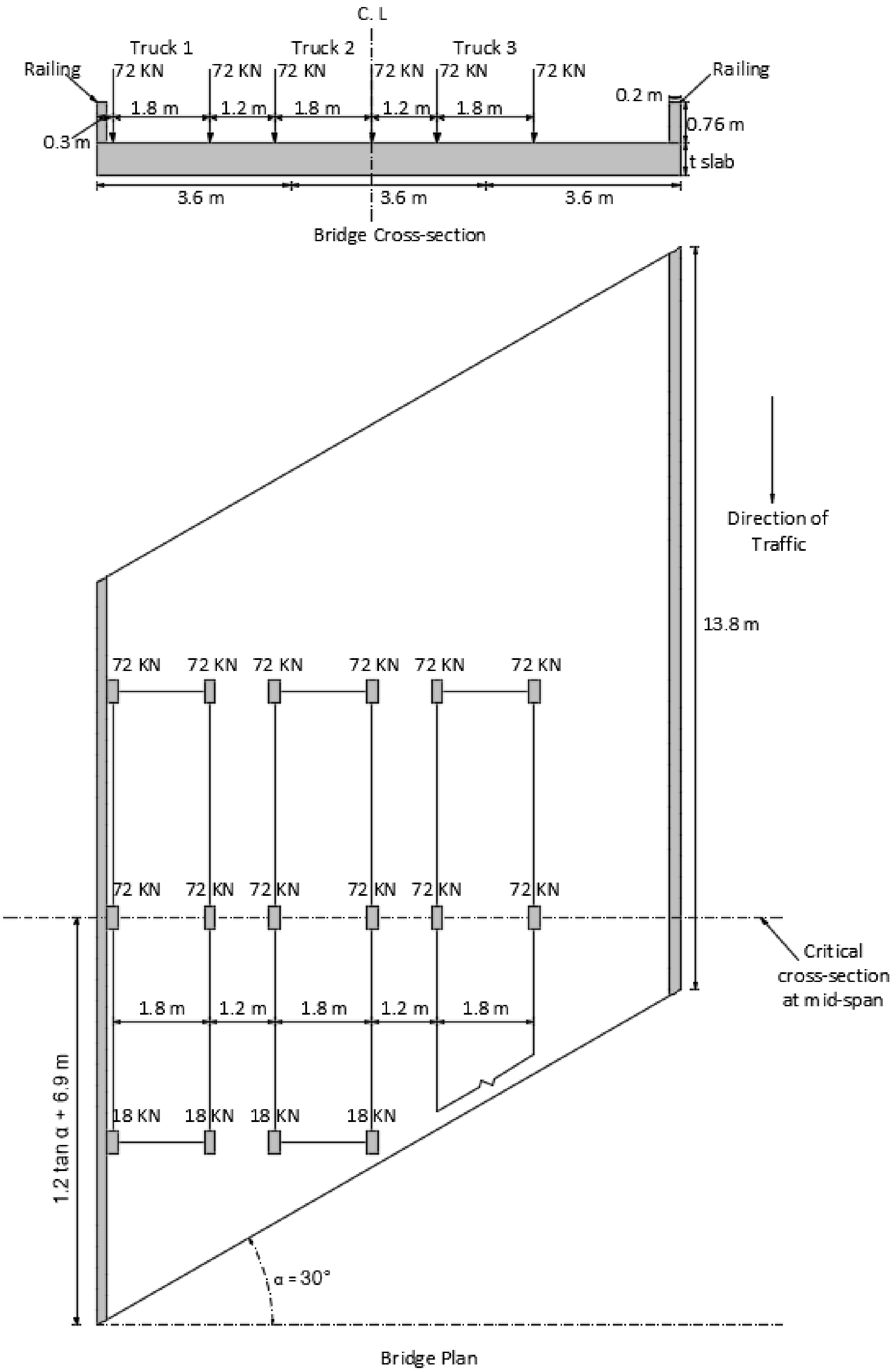

Fig. 2 Typical cross-section and layout for a 13.8 m (46 ft) span, three-lane, 30 degrees skewed bridge with railings, subject to HS20 Edge loading condition

axle. However, a previous study reported that the difference between placing the middle axle aligned with the mid-span versus the actual location mentioned above was negligible in determining the maximum bending moment in concrete slab bridges, which will therefore be adopted in this study [8]. Transversely, an Edge loading condition was applied, 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 [8]. It is also worth noting that only the left-most truck was centered longitudinally, while adjacent trucks were aligned with the edge truck as shown. This condition resulted in slightly higher moments than for the case where each adjacent truck was centered longitudinally in its own lane [9]. Figure 2 shows the Edge loading condition for the bridge case described earlier.

The material properties used in modeling the highway bridges were normal-strength reinforced concrete. The compressive strength of the concrete was 27,500 kPa (4,000 psi), the modulus of elasticity was 25×10^6 kPa (3.6×10^6 psi), and Poisson's ratio was 0.2. Grade 60 reinforcing steel could be assumed in the design of slab reinforcement, but the FEA models did not include such property in the analysis.

4. FINITE ELEMENT MODELING AND RESULTS

A total of 96 slab bridge cases were investigated using the FEA using the general computer program SAP2000 [12]. The bridge discretization was tested in previous studies where shell elements with six degrees of freedom at each node were used to model the slab [8]-[9]. A typical four-node square element size of 0.3x0.3 m (1x1 ft) was adopted and for the slab discretization four-node quadrilateral and three-node triangular elements were additionally used at the supports to accommodate for skewness. Railings placed both edges of the slab were modeled as space frame elements placed "eccentrically" along the slab edges with the second moment of area calculated about its base. This was based on previous studies which investigated the appropriate railing modeling on straight concrete slab bridges [10]-[11]. This study considered all elements to be linearly elastic and the analysis assumed small deformations and deflections. Figure 3 shows a typical FEA model for the bridge case described earlier subjected to AASHTO HS20 Edge loading condition.

The FEA results are reported in terms of the maximum longitudinal bending moments at critical cross-section locations in the concrete slab bridges, shown in Fig. 3. The FEA results for skewed bridges with railings were compared with straight reference bridge cases without railings (R0- 0 degrees) as well as with AASHTO Standard [1] and LRFD [2] procedures. Figure 4 shows sample plots of the FEA longitudinal bending moment at the critical sections for 13.8 m (46 ft) span, three-lane bridge cases with the various railing configurations (R0 for no railing; and R2 for two railings), and select skew angles (only 0, 30, and 50 degrees for clarity) subject to Edge HS20 loading conditions. The corresponding AASHTO moments are also plotted in the figure. It is worth mentioning that the maximum FEA longitudinal moments in Fig. 4 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 and/or railing when present.

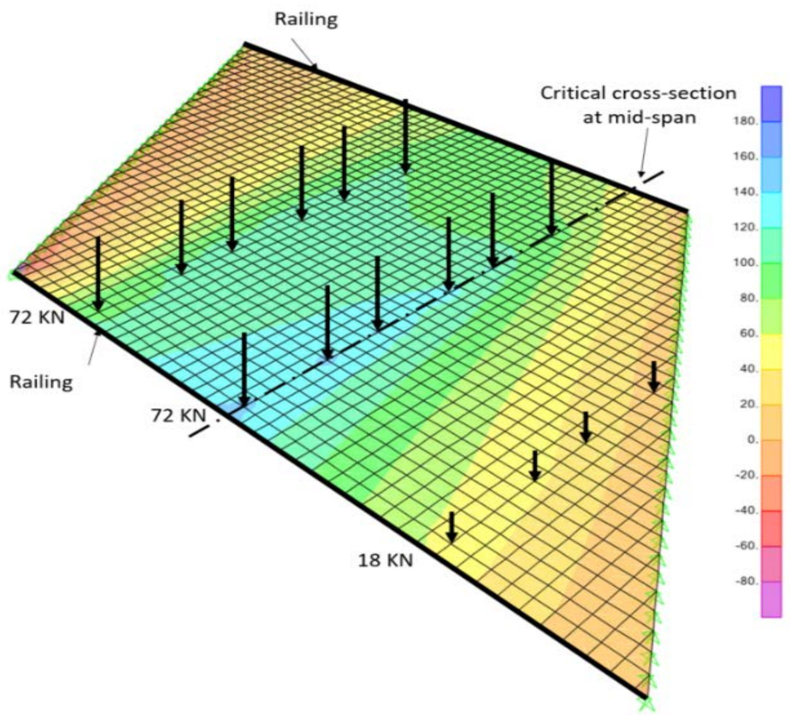

Fig. 3 FEA discretization and moments (KN-m/m) in a 13.8 m (46 ft) span, three-lane, 30° skewed bridge with railings, subject to HS20 Edge loading condition

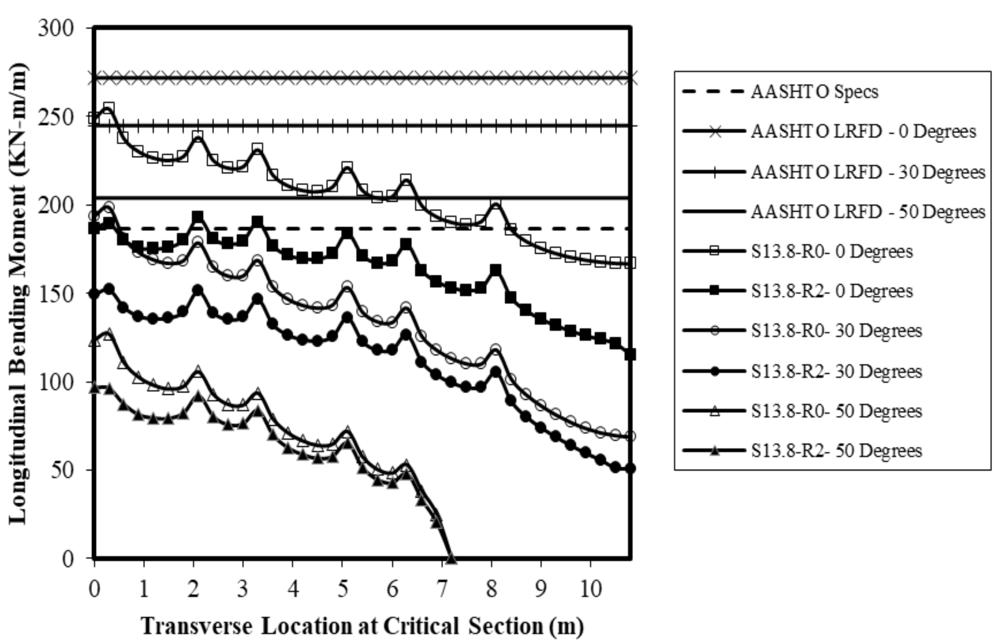

Fig. 4 FEA and AASHTO Standard (Specs) and LRFD moments for 13.8 m (46 ft) span, three-lane skewed bridges with railings and reference straight bridges without railing

5. FEA RESULTS VS. AASHTO PROCEDURES

Table 1 summarizes the increase or decrease in predicting the bending moments in the concrete slabs when comparing the maximum FEA with the AASHTO moments for all bridge cases analyzed.

Using Table 1, it can be observed that, for short-span bridge cases with no railing, AASHTO Standard generally tends to give similar results to the FEA slab moments for skew angle up to 20 degrees, and overestimates the FEA moments by about 20 to 50% for skew angle up between 30 and 50 degrees. For longer span bridge cases with no railing, AASHTO Standard generally tends to underestimate to the FEA slab moments by about 10 to 35% for skew angle up to 20 degrees, and gives similar results or overestimates the FEA moments by about 10 to 45% for skew angle up between 30 and 50 degrees. Adding two railings, AASHTO Standard overestimates FEA moments for almost all bridge cases and this overestimation increases with the increase in skew angle reaching about 60% for short-span bridges with skew angle of 50 degrees; only in few long-span bridge cases with two railings and for skew angles up to 20 degrees, then AASHTO gives similar results or underestimates the FEA moments by about 10 to 20%.

Also with reference to Table 1, it can be deduced that AASHTO LRFD generally overestimates the FEA slab moments in almost all bridge cases with or without railings, and gives similar results in a few cases without railing. For short-span bridge cases with no railing, AASHTO LRFD generally tends to give similar results to the FEA slab moments for skew angle up to 20 degrees, and overestimates the FEA moments by about 15 to 40% for skew angle up between 30 and 50 degrees. For longer span bridge cases with no railing, AASHTO LRFD overestimates the FEA slab moments by about 10 to 20% for skew angle up to 20 degrees, and by about 25 to 50% for skew angle between 30 and 50 degrees. Adding two railings, AASHTO LRFD overestimation of the FEA moments increases with the increase in skew angle from about 20% for straight bridges with no skewness to about 50% for bridges with skew angle of 50 degrees.

It is worth noting that generally AASHTO overestimation is higher for three-lane bridges as compared to four-lane bridges except for high skew angles of 40 to 50 degrees when the results of three- and four-lane bridges become similar.

6. FEA RESULTS OF SKEWED BRIDGES WITH RAILINGS VS. REFERENCE BRIDGES

The maximum slab bending moments are summarized in Table 2 for all bridge cases in terms of ratios of FEA results for skewed bridges with railings to the corresponding reference straight bridges without railings. Table 2 shows that the maximum longitudinal slab moment reduces with the increase in skew angle and it is more pronounced for bridges with two railings. For angle of skewness less than 20º, the reduction in the moment reaches 30% for bridges with two railings, and about none for bridges with no railing. For angle of skewness equal to 20º, the reduction in the moment reaches 30% for bridges with two railings, and about 10% for bridges with no railing. For angle of skewness equal to 30º, the reduction in the moment reaches 40% for bridges with two railings, and about 25% for bridges with no railing. For angle of skewness equal to 40º, the reduction in the moment reaches 50% for bridges with two railings, and about 40% for bridges with no railings. For angle of skewness equal to 50º, the reduction in the moment reaches 65% for bridges with two railings, and about 55% for bridges with no railing.

It is worth noting that generally the reduction in FEA moments due to skewness and railings is slightly affected by the number of lanes or span lengths considered.

7. SUMMARY AND CONCLUSIONS

AASHTO Standard [1] and AASHTO LRFD [2] 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, and only AASHTO LRFD considers skewness to reduce the designs slab moments.

A parametric study using finite-element analysis was performed to investigate the influence of railings and skewness on the longitudinal slab moments in imply-supported, one-span, three- and four-lane concrete slabs bridges.

Table 1 Comparison of FEA maximum moments and AASHTO moment

Number of Lanes	Span Length (m)	FEA Maximum Longitudinal Moments and LRFD Moments (KN-m/m)						AASHTO Standard (KN-m/m)
		0 Degrees			10 Degrees			
		R0	R2	LRFD	R0	R2	LRFD	
3	7.2	98	78	102	96	77	102	97
	10.8	169	129	190	167	128	190	146
	13.8	238	192	272	234	190	272	186
	16.2	293	249	347	286	244	347	226
4	7.2	100	86	97	99	84	97	97
	10.8	176	142	180	174	141	180	146
	13.8	249	208	269	244	205	269	186
	16.2	305	267	347	298	261	347	226
Number of Lanes	Span Length (m)	FEA Maximum Longitudinal Moments and LRFD Moments (KN-m/m)						AASHTO Standard (KN-m/m)
		20 Degrees			30 Degrees			
		R0	R2	LRFD	R0	R2	LRFD	
3	7.2	90	71	98	76	60	91	97
	10.8	152	119	183	130	103	171	146
	13.8	213	175	261	181	151	245	186
	16.2	261	225	333	223	194	312	226
4	7.2	91	73	93	76	60	87	97
	10.8	158	127	173	132	106	162	146
	13.8	221	186	258	186	157	242	186
	16.2	271	239	333	229	203	312	226
Number of Lanes	Span Length (m)	FEA Maximum Longitudinal Moments and LRFD Moments (KN-m/m)						AASHTO Standard (KN-m/m)
		40 Degrees			50 Degrees			
		R0	R2	LRFD	R0	R2	LRFD	
3	7.2	63	51	86	48	39	77	97
	10.8	103	84	160	80	63	143	146
	13.8	144	123	228	111	92	204	186
	16.2	180	158	292	138	119	260	226
4	7.2	63	51	81	48	39	72	97
	10.8	104	85	151	80	63	135	146
	13.8	146	125	226	110	91	202	186
	16.2	183	162	292	138	120	260	226

Table 2 Ratio of FEA maximum moments to reference bridge cases

Number of Lanes	Span Length (m)	Ratio of FEA Maximum Longitudinal Moments to Reference Straight Bridge Cases without Railing					
		0 Degrees		10 Degrees		20 Degrees	
		R0	R2	R0	R2	R0	R2
3	7.2	1.00	0.80	0.98	0.79	0.92	0.73
	10.8	1.00	0.77	0.99	0.76	0.90	0.71
	13.8	1.00	0.81	0.98	0.80	0.89	0.73
	16.2	1.00	0.85	0.98	0.83	0.89	0.77
4	7.2	1.00	0.86	0.98	0.84	0.91	0.73
	10.8	1.00	0.81	0.99	0.80	0.90	0.72
	13.8	1.00	0.84	0.98	0.82	0.89	0.75
	16.2	1.00	0.88	0.98	0.86	0.89	0.78

Number of Lanes	Span Length (m)	Ratio of FEA Maximum Longitudinal Moments to Reference Straight Bridge Cases without Railing					
		30 Degrees		40 Degrees		50 Degrees	
		R0	R2	R0	R2	R0	R2
3	7.2	0.77	0.61	0.64	0.52	0.49	0.40
	10.8	0.77	0.61	0.61	0.50	0.47	0.38
	13.8	0.76	0.63	0.61	0.52	0.47	0.39
	16.2	0.76	0.66	0.62	0.54	0.47	0.41
4	7.2	0.76	0.60	0.62	0.51	0.48	0.39
	10.8	0.75	0.60	0.59	0.48	0.45	0.36
	13.8	0.75	0.63	0.59	0.50	0.44	0.37
	16.2	0.75	0.67	0.60	0.53	0.45	0.39

The bridge parameters considered were the span length, number of lanes, railings on both edges, and skew angle. The FEA moments were assessed with AASHTO procedures and with reference bridge cases without railing. The study concluded that the presence of railings increases the load carrying capacity of the bridges if they are modeled as integral parts of the slab, and that the slab moments reduce with the increase in the increase of skew angle. Further the influence of railings and skew angle is cumulative and becomes significant with high skewness and two railings. It can also be noted that integral railings can be used as one alternative strengthening technique to upgrade already existing bridges that require rehabilitation or upgrading, or if heavier loads are foreseen.

8. ACKNOWLEDGMENT

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