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LIVE LOAD DISTRIBUTION IN COMPOSITE PLATE GIRDER SKEW BRIDGES

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ABSTRACT

Current international specifications provide live load distribution factors for composite bridges. The factors do not account for change in straining actions due to skew angle, cross diaphragm presence, and continuity, consequently, resulting in inappropriate design. The objective of this paper is to evaluate the effects of the above parameters on the distribution factors in skew composite bridges. A parametric study was conducted on two-equal span bridge models utilizing the finite element method. The models were subjected to AASHTO standard lane loading and the corresponding concentrated loads positioned to maximize span and support moments with resulting girder moments being used to compute the distribution factors. The results indicated that the skew angle affected the distribution factors more significantly than the cross diaphragms. In addition, the negative moments distribution factors are 10% to 20% higher than positive moment factors. Based on the results, it was suggested to increase the AASHTO LRFD factors by 10%. An innovative technique was presented to modify the AASHTO LRFD factors to account for diaphragm presence. The technique estimates an equivalent slab thickness with a comparable effect to that of the diaphragms.

Keywords: Composite Bridges-Plate Girder Bridges-live load-distribution factors-skew-diaphragms-bracing.

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INTRODUCTION

Since the beginning of the twentieth century, composite bridges have been undergoing a steady development in design and construction. Among the most popular forms of composite bridges are the composite concrete deck-steel plate girder bridges. In a typical plate girder bridge, the reinforced concrete slab acts as a continuous deck over the plate girders, which carry the loads to the abutment and piers. Cross bracings, or alternatively diaphragms, might consist of I-shaped rolled or built-up sections (see Figure 1), or instead, they might be K- or X-shaped truss systems (Figures 2 and 3, respectively). Cross diaphragms provide temporary supports for the steel girder compression flanges during construction and enhance the transverse stiffness. Further, they are essential at abutments and piers to transmit superstructure horizontal forces to the substructure. In addition, diaphragms distribute eccentric vertical loads laterally among the girders (Mertz 1996). Occasionally, bridge surroundings including natural or man-made obstacles might enforce using skew configuration with the longitudinal axis (usually the same as the traffic direction) forming an acute angle with the piers and abutments (see Example in Figure 1). Examples of these obstacles are complex intersections, space limitations, or mountainous terrain. Skew bridges might have single or multiple spans with the latter being the most common.

Figure 1 Skew composite plate girder bridge with I-shaped diaphragms.

Figure 2 Skew composite plate Girder bridge with K-shaped diaphragms.

Figure 3 Composite plate girder bridge with X-shaped diaphragms.

PREVIOUS INVESTIGATIONS

Several researches were conducted on the subject of load distribution in skew composite bridges. Khaleel and Itani (1990) revealed that for a skew angle of 60°, the maximum bending moment in the interior girders is approximately 71% of that in a normal bridge, whereas, the reduction in maximum bending moment is only 20% in the exterior girders.

Ebeido and Kennedy (1996) presented reaction and shear distribution factors for two-span continuous skew composite bridges subjected to AASHTO standard truck loading. The bridges were assigned partial loading in case of calculating maximum positive moment and full loading in case of calculating maximum negative moment. They revealed that the presence of the skew causes significant reductions in both girder span and support moments. They observed that the skew angle has insignificant effect on the distribution factors for angles less than 30°. However, the distribution factors changed significantly with skew angles over 30°. Moreover, it was concluded that the span length of the bridge had a major effect on its load distribution characteristics. Its effect was best reflected by the bridge aspect ratio defined by the span length/bridge width.

Zokaie (2000) presented the research work on which the AASHTO LRFD (American 1998-1) distribution factors are based. The work assumed equal girder spacing, uniform girder cross sections, and constant angle of skew. The research did not incorporate the effect of cross diaphragms nor the continuity.

Barr et al. (2001) experimentally determined the live load distribution factors in prestressed girder bridges. Discrepancies up to 28% were recorded among the experimental results and the AASHTO LRFD distribution factors. It was concluded that end diaphragms, skew angle and the load type significantly affected the distribution factors. On the other hand, continuity and intermediate diaphragms were less significant.

LIVE LOAD DISTRIBUTION FACTORS IN AASHTO SPECIFICATIONS

AASHTO Standard Specifications

To simplify the design process, AASHTO Standard Specifications (American 1998-2) provide live load distribution factors. These factors reduce bridge design from a three-dimensional problem to a two-dimensional one. The longitudinal girder straining actions are obtained by multiplying the straining actions from a two-dimensional beam model loaded with half-axle truck loads by a distribution factor. The live load distribution factor is obtained by dividing the longitudinal girder spacing in feet, S, by a constant, D, depending on the number of lanes and girder type whether interior or exterior. In composite plate girder bridges with concrete deck slab, the following empirical formulas for distribution factors are suggested for interior girders with one or more traffic lanes, respectively.

$$\text{D.F.} = \frac{S}{7} \tag{1}$$

$$\text{D.F.} = \frac{S}{5.5} \tag{2}$$

For exterior girders, the distribution factor for one-traffic lane is equal to,

$$\text{D.F.} = \frac{S}{4.0 + 0.25S} \tag{3}$$

To account for the improbability of having all lanes loaded simultaneously, the specifications impose reduction factors for live load moments and shears for multi-presence of vehicles in different lanes. Full distribution factors are dedicated for one and two lanes; whereas, a 10% reduction factor is applied for three loaded lanes and 25% reduction factor is applied for four or more loaded lanes.

AASHTO LRFD Specifications

The AASHTO LRFD Specifications (American 1998-1) is more developed than its precedent (the AASHTO Standard Specifications). The specifications introduced distribution factors for straight bridges depending on the properties of the longitudinal girders and the deck slab. The specifications accredit the impact of the ratio of the bridge longitudinal stiffness to its transverse stiffness in determining the live load distribution factors. For one design lane loaded, the live load distribution factor for computing moments, for interior girders, is,

$$\mathrm{D.F.} = 0.06 + \left(\frac{\mathrm{S}}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{k_g}{Lt_s^3}\right)^{0.1} \tag{4}$$

For two or more loaded lanes, the distribution factor becomes,

$$\mathrm{D.F.} = 0.075 + \left(\frac{\mathrm{S}}{2900}\right)^{0.6}\left(\frac{\mathrm{S}}{\mathrm{L}}\right)^{0.2}\left(\frac{\mathrm{k_g}}{\mathrm{Lt_s^3}}\right)^{0.1} \tag{5}$$

With

$$\mathrm{k_g} = \mathrm{n}\,(\mathrm{I} + \mathrm{A}\,\mathrm{e_g^2}) \tag{6}$$

$$\mathrm{n} = \frac{\mathrm{E_B}}{\mathrm{E_D}} \tag{7}$$

Where			
	S	=	Spacing of beams (mm),
	t_s	=	Depth of concrete slab (mm),
	L	=	Span of beam (mm),
	N_b	=	Number of beams,
	k_g	=	Longitudinal stiffness parameter (mm^4),
	A	=	Area of non-composite beam (mm^2),
	I	=	Moment of inertia of non-composite beam (mm^4),
	E_B	=	Modulus of elasticity of the beam material (MPa),
	E_D	=	Modulus of elasticity of the deck material (MPa), and
	e_g	=	Distance between center of the beam and the deck (mm).

The distribution factors in Equations 4 and 5 are valid for: (1) $1100 < S < 4900$ mm, (2) $110 < t_s < 300$ mm, (3) $6000 < L < 73000$ mm, and (4) $N_b \geq 4$.

For exterior girders, the specifications require using the lever rule in case of one loaded lane. For two or more loaded lanes, the specifications give a modification factor for Equation 5 as follows,

$$\mathrm{D.F.} = e\,\mathrm{D.F.}_{\mathrm{Interior}} \tag{8}$$

$$e = 0.77 + \frac{d_e}{2800} \tag{9}$$

Where d_e = distance from exterior web of exterior beam to the interior edge of the road curb. However, the specifications require the distribution factors for exterior girders not to be less than that which would be obtained assuming rigid cross sections if cross diaphragms are present. For skew bridges, a reduction factor, r, is then applied to the straight bridge distribution factors.

$$r = 1 - c_1 \left(\tan\theta\right)^{1.5} \tag{10}$$

Where

$$c_1 = 0.25\left(\frac{k_g}{Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} \tag{11}$$

And θ = Angle of skew and $30° \leq \theta \leq 60°$.

The applicability range is for: (1) 1100 < S < 4900 mm, (2) 6000 < L < 73000 mm, and (3) $N_b \geq 4$. No reduction factors are applied for angles of skew less than 30°. On the other hand, bridges with angle of skew greater than 60° have the same reduction factors as those with skew angle of 60°. It should be noted that the AASHTO LRFD factors are fractions of the truck load (not fractions of half-axle truck loads). To be compared with those of AASHTO Standards, they should be multiplied by two, which is done in the rest of this paper.

SIGNIFICANCE OF THE CURRENT WORK

The previous investigations did not provide quantified information regarding the effect of cross diaphragms and continuity on load distribution factors. On the other hand, the AASHTO Standard and AASHTO LRFD Specifications ignore the contribution of the intermediate cross diaphragms and continuity in distributing the live load. The AASHTO Standard Specifications do not consider the alteration in girder moments due to skew as well. This might lead to inappropriate bridge design. Therefore, the current work investigates the effect of the skew, continuity, and cross diaphragm presence on the live load distribution characteristics in composite bridges through a theoretical study utilizing the finite element method. Only bridges having X-shaped cross diaphragms are considered in the current work.

THEORETICAL INVESTIGATION

Analyzed Bridges

The commercial finite element software package, SAP2000 Version 7.40 (SAP2000-1997), was utilized in the study. Two equal-span continuous bridges with X-type diaphragms with skew angles of 0°, 30°, 45°, and 60° were analyzed. The bridge spans varied between 30 and 60 meters and its width was changed to accommodate two and three lanes with distance between curbs of 9.00 meters and 11.0 meters, respectively. The number of longitudinal girders was 3, 4, and 5, for the two-lane bridges and 4, 5, and 6, for the three-lane ones. End diaphragms were provided over piers and abutments. The effect of intermediate diaphragms was examined by analyzing three cases: (1) bridges with no intermediate cross diaphragms, (2) bridges with intermediate cross diaphragms spaced at 10 meters and (3) bridges with intermediate cross diaphragms spaced at 5 meters. All girder bearings restrained the vertical movements; whereas over the piers (interior supports of the bridge), only two particularly selected interior bearings restrained the lateral and longitudinal movement to minimize the stresses resulting from thermal movement of the bridge. The dimensions of the plate girders and the thickness of the deck slab matched those in constructed bridges (Khalil 1998).

Bridge Models

Several bridge idealization techniques were considered (Bassyouni 2003). After carefully investigating the models in previous investigations and conducting a comparison between them, it was decided to utilize a model having a cross section as shown in Figure 4 as it provides both reasonable accuracy and computational efficiency. The steel girders were idealized using three dimensional beam elements with six degrees of freedom per node located at the plate girder center of gravity. The concrete deck slab was modeled using linear elastic quadrilateral shell elements having six degrees of freedom per node with identical elastic flexural rigidities in orthogonal directions. To account for deck slab cracking in the negative moment regions over intermediate supports, the deck slab was assigned a reduced stiffness (75% of the crack-free stiffness) for a distance of a quarter of the span length. To ensure reasonable analysis results, element sizes and lengths were selected based on a mesh sensitivity study (Fayed 2002). The bridge skew configurations mandated using skewed elements. The beam elements were connected to the shell elements through two rigid links: the first from the steel girder center of gravity to the lower surface of the slab, and the second from the lower slab surface to the slab center of gravity. The adopted diaphragm section was single angles 100 x 100 x 10 mm. In the models, the diaphragm elements had axial stiffness only (truss elements) with both tension and compression capabilities.

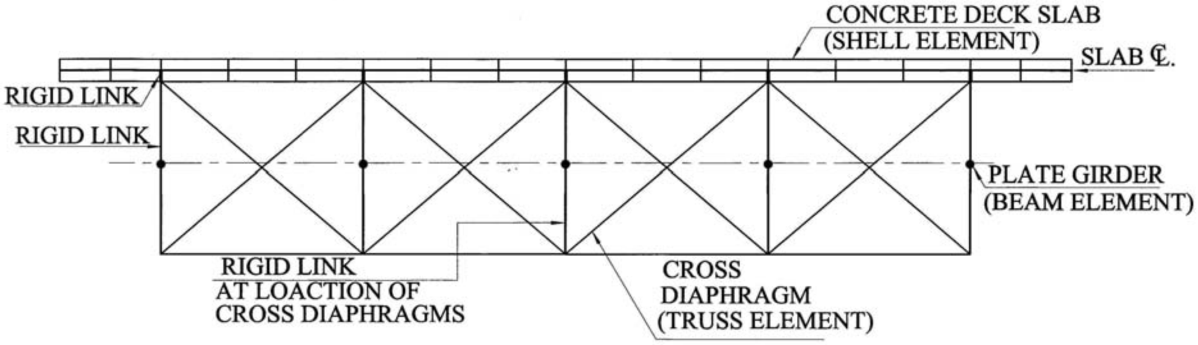


Figure 4 Idealized bridge cross section.

Bridge loads

All models were loaded with the AASHTO lane loading along with the corresponding concentrated loads positioned at locations producing maximum positive and negative moments, respectively. These locations were identified using influence area diagrams. The AASHTO lane loading consists of a uniform load of approximately 0.95 ton per linear meter of the lane. The lane width is approximately 3.65 meters. An accompanying one concentrated load of 8.20 ton per lane is used for positive moment computations and two concentrated loads each of 8.20 ton per lane are utilized for negative moment computations.

RESULTS AND DISCUSSION

The analysis outputs the bending moments for exterior and interior girders. Live load distribution factors for positive and negative moments at exterior and interior girders, respectively were determined as,

$$\text{D.F.} = \frac{M_t}{M_h} \qquad (12)$$

Where M_t = Maximum moment from the theoretical (finite element) analysis,

M_h = Maximum moment from a two-dimensional beam model loaded with half axle lane load.

Table 1 lists the resulting distribution factors for the 40-meter span bridge model. Results are listed for bridges with skew angles of 0°, 30°, 45°, and 60°, respectively. The table lists the computed factors for exterior and interior girders in both the positive and negative moments regions, respectively. Further, the table lists the load distribution factors for cases where cross diaphragms were provided at 10-meter and 5-meter spacing. Additionally, the table lists the distribution factors of AASHTO Standard Specifications and AASHTO LRFD Specifications, respectively. Although, AASHTO LRFD formulas are not applicable for three-girder bridges, they are listed in the table.

Table 1 Live load distribution factors for two equal 40-meter span bridge.

Skew Angle		0			30			45			60		
Number of girders		3	4	5	3	4	5	3	4	5	3	4	5
Girder spacing (meter)		3.6	2.4	1.8	3.6	2.4	1.8	3.6	2.4	1.8	3.6	2.4	1.8
P[a]	**Ext[c]-no[d]**	1.61	1.25	1.02	1.56	1.22	1.00	1.55	1.20	0.97	1.46	1.12	0.92
	Ext-10[e]	1.62	1.26	1.03	1.56	1.22	1.00	1.54	1.19	0.96	1.44	1.08	0.90
	Ext-5[f]	1.62	1.26	1.03	1.56	1.22	1.00	1.53	1.18	0.96	1.42	1.08	0.88
N[b]	**Ext-no**	1.71	1.33	1.10	1.69	1.35	1.11	1.69	1.34	1.11	1.64	1.31	1.10
	Ext-10	1.75	1.36	1.11	1.70	1.33	1.11	1.70	1.33	1.10	1.63	1.29	1.08
	Ext-5	1.76	1.37	1.12	1.70	1.31	1.11	1.70	1.32	1.09	1.63	1.28	1.07
AASHTO Standard-Ext		1.67	1.32	1.07	1.67	1.32	1.07	1.67	1.32	1.07	1.67	1.32	1.07
AASHTO LRFD-Ext		1.68	1.26	1.03	1.63	1.23	1.01	1.56	1.18	0.98	1.40	1.09	0.91
P[a]	**Int[g]-no**	1.39	1.18	1.02	1.35	1.17	1.01	1.35	1.13	0.99	1.25	1.06	0.92
	Int-10	1.36	1.17	1.02	1.33	1.16	1.01	1.33	1.12	0.98	1.24	1.05	0.91
	Int-5	1.35	1.17	1.02	1.33	1.15	1.00	1.33	1.11	0.97	1.25	1.04	0.90
N[b]	**Int-no**	1.67	1.36	1.15	1.52	1.31	1.11	1.54	1.26	1.07	1.43	1.20	0.97
	Int-10	1.57	1.33	1.14	1.50	1.30	1.12	1.53	1.26	1.08	1.43	1.20	0.98
	Int-5	1.53	1.32	1.14	1.48	1.30	1.12	1.52	1.26	1.08	1.43	1.19	0.99
AASHTO Standard-Int		2.15	1.44	1.07	2.15	1.44	1.07	2.15	1.44	1.07	2.15	1.44	1.07
AASHTO LRFD-Int		1.54	1.15	0.95	1.49	1.12	0.93	1.43	1.09	0.90	1.28	1.00	0.84

[a] positive moment regions

[b] negative moment regions

[c] External girder

[d] no intermediate diaphragms

[e] spacing between intermediate diaphragms = 10 meters

[f] spacing between intermediate diaphragms = 5 meters

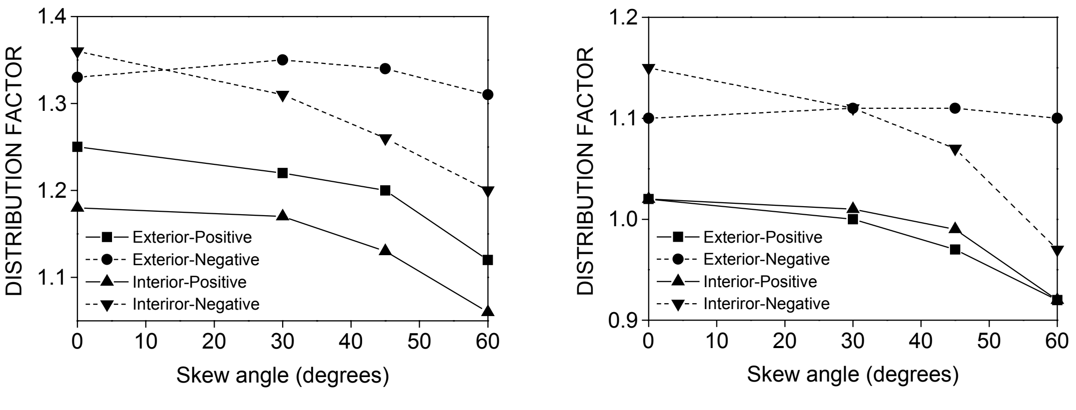
[g] Interior girder

Under the same conditions, (positive or negative moment, same angle of skew), the distribution factors increased as the number of carrying girder was decreased or the spacing between the longitudinal girders was increased. This is in line with the AASHTO Standards formula in which the distribution factors increase linearly with the increase of girder spacing.

Figure 5 shows the effect of the skew angles on the distribution factors for positive moment and negative moment distribution factors for the cases of four and five girders, respectively. From this figure and from Table 1, it is clear that the negative moment distribution factors are always higher than the corresponding positive moment distribution factors. In this respect, inspecting the values listed in Table 1 shows that the ratio of the negative moment to the positive moment factors ranged from 106% to 120% for exterior girders. This ratio increased with the increase in the skew angles as the average ratios (for different diaphragm configurations) were 108%, 110, 113%, and 118% for skew angles of 0°, 30°, 45°, and 60°, respectively.

For interior girders, on the other hand, the ratio of the negative moment factors to the positive moment factors ranged from 110% to 120%. However, with the higher ratios at lower skew angles. In this regard, the average ratios for different diaphragm configurations and girder spacing were 114%, 113%, 112%, and 112%, for skew angles of 0°, 30°, 45°, and 60°, respectively. It should be noted that stresses at or near an internal bearing are reduced due to the fanning of the reaction force at these locations with reductions reaching as high as 10% (AASHTO-1998-1). Obviously, these reductions cancel portions of the increased negative moment factors.

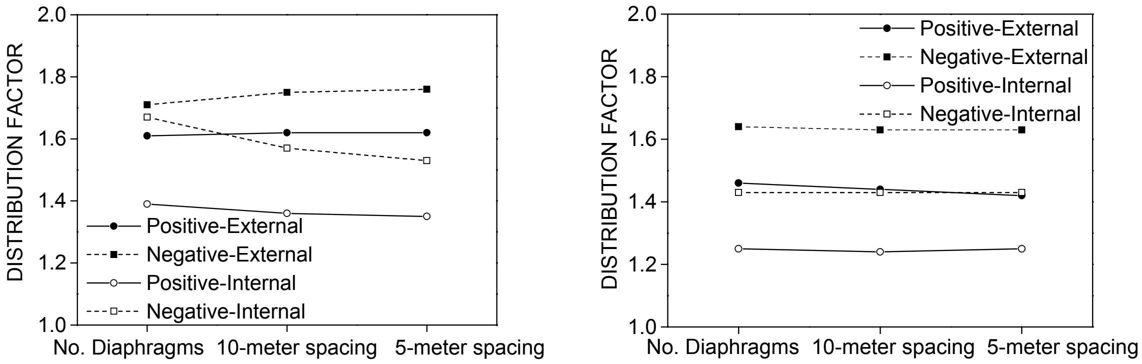
Additionally, the same figure showed that the distribution factors generally decreased with the increase of skew angles. Generally, it was noticed that increasing skew angles up to 30° reduced the distribution factors slightly. As the skew angle was increased over 30°, the reduction in the live load distribution factors was more pronounced. One exception is for the case of negative moment distribution factors in exterior girders. In this case, when the skew angle was increased from 0° to 30° the distribution factors slightly increased with minor changes in the distribution factors at higher skew angles. The reduction in moment distribution factors for interior girders is attributed to the reduction of the effective span of the bridge due to the minor presence of the two-way plate action.



4 girders **5 girders**

Figure 5 Effect of skew angle on distribution factors.

Figure 6 shows the effect of cross diaphragms on the negative and positive moment distribution factors of external and interior girders for angles of skew 0°, and 60°, respectively when the bridge cross section is composed of three girders. Clearly, the presence of cross diaphragm changed the load distribution characteristics among girders. Cross diaphragms enhance the bridge transverse rigidity, and hence, increase the load factors of exterior girders. On the other hand, changes in load distribution characteristics are highly pronounced when moving from bridge models with no intermediate diaphragms to those with diaphragms spaced at 10 meters. Additional minor changes occur when the spacing between cross diaphragms is further reduced to 5.00 meters signifying that transverse stiffness are not highly altered when diaphragm spacing is within 10 meters.



0° skew angle **60° skew angle**

Figure 6 Effect of diaphragms on distribution factors.

Figure 7 shows the positive and negative moments distribution factors for a bridge with four beams in the cross section for interior girders. Figure 8 is similar, however, for the exterior girders. Moreover, both figures show the AASHTO Standards and AASHTO LRFD factors for the corresponding bridges.

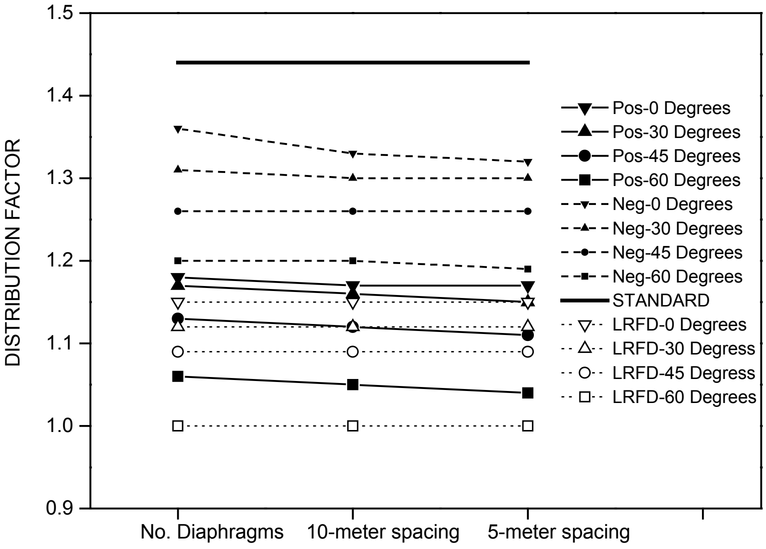


Figure 7 Distribution factors for interior girders (four girders-40 meter span bridge).

It is obvious from Figures 6 and 7 that the aforementioned discussion regarding the effect of diaphragms is also applicable for the four-girder bridges. Figure 7, further, shows that AASHTO Standard Specifications provided conservative estimates for the distribution factors in interior girders for all conditions (different skew angles, with or without diaphragms, and negative and positive moments). The ratio of overestimating the factors reaches a maximum of 74% (at 60° skew angle and 3 girders). For the positive moment distribution factors in exterior girders as seen in Figure 8, AASHTO Standard Specifications provide less conservative factors as the maximum ratio of overestimating the factors was 21%, (at 60° skew angle and 4 girders). In contrast, AASHTO Standard factors are slightly not conservative with respect to negative moments in exterior girders especially at low skew angles, as the theoretically computed factors reach as high as 105% those of the AASHTO Standards.

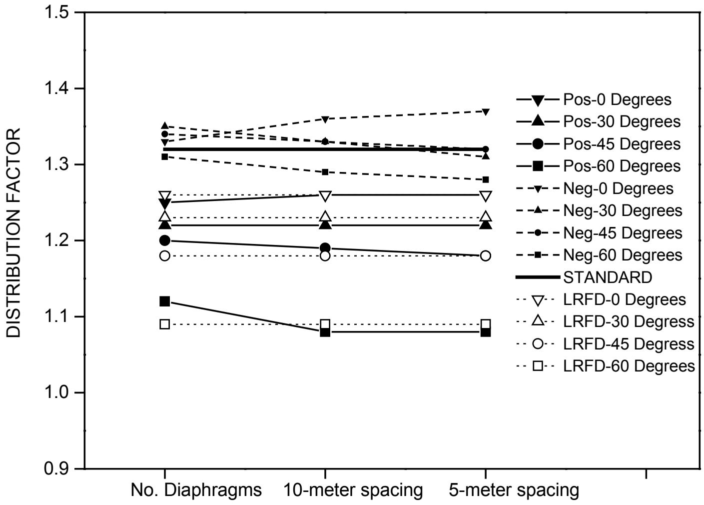


Figure 8 Distribution factors of exterior girders (four girders-40 meter span bridge).

AASHTO LRFD Specifications factors provided closer estimates for the distribution factors even for skew bridges for both positive and negative moments. In some instances, the LRFD factors were not conservative for the positive moment computations. In these cases, the factors undervalue the moments by a maximum of 11%. In contrast, the LRFD factors consistently underestimate the negative moment values for both exterior and interior girders. Ratios of underestimation were generally higher at high skew angles with a maximum of 22%. It should be noted that this ratio (22%) is significantly higher than the 10% reduction claimed to occur by the LRFD Specifications by the fanning action. Moreover, it seemed that the specifications overestimated the effect of skew on reducing the moments.

All the above discussions apply to all bridges in the current investigation and are not limited to the 40-meter span bridges as documented in (Bassyouni 2003). In this respect, Table 2 shows a comparison between the computed factors for the 30, 40, and 60-meter span bridges for skew angles of 30° and 60°, respectively. The table shows that the theoretically computed distribution factors changed only slightly with the span length variation and in line with those of the AASHTO Specifications.

Table 2 Distribution factors for the 30, 40, and 60-meter span bridges.

Skew Angle		30			60		
Bridge span		30	40	60	30	40	60
Girder spacing (meter)		2.4	2.4	2.4	2.4	2.4	2.4
P[a]	**Ext**[c]**-no**[d]	1.21	1.22	1.22	1.11	1.12	1.12
	Ext-10[e]	1.20	1.22	1.22	1.09	1.08	1.08
	Ext-5[f]	1.20	1.22	1.22	1.08	1.08	1.06
N[b]	**Ext-no**	1.32	1.35	1.37	1.26	1.31	1.35
	Ext-10	1.31	1.33	1.36	1.27	1.29	1.32
	Ext-5	1.30	1.31	1.34	1.28	1.28	1.31
AASHTO Standard-Ext		1.32	1.32	1.32	1.32	1.32	1.32
AASHTO LRFD-Ext		1.12	1.23	1.19	1.12	1.09	1.07
P[a]	**Int**[g]**-no**	1.18	1.17	1.14	1.05	1.06	1.06
	Int-10	1.17	1.16	1.13	1.04	1.05	1.05
	Int-5	1.15	1.15	1.12	1.03	1.04	1.04
N[b]	**Int-no**	1.28	1.31	1.34	1.18	1.20	1.29
	Int-10	1.27	1.30	1.34	1.18	1.20	1.24
	Int-5	1.26	1.30	1.32	1.17	1.19	1.21
AASHTO Standard-Int		1.44	1.44	1.44	1.44	1.44	1.44
AASHTO LRFD-Int		1.17	1.12	1.09	1.09	1.00	0.98

[a] positive moment regions

[b] negative moment regions

[c] Exterior girder

[d] no intermediate diaphragms

[e] spacing between intermediate diaphragms = 10 meters

[f] spacing between intermediate diaphragms = 5 meters

[g] Interior girder

PROPOSED CHANGES TO AASHTO LRFD

It has been shown that AASHTO LRFD factors are closer to those resulting from the theoretical investigation than those of AASHTO Standards. In the AASHTO LRFD formulation, effects of continuity and diaphragms were disregarded. To enhance the AASHTO LRFD formulas (Equations 4 and 5 in this paper), it is suggested to increase the distribution factors by 10%. Implementing this modification would almost eliminate the discrepancy between the results of the current study and those of AASHTO LRFD in the positive moment regions. In addition, the differences in the negative moment regions would be limited to approximately 10%, an accepted value considering the reductions in negative moments due to fanning effects at intermediate supports. To consider diaphragm presence, a further modification to AASHTO LRFD formula is applied through an innovative technique. The idea of that technique is to present the diaphragm in AASHTO equations as an extra deck slab thickness, thus, eliminating the need for three-dimensional modeling. The equivalent slab thickness is computed as in the following paragraph.

First, the cross correlation between the deflected shape of the cross section of a three-dimensional model of the bridge, x, with varying deck slab thickness without cross diaphragms and the deflected shape of a bridge cross section with infinitely rigid deck, y, is estimated. This is done under the action of an arbitrary load, say a concentrated load applied to the mid-span point of the exterior beam. The cross correlation coefficient, ρ_{xy} (Newland 1995), is,

$$\rho_{xy} = \frac{E[(x-m_x)(y-m_y)]}{\sigma_x \sigma_y} \tag{13}$$

Where E is the expectation, m_x is the average deflection of the cross section, m_y is the average deflection of the infinitely rigid cross section, and σ_x, σ_y are the corresponding standard deviations, respectively. The process is repeated for cases with intermediate diaphragms present, and similarly cross correlations are computed.

Figure 9 shows the relation between the deck slab thickness and the correlation factors for the case of a two 30-meter-span bridge with five girders in the cross section and zero skew angle with: (1) no intermediate diaphragms, (2) intermediate diaphragms spaced at 10 meters, and (3) intermediate diaphragms spaced at 5 meters. Cross correlations close to unity indicate high rigidity of the cross section. As can be seen, intermediate diaphragms enhanced the transverse stiffness, and, consequently increased the cross correlations. The enhancement is more pronounced in case of 5-meter spaced diaphragms. In this case, bridges act approximately as bridges having rigid cross section (correlation factor of approximately unity). Figure 10 is similar to Figure 9; however, for the case of bridge with a 60° skew angle. Comparing the two figures, it is apparent that skew angle affects the correlation factors slightly. Therefore, it is suggested to utilize the same graph (Figure 9) for straight and skew bridges.

As shown in Figure 9, a bridge with a 150-mm slab thickness with cross diaphragms spaced at 10 meters has a deflected transverse cross section with the same shape as that of a bridge having an equivalent 280-mm deck thickness. To compute the AASHTO LRFD factors for that bridge, the slab thickness in Equations 4 and 5 shall be 280 mm (not 150 mm). Several examples to verify the procedure were performed and excellent agreement resulted (Bassyouni 2003).

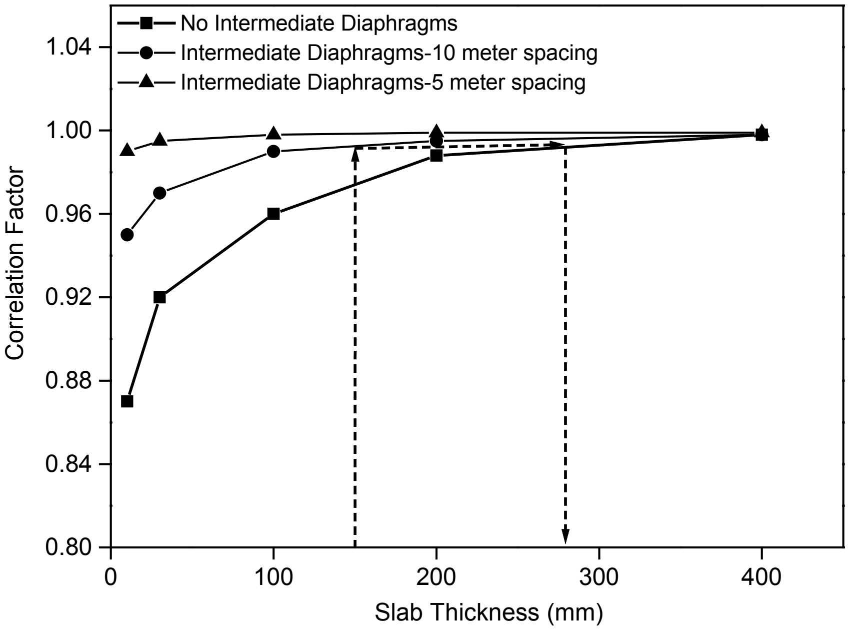


Figure 9 Relation between correlation factors and slab thickness for different number of intermediate diaphragms (five girders-zero skew angle).

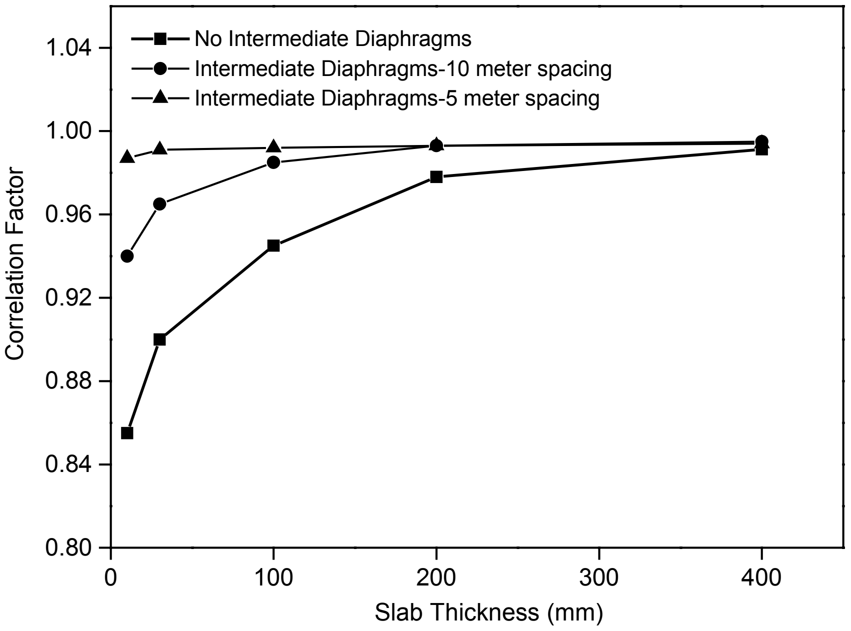


Figure 10 Relation between correlation factors and slab thickness for different number of intermediate diaphragms (five girders-skew angle = 60°).

SUMMARY AND CONCLUSIONS

In this paper, the live load distribution factors in AASHTO Standard and LRFD Specifications were reviewed. It was found that diaphragm contribution and continuity effects were disregarded in the LRFD Specifications. AASHTO Standard Specifications, further, ignore the effect of skew angles. A theoretical parametric study utilizing the finite element method was conducted on continuous bridges with several skew angles and diaphragm configurations. The bridges had two spans with span lengths varying between 30 and 60 meters. The study revealed that negative moment distribution factors were higher than the corresponding positive moment distribution factors. The ratio of the negative moment to the positive moment factors ranged from 106% to 120% for exterior girders and from 110% to 120% for interior girders. Further, the study showed that skew bridges had lower load distribution factors than those without skew. The effect of skew angles was more pronounced when the skew angles were higher than 30°. Introducing cross diaphragms enhanced the transversal stiffness, and hence, altered the load distribution characteristics in straight and skew bridges. For the studied cases, diaphragm spacing of ten meters had comparable effect to diaphragm spacing of five meters. AASHTO LRFD provided closer estimates for the distribution factor than AASHTO Standards, however, LRFD were not conservative in many cases. Thus, it was suggested to increase the LRFD factors by 10%. An innovative technique allowed using AASHTO LRFD formula while accounting for diaphragm presence. This was done by presenting the diaphragms with an extra deck slab thickness. That thickness was computed by correlating the deflection shape of bridge cross section to that of infinitely rigid cross section.

REFERENCES

Mertz, D. R. (1996), "Cross-Frame Diaphragms for Steel Girder Bridges Using the AASHTO LRFD Bridge Design Specifications", *Proceedings of the ASCE Structural Congress XIV*, ASCE, pp. 307-311, New York.

Khaleel, M. A., and R. Y. Itani (1990), “Live Load Moments for Continuous Skew Bridges” *Journal of Structural Engineering*, ASCE, Vol. 116, No. 9, page 2361-2373.

Ebeido, T., and J. B. Kennedy (1996), “Girder Moments in Continuous Skew Composite Bridges”, *Journal of Bridge Engineering*, ASCE, Vol. 1, No. 1, pages 37-45.

Zokaie, T. (2000), “AASHTO-LRFD Live Load Distribution Specifications”, *Journal of Bridge Engineering*, ASCE, Vol. 5, No. 2, pages 131-138.

American Association of State Highway and Transportation Officials (1998-1), *AASHTO LRFD Bridge Design Specifications, Customary US Units*, First Edition, Washington, D.C.: American Association of State Highway and Transportation Officials.

Barr, P. J., Eberhard, O., and J. F. Stanton (2001), “Live-Load Distribution Factors in Prestressed Concrete Girder Bridges”, *Journal of Bridge Engineering*, ASCE, Vol. 6, No. 5, pages 298-306.

American Association of State Highway and Transportation Officials (1998-2), *Standard Specifications for Highway Bridges*, Sixteen Edition, Washington, D. C., American Association of State Highway and Transportation Officials.

SAP2000 (1997), Integrated Finite Element Analysis and Design of Structures Package, Computers and Structures, Version 7.40, California, USA.

Khalil, A. H., Wipf, T., Greimann, L., and D. Wood (1998), “Evaluation of a Method to Prevent Fatigue Cracking at Diaphragm/Plate Girder Connections”, in the Proceedings of *First International Conference on Civil Engineering*, Helwan University, Egypt.

Bassyouni, G. F. (2003), *Live Load Distribution in Continuous Skew Steel Plate Girder Bridges*, M. Sc. Thesis, Ain Shams University, Cairo, Egypt.

Fayed, M. N., Mourad, S., Khalil, A. H., and G. Bassyouni (2002), “Diaphragm Effect on Live Load Distribution Factors in Skew Steel Plate Girder Bridges”, in the Proceedings of *Regional Conference on Civil Engineering*, ASCE-Egyptian Section, Cairo, Egypt.

D. E. Newland (1996), *Introduction to Random Vibrations, Spectral and Wavelet Analysis*, Addison-Wesley Publishing Company, 3rd Edition, England.