International Journal of Physical Sciences Vol. 7(6), pp. 966 - 973, 2 February, 2012
Available online at http://www.academicjournals.org/IJPS
DOI: 10.5897/IJPS11.1636
ISSN 1992 - 1950 © 2012 Academic Journals

Full Length Research Paper

Influence of skew abutments on behavior of concrete continuous multi-cell box- girder bridges subjected to traffic loads

Iman Mohseni[1]*, A. R. Khalim Rashid[2] and A. Akbar Pabarja[2]

[1]Department of civil engineering, Behbahan Branch, Islamic Azad University, Behbahan, Iran.
[2]Department of Civil Engineering, Universiti Kebangsaan Malaysia (The National University of Malaysia), 43600, UKM, Bangi, Selangor D.E., Malaysia.

Accepted 03 January, 2012

Current American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO-LRFD) provided several equations for distribution factor of moment and shear of multi-cell box-girder bridges. However, the equations involved a large number of parameters, but the preliminary study shows that it is not compatible with finite element results. In many cases, the change in straining actions due to skew angle, continuity and effect of diaphragms result in tremendous errors in the results. So, this study aims at evaluating the accuracy of these equations by doing a parametric study to determine the effectiveness of various parameters on lateral distribution of live loads. To this end, three samples of numerical model bridges were considered and analyzed by means of finite element method. The results indicate that the span length, number of box, number of lanes and web spacing significantly had an effect on the exterior and interior moment distribution factor of bridges. Proposed improved equations were deduced accordingly for AASHTO-LRFD, (2007) moment loads. Results from the proposed equations are consistent with numerically derived results from grillage and finite element method.

Key words: Multi-cell, box-girder, distribution factor, skews bridges, grillage method, truck load.

INTRODUCTION

The responses of a bridge under live load or live load distribution factor are important for both design and evaluation purposes since it enables the engineer to predict the strength and serviceability of bridges. However, determining the accurate maximum responses and load distributions is difficult due to the complexity of bridge structures. Nowadays, with advances in computer technology and modern finite element (FE) programs and user-friendly graphical interfaces, we can calculate the internal force and moment of all types of bridges. Codes of practice (AASHTO-LRFD, 2007; AASHTO, 2004; OHBDC, 1983) have recently recommended the concept of load distribution factor to simplify the analysis and design of concrete bridges. These codes suggest some methods to analyze bridges which include finite element method, grillage analysis, and empirical equation for live load distribution factors (LLDF). Distributions of traffic loads to different girders are not uniform and the girder close to the traffic load receives the largest portion of the loads. The LLDF was calculated (AASHTO, 2004; Barker and Puckett, 1997) from:

$$LLDF = \frac{F_{Refined}}{F_{Beamline}} \qquad (1)$$

Where $F_{Refined}$ corresponds to the largest live loads in girder from the refined methods; while $F_{Beamline}$ corresponds to maximum live loads (moment or shear) from a simple beam-line model subjected to one lane of traffic. Several investigations (Hughs and Idriss, 2006; Huo et al., 2003; Samaan and Sennah, 2002; Song et al., 2003; Zheng, 2008; Zokaie et al., 1993) have been carried out on the load distribution of box-girder bridges under wheel loads. The National Cooperative Highway Research Program (NCHRP) developed Association of

*Corresponding author. E-mail: mohseni1979@gmail.com. Tel: +6017 3367 502. Fax: +603 8921 5051.

State Highway and Transportation Officials Load Resistance Factor Design (AASHTO-LRFD 2007) equations for live load moment and shear distribution factor based on a study conducted by Zokaie et al. (1993). Unfortunately, Zokaie et al. (1993) did not provide sufficient details to justify and substantiate the accuracy of the proposed modification factor for continuity (Barr et al., 2001; Ebeido and Kennedy, 1996; Mabsout et al., 1999). Preliminary studies revealed that the new equations in AASHTO-LRFD (2007) for calculating distribution of live load under estimated results can lead in many cases, (Huo and Zhang, 2008). Thus, for a reasonable design of continuous multi-cell box-girder bridges, it is essential to obtain the maximum distribution factors for positive and negative stress, moment, deflection and stress. This paper presents the results of parametric studies on three continuous two span multi-cell box-girder bridges by examining different parameters such as span lengths, number of boxes, and number of traffic lanes. Correction factor equations for the purpose of design value were determined through statistical analysis and elastic response of bridge for AASHTO-LRFD (2007) truck loading to improve the estimation of internal and external load distribution of multi-cell box-girder bridges.

METHODOLOGY

Concrete box-girder bridge is the best choice due to its ability to construct bridges with long span, reducing the cost problems and aesthetic value (Suhatril et al., 2011). However, evaluating the general behavior of this type of bridge under live loads (traffic load and earthquake) is very important, specially, for skewed bridges that the live load distribution factor is still a controversial issue for them.

The objective of the present study is to evaluate the accuracy of distribution factors of current specifications for continuous multi-cell box-girder bridges. The primary study indicates that current codes often underrate or overrate the response of bridges under truck loads. Therefore, to make the equations more accurate, this study investigated the effect of divergent parameters on moment distribution factor of bridges by using a parametric approach on a large number of bridges.

The different parameters in this study include skew angle, span length, number of box and number of lane. The effect of the skewness on LLDF was evaluated. The modeling bridges were analyzed using finite element method and grillage method. Then, the analytical results are compared with AASHTO-LRFD (2007) and standard specifications for highway bridges (AASHTO, 2004) to investigate the accuracy of code's equations. At last, two correction expressions were proposed to improve the results of AASHTO LRFD distribution factor equations through statistical analysis.

Live Load Distribution Factor of Different Specifications for Multicell Bridges:

AASHTO Standard Specification

The distribution factors for the interior beams of cast-in-place concrete multicell beam bridges are calculated using the S-over equations as following:

For one lane loaded:

$$\mathrm{DF} = \frac{S}{8} \tag{1}$$

For two or more traffic lanes loaded:

$$\mathrm{DF} = \frac{S}{7.0} \tag{2}$$

AASHTO-LRFD FORMULAS

American Association of State Highway and Transportation officials load and resistance factor design (AASHTO-LRFD, 2007) proposed several equations to calculate the live load distribution factor for various types of bridges. The moment distribution factor equations of interior beams of multicell box- beam bridges are proposed as follow:

$$MDF = \left[1.75 + \frac{s}{3.6}\right]\left[\frac{1}{L}\right]^{.35}\left[\frac{1}{N_C}\right]^{0.45} \tag{3}$$

For two or more design lanes loaded:

$$MDF = \left[\frac{13}{N_C}\right]^{.3}\left[\frac{S}{5.8}\right]^{.35}\left[\frac{1}{L}\right]^{0.25} \tag{4}$$

For exterior beams, the AASHTO LRFD (2007) lists the moment distribution factor equations and ranges of applicability cast-in-place multicell beam bridges. The equation is:

$$MDF = \frac{W_e}{14} \tag{5}$$

Where S, N_C, L, W_e are spacing of beams (ft), number of cells, span length and half of web spacing plus the total overhang in feet, respectively.

For skew bridges, a reduction factor, SRF, must be applied to the straight bridge distribution factors are following:

$$SRF = 1.05 - 0.25 tang\theta \leq 1.0 \tag{6}$$

$$IF\ \theta > 60\ then\ use\ \theta = 60 \tag{7}$$

DESCRIPTION OF BRIDGE PROTOTYPES

The bridges selected for this study originated from four continuous multi-cell box-girder bridges in United States proposed by Huo et al. (2003) to examine the accuracy of Henry's methods for calculating LLDF. The properties of chosen bridges such as the number of spans, number of boxes (N_b) and number of lane loads (N_l) are shown in Table 1. Two bridges are right bridges and other are skewed bridges. The cross section and plan view of bridge No. 1 are shown in Figure 1. It is three span bridges with various span lengths. Previous studies revealed that secondary properties such as parapet and barriers have an insignificant impact on LLDF of bridges (AASHTO-LRFD, 2007; Huo et al., 2003). Therefore, this study did not include their effect into bridge modeling. Nevertheless, the end diaphragms included in the exterior and interior supports and intermediate diaphragms are located at spaces 30 feet based on AASHTO specification.

Table 1. Geometric of cast- in- place concrete multi-cell box-girder bridges.

No. bridge	No. span	Max. L	Girder spacing	Slab thickness	skew	Nc	Width
1	3	140.0	10.33	9.25”	26.23	4	50
2	2	110.0	9.50	8.00”	10.60	3	36
3	2	98.75	9.00	8.25”	0	4	44
4	2	133.83	9.25	8.00”	0	4	44

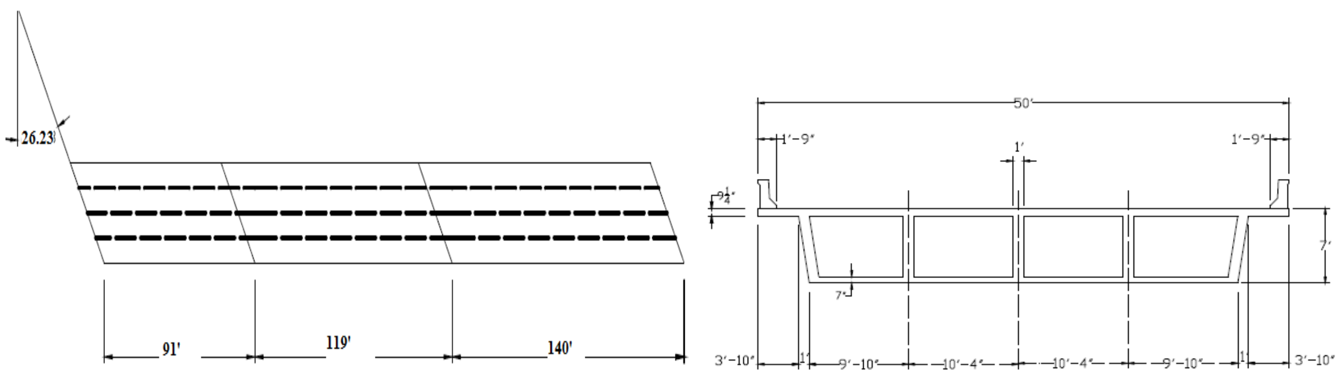

Figure 1. Cross section and plan view of bridge No.1.

MODELING

Finite element model

The CSIBRIDGE version 15 structural analysis software was used to model the superstructure of bridges and determine the responses of the bridges subjected to vehicle live loads. Bridge superstructures were typically modeled with frame elements for longitudinal and shell element transverse members (diaphragms) to form an integrated superstructure. The frame element includes the effects of biaxial bending, torsion, axial deformation and biaxial shear deformations.

Verification of finite element modeling

The finite element modeling in this study is validated by measurement results obtained by field testing on Tsing- Yi- South Bridges in Hong Kong (Ashebo et al., 2007a, b). The comparisons of bending moment, first natural frequencies and strain indicated good compatibility between numerical modeling and test. Based on this validation, the same final element modeling is provided to analyze the prototype bridge results.

Grillage model geometric

The non-orthogonal grillage method was used to determine the moment distribution factor of multi-cell box bridges. The cross section shown in Figure 1 is modeled with a set of longitudinal and transverse beam element. Figure 2 shows the placement of transverse and longitudinal grillage members adopted in this study. Longitudinal grillage beams are usually placed coincidently with webs of the actual structure and transverse medium including both the top and bottom flanges are represented by equally- spaced transverse grillage beams (Hambly, 1976; Jaeger and Bakht, 1982). Grillage analysis requires the calculation of the moment of inertia, I, torsional moment of inertia, J, and equivalent shear area of a transverse grillage member, a_s.

Hambly (1976) suggested the following equations for equivalent members' details of multi-cell box grillage member properties:

$$i = \frac{d' d'' h^2}{(d'' + d')} \quad \text{Per unit length} \qquad (12)$$

$$j = \frac{2h^2 d'' d'}{(d'' + d')} \quad \text{Per unit length} \qquad (13)$$

$$a_s = \frac{(d'^3 + d''^3)}{l^2}\left[\frac{d_W^3 * l}{d_W^3\, l + (d'^3 + d''^3)h}\right]\frac{E}{G} \quad \text{per unit width} \qquad (14)$$

Where d' and d" are the top and bottom slab and h' and h" are distances from their centroid of deck, d_w, L and h are web thickness, distance between two adjacent web and total depth of cells, respectively. For bridge No. 3, grillage properties are i_s= 2.10, j_s= 4.185 and a_s= 0.004.

For all prototype bridges, the first abutment is considered as a hinge with movement restrained in the X and Z directions. All other supports are treated as rollers and restrained only in the Z direction. X represents the longitudinal direction along the bridge, and Z shows the vertical direction. The bridges are loaded by AASHTO standard HS20-44 truck. For three-dimensional models, many trucks are placed on a bridge in the transverse direction depending on the width of the bridge to record the maximum response of bridges. The AASHTO Standard intensity reduction factors were used for three and four truckload results (0.9 and 0.75, respectively).

Verification of grillage bridge modeling

The non-orthogonal grillage model used in this study was verified

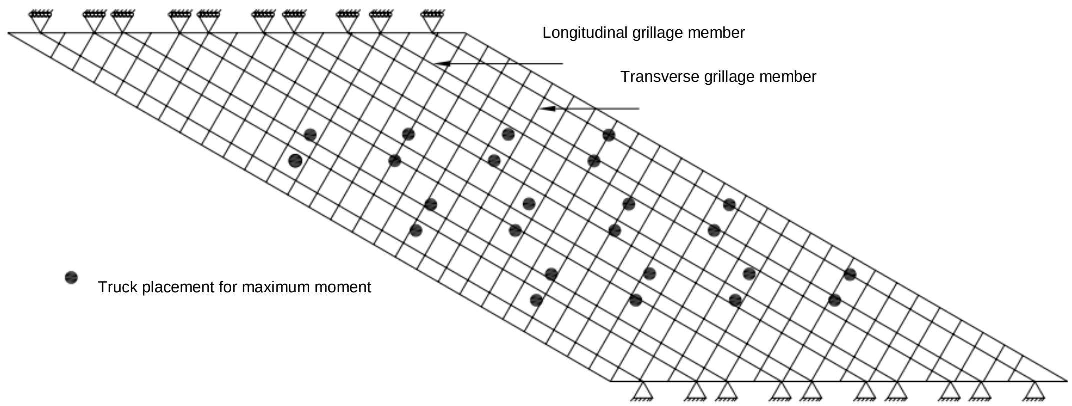

Figure 2. Application of design truck live load for maximum moment on grillage model.

Table 2. Maximum moment distribution factor for bridges 1 and 2.

No. bridge	Skew	Max. L	Beam	FEA	LRFD	Ratio FEA/LRFD	AASHTO specification	Ratio FEA/AASHTO
1	26.2	140	Exterior	0.490	0.616	0.795	0.678	0.722
	26.2	140	Interior	0.689	0.670	1.020	0.678	1.010
2	16.5	110	Exterior	0.570	0.616	0.920	0.678	0.841
	16.5	110	Interior	0.810	0.570	1.420	0.678	1.190
3	0.00	98.75	Exterior	0.415	0.580	0.715	0.678	0.612
	0.00	98.75	Interior	0.620	0.701	0.900	0.678	0.910
4	0.00	133.83	Exterior	0.409	0.616	0.664	0.678	0.603
	0.00	133.83	Interior	0.687	0.668	1.030	0.678	1.010

by modeling the multi-cell box-girder bridges of Example 5.5.1 and 5.5.3 of the study done by Hambly (1976) and comparing the obtained data of reaction, shear, moment and shear flow from grillage analysis and the publication. The trivial differences (less than 6%) between obtained data indicate that the grillage modeling technique is valid.

LOADING CONDITION

The live load moment in prototype bridges are obtained subjected to AASHTO standard HS20-44 truck loading. The HS20-44 AASHTO truck is a three axle truck that front axle weight is 10 kip and weight of other axles is 40 kip. The distance between front and second axle is 4.30 m and changes between 4.30 to 9 m to obtain maximum responses. In the case of live load moment for non-skewed bridges, once the location of the maximum moment was found with one truck, the additional trucks are placed alongside the first. For the skewed bridges, the maximum bending moment is obtained when the trucks are exactly located at the mid span of each lane.

RESULTS OF ANALYSIS AND COMPARISON

The analytical results and moment distribution factors obtained from finite element analysis and bridge code specifications are shown in Table 2. The maximum distribution factors of moment at each girder are determined by dividing the maximum moment of each bridge by the maximum moment of a single beam line analysis at the corresponding location.

Due to the difference in the cross section details of internal and external girder, even in the same bridge, the finite element results were less than the results of codes. On the contrary, the AASHTO specification often obtains very conservative results for bridges. The only bridge possessing the conservative distribution factor in interior beams has a skew angle of 26° and maximum span length of 140 ft. The longer span length and larger angle could be convincing reasons for the conservative distribution factor from AASHTO-LRFD standard.

Effect of span length on moment distribution factors

Figure 3 shows the moment distribution factor versus span length for exterior and interior beams of bridges. On one hand, the well-organized matches between external

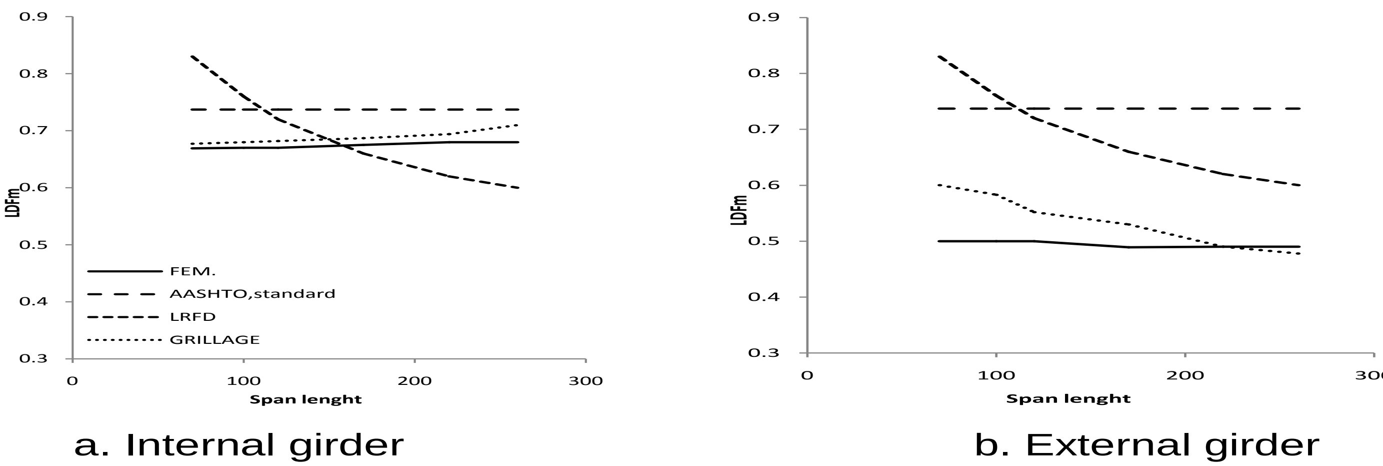

Figure 3. Moment distribution factor versus span length.

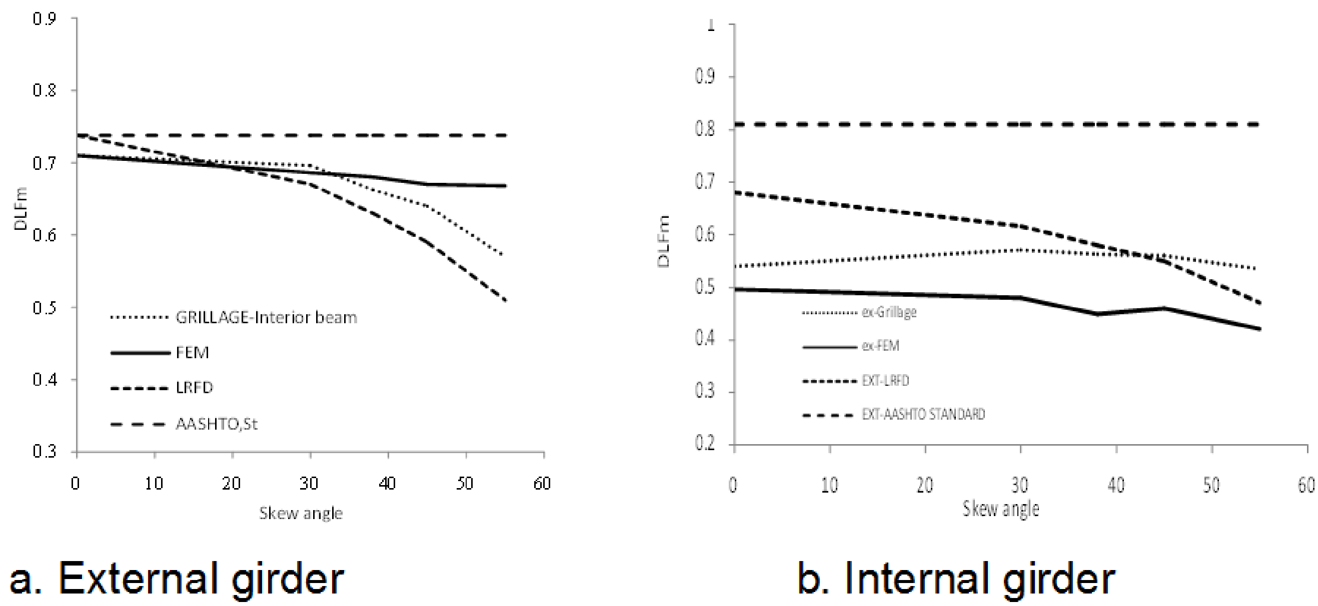

Figure 4. Moment distribution factor versus skew.

moment distribution factor from grillage and finite element methods can be observed. On the other hand, AASHTO standard specification and AASHTO-LRFD (2007) obtain very conservative results for interior and exterior distribution factor of multi-cell box-girder bridges. However, with exceeding the maximum span length from 170 ft, the distribution factor of moment from AASHTO-LRFD specification provides very underestimated results for exterior girder.

Effect of skew angle on moment distribution factors

The effect of skew angle on moment distribution factor of bridge No.1 from Table 1 is shown in Figure 4. The skew angle for bridges in this study ranged from 0 to 60°. By comparing the responses of various methods, it can be observed that all methods obtained very conservative moment distribution factors for exterior girders, while the AASHTO-LRFD (2007) attained it partly underestimated for skew angle more than 30°.

Figure 5 shows that the existence of skew angle leads to significant reductions in moment distribution factor in mid-span of bridges. It should be noted that moment distribution factor of internal girders are higher than those of external girder. On the other hand, the effect of skew angle on internal girders is more significant than external girders.

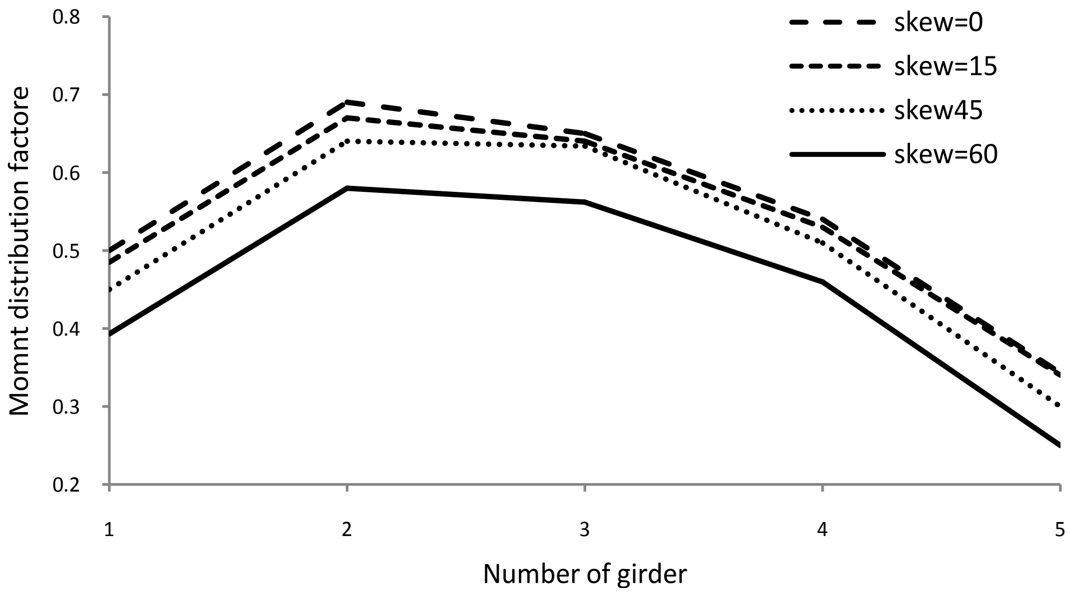

Figure 5. Moment distribution factor versus transverse location of beams.

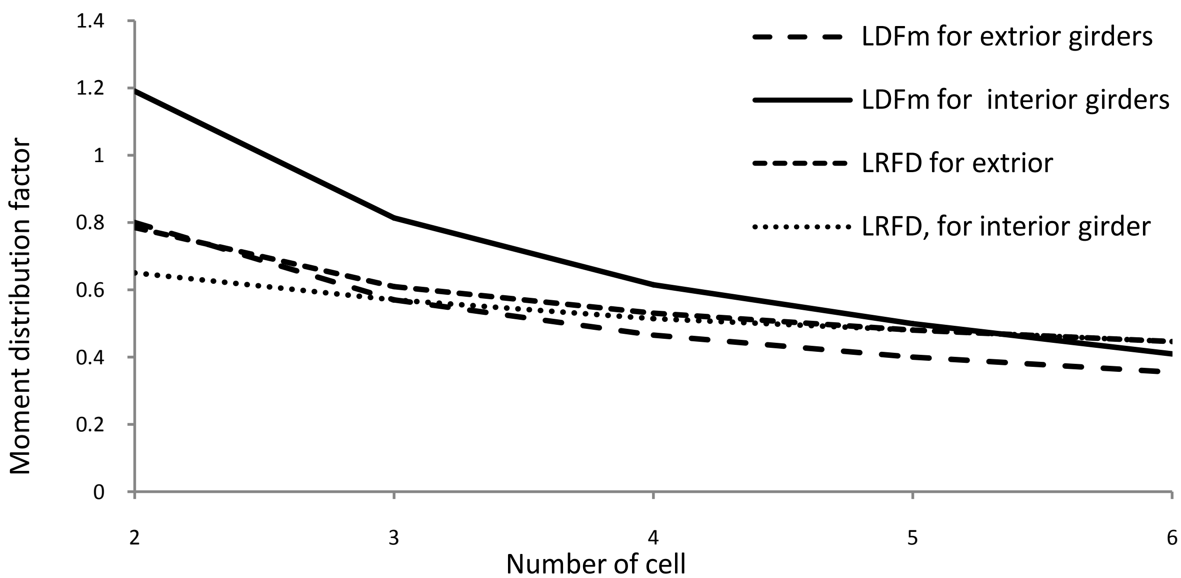

Figure 6. Moment distribution factor versus number of cells.

Effect of number of cells on moment distribution factor

The Figure 6 shows that the number of cells greatly affects the distribution factors of maximum positive moment. It can be observed that the number of cell (N_B) raises the distribution factor for maximum positive moment declines. However, AASHTO LRFD (2007) estimated higher values for interior girders than finite elements analysis; its estimation is fairly close to values of moment distribution factors of exterior girders.

Correction factors for estimating moment distribution factor of AASHTO LRFD

In this section, two correction factors are determined to multiply the moment distribution factor equations proposed in AASHTO LRFD (2007) for multi-cell box-girder bridges. For this purpose, the maximum distribution factor of each bridge is obtained though parametric analysis on 70 prototype multi-cell box bridges. The properties of prototype bridges are shown in Table 3. The Canadian Highway Bridge Design codes (CHBDC, 2000) are used to

Table 3. The cross sectional properties of prototype bridges.

Set	L	N_B	N_L	W	t1	t2	Skew (θ)
1	32, 47, 62, 77, 92	3	1, 2	9.00	0.18	0.15	0, 30, 45, 60
2	32, 47, 62, 77, 92	3, 4	1, 2, 3	12.50	0.22	0.15	0, 30, 45, 60
3	32, 47, 62, 77, 92	3, 4, 5	1, 2, 3, 4	18.20	0.24	0.15	0, 30, 45, 60

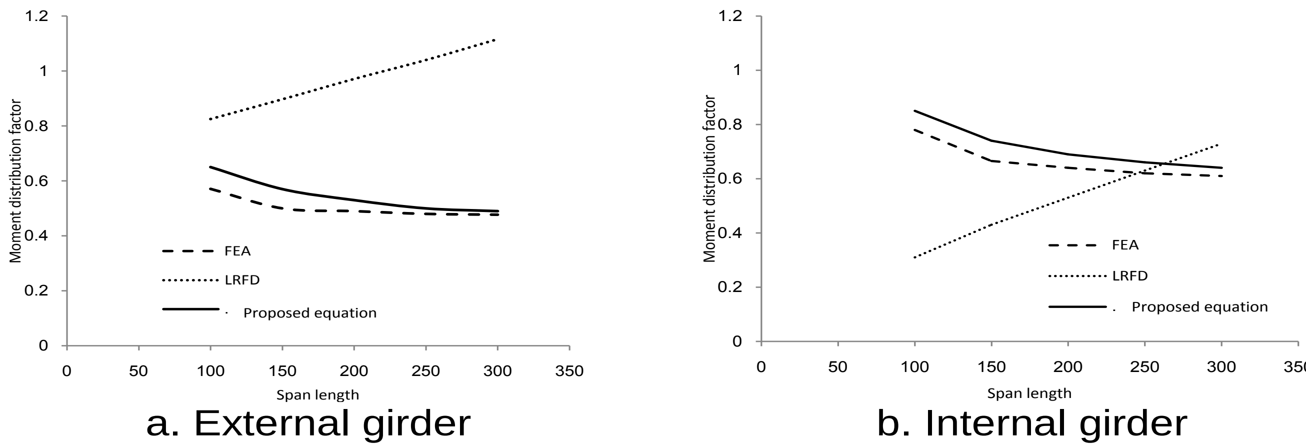

Figure 7. Verification of proposed correction factors for moment distribution factor equations.

determine these cross - sectional properties. Then, by using a statistical method (Dicleli and Erhan, 2009) such as the least squares method and best fit analysis of nonlinear data on obtained results, the following expressions are proposed for external and internal correction factors (CF_{EX} and CF_{IN}) of AASHTO-LRFD (2007) equations:

Internal girder:

$$CF_{IN} = 111.2\,\frac{1}{L^{0.9}} * \frac{1}{N_C^{0.5}} \quad (15)$$

$$MDF = \left[\frac{166}{N_C}\right]^{0.68} \left[\frac{S}{5.8}\right]^{.35} \left[\frac{1}{L}\right]^{0.79} \quad (16)$$

External girder:

$$CF_{EX} = 10.30\,\frac{1}{L^{0.36}} * \frac{1}{N_C^{0.80}} \quad (17)$$

$$MDF = 0.73[W_e]\left[\frac{1}{N_C}\right]^{.8} \left[\frac{1}{L}\right]^{0.36} \quad (18)$$

The accuracy of proposed equations should be validated. For this end, the internal and external moment distribution factors of bridge No. 4 (Table 1) are obtained by means of Equations 10 and 11 and compared with the results determined by finite element and grillage analysis and AASHTO- LRFD (2007) equations. As shown in Figures 7 a and b, the proposed equation is matched very well with finite element analysis.

CONCLUSION

The lateral load distribution in continuous multi-cell beam bridges was investigated by doing a parametric study on four prototype bridges. The superstructure of bridges was modeled by means of a finite element and grillage methods. It was also found out that the main parameters affecting the moment distribution factor are the span length, number of cells and skew angle of superstructure. The comparison of moment distribution factors of internal and external girder with current bride codes indicates that the AASHTO specification obtained very conservative results for both interior and exterior girders, and the AASHTO-LRFD determined the overrate bending moment for interior girder, while for exterior girder it obtained conservative results for short span and underrate results for medium and long span bridges.

The increasing skew angle greatly influences the bending moment of bridges and results in significant decrease in moment distribution factor of bridges.

REFERENCES

AASHTO LRFD (2007) bridge design specifications, American Association of State Highway and Transportation Officials, 4th Edition, Washington D.C.

AASHTO (2004). bridge design specifications, American Association of State Highway and Transportation Officials, Washington D.C.
Ashebo DB, Chan T, Yu L (2007a). Evaluation of dynamic loads on a skew box girder continuous bridge Part I: Field test and modal analysis. J. Struct Eng., 29(6): 1052-1063.
Ashebo DB, Chan T, Yu L (2007b). Evaluation of dynamic loads on a skew box girder continuous bridge Part II: Parametric study and dynamic load factor. J. Struct Eng., 29(6): 1064-1073.
Barker R, Puckett J (1997). Design of highway bridges: based on AASHTO LRFD. John Wiley & Sons. New York, N.Y.
Barr P, Eberhard M, Stanton J (2001). Live-load distribution factors in prestressed concrete girder bridges. J. of Bridge Eng., 6(5): 298-306.
CHBDC (2000). Canadian Highway Bridge Design Code. Canadian Standards Association (CSI), Toronto, Ontario, Canada, p. 734.
Dicleli M, Erhan S (2009). Live load distribution formulas for single-span prestressed concrete integral abutment bridge girders. J. Bridge Eng., 14(6): 472-294.
Ebeido T, Kennedy J (1996). Shear and reaction distributions in continuous skew composite bridges. J. Bridge Eng., 1(4): 155-165.
Hambly EC (1976). Bridge Deck Behaviour. 1st Edition. Chapman and Hall, London, p. 170.
Hughs E, Idriss R (2006). Live-load distribution factors for prestressed concrete spread box-girder bridge. J. Bridge Eng., 11(5): 573-581.
Huo X, Conner S, Iqbal R (2003). Re-examination of the Simplified Method (Henry's Method) of Distribution Factors for Live Load Moment and Shear. Final Report. Tennessee DOT Project No. TNSPR-RES 1218. Tennessee Technological University, Cookeville, TN, p. 254.
Huo X, Zhang Q (2008). Effect of Skewness on the Distribution of Live Load Reaction at Piers of Skewed Continuous Bridges. J. Bridge Eng., 13(1): 110-114.
Jaeger LG, Bakht B (1982). The grillage analogy in bridge analysis. Canadian J. Civil Eng., 9(2): 224-235.
Mabsout M, Tarhini K, Frederick G, Kesserwan A (1999). Effect of multilanes on wheel load distribution in steel girder bridges. J. Bridge Eng., 4(2): 99-106.
OHBDC (1983). Ontario Highway Bridge Design Code. Ontario Ministry of Transportation and Communications. Downsview, Ontario, Canada.
Samaan M, Sennah K (2002). Distribution of wheel loads on continuous steel spread-box girder bridges. J. Bridge Eng., 7: 175-183.
Song S, Chai Y, Hida S (2003). Live-Load Distribution Factors for Concrete Box-Girder Bridges. J. Bridge Eng., 8(5): 273-281.
Suhatril M, Adnan A, Taib IM, Karbakhsh A (2011). The comparison of soil-pile interaction and fixed base support for integral prestressed concrete box-girderbridge. Int. J. Phys. Sci., 6(28): 6580-6595
Zheng L (2008). Development of new distribution factor equations of live load moment and shear for steel open-box girder bridges.Doctoral Thesis, Tennessee Technological University, Cookeville, Tennessee.
Zokaie T, Mish K, Imbsen R (1993). Distribution of wheel loads on highway bridges, Phase 3. Final Report to National Cooperative HighwayResearch Program (NCHRP) project 12-26.Washington D.C.