LIVE LOAD TESTING FOR MOMENT DISTRIBUTION FACTORS ON A TWIN-BRIDGE IN BLOUNT COUNTY, TENNESSEE

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ABSTRACT

This study focused on the live load testing for moment distribution factors of normal strength concrete bridge and high strength concrete bridge. A live load test was conducted on a twin-bridge in Blount County, Tennessee. High strength concrete (HSC) beams were used in one of the twin bridge, and normal strength concrete (NSC) beams were used in the other. Live load test was conducted using Tennessee rating trucks with various loading paths. Based on the live load testing, moment distribution factors under different loading cases were obtained. Finite element analysis, AASHTO LRFD method and a proposed method for distribution factors were used to determine the live load distribution factors of moment for this twin bridge. The proposed method was a simplified procedure for live load distribution, and was derived based on a study of over 500 bridges. The comparison of distribution factors from all the methods mentioned above was performed for both HSC and NSC beams. The research results showed that HSC bridge exhibited higher moment distribution factors than NSC bridge, due to higher beam stiffness. The distribution factors obtained from the live load testing were close to that from the finite element analysis. Both AASHTO LRFD method and proposed equations generate slightly higher moment distribution factors than live load testing results. Compared with AASHTO LRFD method, the proposed equations had the closest results to the live load testing and finite element analysis for this twin bridge.

Keywords: Bridge, Load distribution, Finite element analysis, Live load test

INTRODUCTION

In bridge design, the concept of distribution factor allows the designers to consider the transverse effect of wheel loads and determine the shear and moments of beams under live loads in a simple and convenient manner. According to the approach of the load distribution, maximum shears and moments in beams are obtained first as if the wheel loads are applied directly to beams. These values are then multiplied by the appropriate live-load distribution factors to obtain critical live load shears and moments of beams in bridges.

Currently, the lateral distribution factor of live load moment in highway bridge design is commonly determined using the method in the AASHTO LRFD bridge design specifications. The equations in the AASHTO LRFD were developed based on models with uniform spacing, beam inertia and skew angle. The continuous bridge models had equal spans, and diaphragm effects are not included. The equations were also calibrated against a database of real bridges with certain ranges of span length, moment of inertia, spacing and so on. Therefore, the equations are accurate when applied to bridges whose parameters are within the respective ranges of bridge models as well as bridges in the database.

Finite element analysis (FEA) is accepted as an accurate method to obtain live load distribution factors. Compared with other major methods used to study the behavior of bridges, such as grillage analysis and orthotropic plate theories, finite element method requires the fewest simplifying assumptions. The AASHTO LRFD equations were developed based on the results of FEA. Many researchers use FEA to conduct parametric studies of distribution factors. Live load testing, or field testing, is a major tool in bridge evaluation and it gives more reliable information about the actual performance of bridges. In an analysis, despite improved structural analysis techniques, there are still some uncertainties, such as real material properties, unaccounted system stiffness, and bearing resistant forces that will affect the results. Many researchers have used field testing to verify bridge live load distribution factors.

Both finite element analysis and live load testing are time consuming and expensive. The AASHTO LRFD equations are much simpler than live load testing and finite element analysis, the equations, however, have limited ranges of applicability for structure parameters. For the cases in which the bridge parameters exceed the ranges of applicability, a refined analysis is mandated. In such cases, engineers have to work on a case-by-case basis and perform a detailed, rigorous analysis of the bridges. Therefore, a simplified live load distribution method with few limitations is desired.

A set of proposed equations that was derived by studying over 500 bridges is briefly introduced in this paper. The proposed equations require fewer bridge parameters and use simpler equation format than the AASHTO LRFD method while maintaining the accuracy of the calculation of live load distribution factors. Comparison study was performed among the live load testing results, finite element analysis results, AASHTO LRFD method and proposed equations. The results of the comparison were used to demonstrate the correctness of the proposed equations.

DESCRIPTION OF BRIDGE

The live load test was conducted on a twin bridge on state route 162 over Pistol Creek in Blount County, Tennessee. The bridge has five spans with span length of 75.1 ft, 74.33 ft × 3, and 75.1 ft, respectively. The total length of the bridge is 373.2 ft, and the width of the bridge is 51.2 ft. The northbound bridge (left lane) is a normal strength concrete (NSC) bridge with specified compressive strengths of 5,500 psi at release and 6,000 psi at 28 days. The southbound bridge (right lane) is a high strength concrete (HSC) bridge with specified compressive strengths of 5,500 psi at release and 10,000 psi at 28 days. The superstructure of the bridge consists of five lines of prestressed beams spaced at 10 ft - 7 in. and cast-in-place reinforced concrete deck with a thickness of 8.75 in. The specified compressive strength of deck concrete is 4,000 psi. AASHTO Type III beams were used in the bridge. For the purpose of comparison, the NSC beams and the HSC beams used the same reinforcement and prestressing pattern. 30 - ½ in. diameter 270 ksi seven wire, low-relaxation steel was used in each beam.

The elevation, typical cross section and plan view of Pistol Creek Bridge are shown in Figure 1 through Figure 3, respectively.

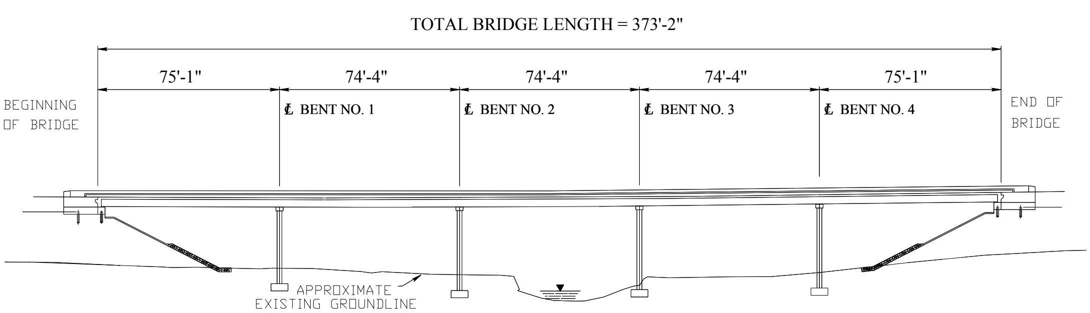


Figure 1. Elevation of Pistol Creek Bridge

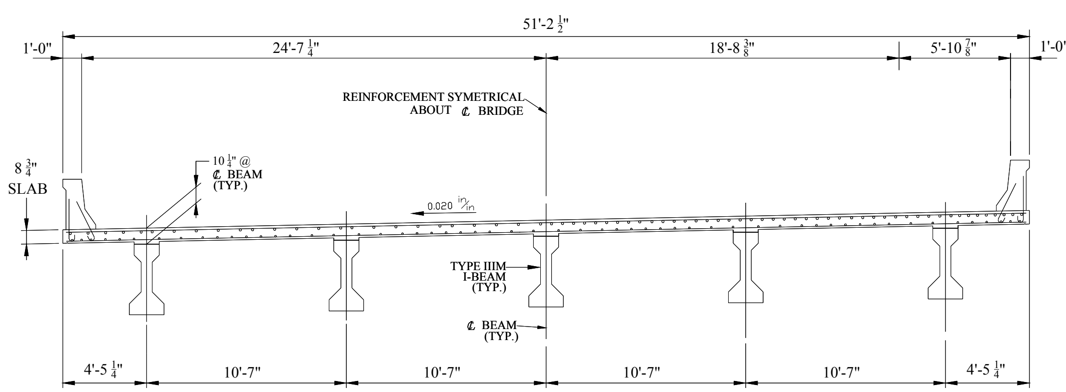


Figure 2. Typical Cross Section of Pistol Creek Bridge

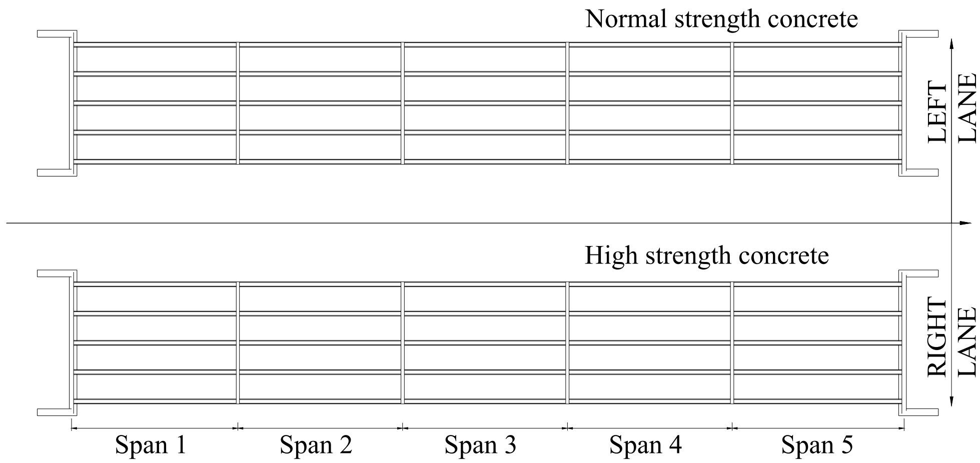


Figure 3. Plan View of Pistol Creek Bridge

LIVE LOAD TESTING

Two three-axle dump trucks from the Tennessee DOT were used during the live load testing. The truck configurations are shown in Figure 8. The distance between front axle and middle axle was 11 ft and between the middle axle and rear axle was 9 ft. The width of truck was 6 ft 9 in. at the front axle and was 6 ft at the rear axle. The two trucks had the same weight of 71.2 kips.

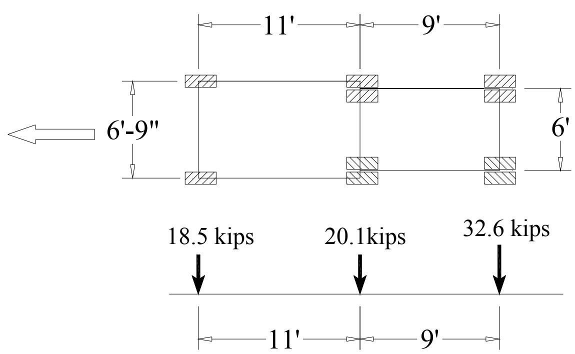


Figure 4. Truck Configurations

Before applying truck loads, the bridge was marked with washable paint as shown in Figure 5. The marks for ①, ②, and ③ indicate the truck loading paths and the longitudinal lines represent the centerlines of the wheels of each truck. The transverse marks are the truck stop positions, and only the positions of middle axles are shown. The NSC and HSC bridges were loaded with one truck and two trucks during the live load testing. The loading cases were pre-determined by finite element analysis in order to produce maximum moment at those instrumented sections with two-trucks and three-trucks loadings. Since only two trucks were available, the effect of three-trucks loading was determined by combining one-truck loading and two-trucks loading.

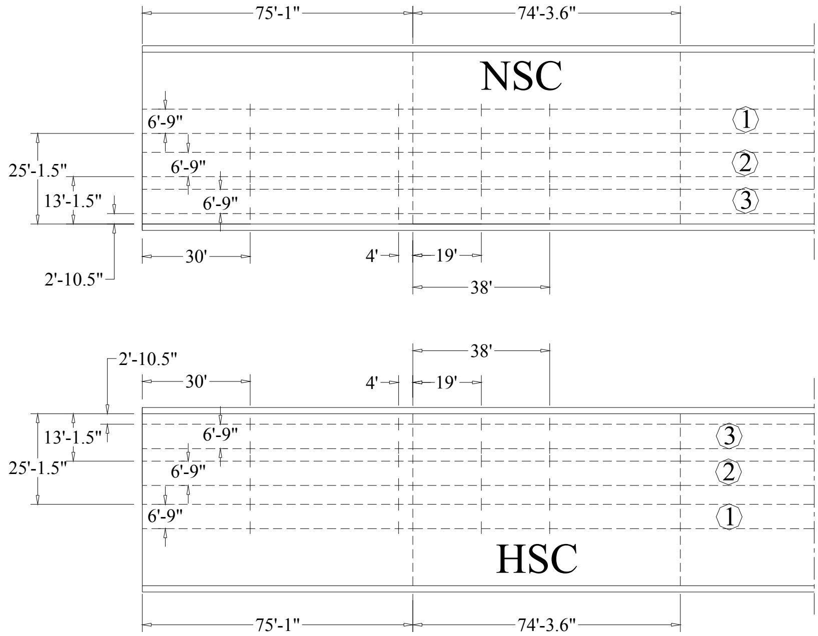


Figure 5. Truck Loading Paths

Live load tests were carried out according to the testing plan for the NSC bridge and the HSC bridge. The trucks moved along the marked lines on the bridge in a slow speed, and then stopped and stayed at four marked positions for at least five minutes to allow the data acquisition system to record adequate data. Figure 6 shows the details of the truck loading plan in the span #2 of NSC and HSC Bridges. Both tests went smoothly, and the strain results were quite stable. The strains in beams showed very little dynamic effect under the low-speed moving trucks. The testing procedures for both bridges were similar.

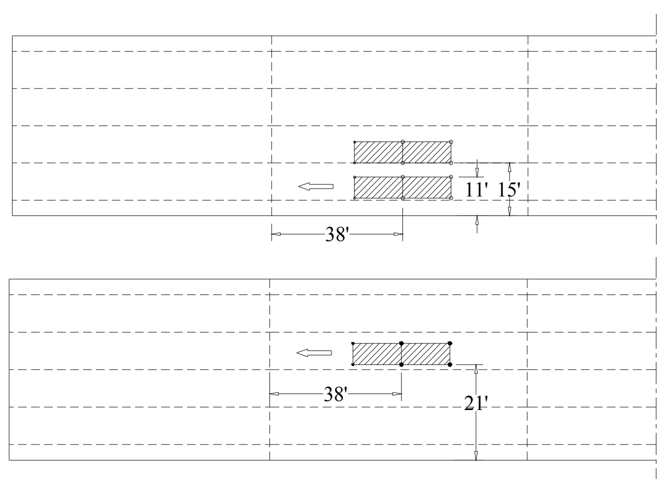


Figure 6. Loading Plan for Live Load Testing in Span #2

TEST RESULTS

The moments in the beams caused by live load were calculated using the strains measured by the bottom gauge and the following equation:

$$M = \frac{\varepsilon E I_c}{h} \quad (1)$$

Where M = moment carried by one beam; I_c = moment of inertia of composite beam; h = distance from bottom gauge to centroid of composite section; E = modulus of elasticity of beam concrete; and ε = change of concrete strain at the bottom gauge location under loading. The distribution factors of moment (DFM) at the 0.4L section of span #1 and the mid-span section of span #2 are shown in Table 1. The factors were calculated using equation (2).

$$DFM = \frac{M}{M_0} \quad (2)$$

Where M = maximum moment obtained from bridge model analysis; and M_0 = maximum moment obtained from single beam analysis. The results include two-trucks and three-trucks loading cases. For three-trucks loading cases, the results were obtained by combining the results from two-truck loading and one-truck loading cases. After considering the multiple presence reduction factor of 0.85, the distribution factors of two-trucks loading and three-trucks loading are close.

Table 1. DFM Results of Live Load Testing

Section		Loading	Strain (10^{-6})	I_c (in^4)	Moment (kip-ft)	M_0 (kip-ft)	DFM
NSC	0.4L Span #1	2 trucks	82.5	478,998	473.9	877.5	0.540
		3 trucks	96.5		554.3		0.537
	Mid-span Span #2	2 trucks	86		494.0	696.3	0.709
		3 trucks	95		545.7		0.666
HSC	0.4L Span #1	2 trucks	69	463,404	462.3	805.3	0.574
		3 trucks	80		536.0		0.566
	Mid-span Span #2	2 trucks	73.5		492.5	638.3	0.771
		3 trucks	82.5		552.8		0.736

The distribution factors of the HSC bridge are slightly higher than those of the NSC bridge by about 6% for span #1 and about 9% for span #2. This agreed with the finite element analysis results. It could be attributed to the higher modulus of elasticity of HSC and hence the higher stiffness of the HSC beams.

The other observation from the test results is that the DFM in span #1 are smaller than those in span #2. The average DFM in span #2 is 0.721, while the average DFM in span #1 is 0.554, which is 23% less than that in span #2. The large differences between the distribution factors of span #1 and span #2 could be due to the difference between the support conditions in the real bridge and in the 2-D finite element model that was used to calculate M_0. In the 2-D analysis, a hinge support was used at the end of span #1. But in the real structure, the beams were attached to the back wall at abutment. The integral abutment behaved as a support between simple support and fixed support. To take into consideration the effect of integral abutment, the single beam model (2-D model) was modified with rotational springs at two ends of the bridge, as shown in Figure 7. The joint stiffness α depends on the ratio of the spring stiffness k to the flexural stiffness 4EI/L of the beam and is defined as kL/4EI. Figure 8 shows the relationship between support joint stiffness and beam maximum moment. When the rotational joint stiffness α equals to 0.012, the results of span #1 are close to those of span #2, as shown in Table 2. This support condition should be close to the actual abutment.

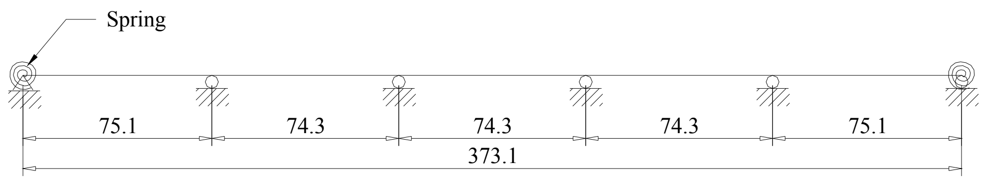


Figure 7. Modified Single Beam Model

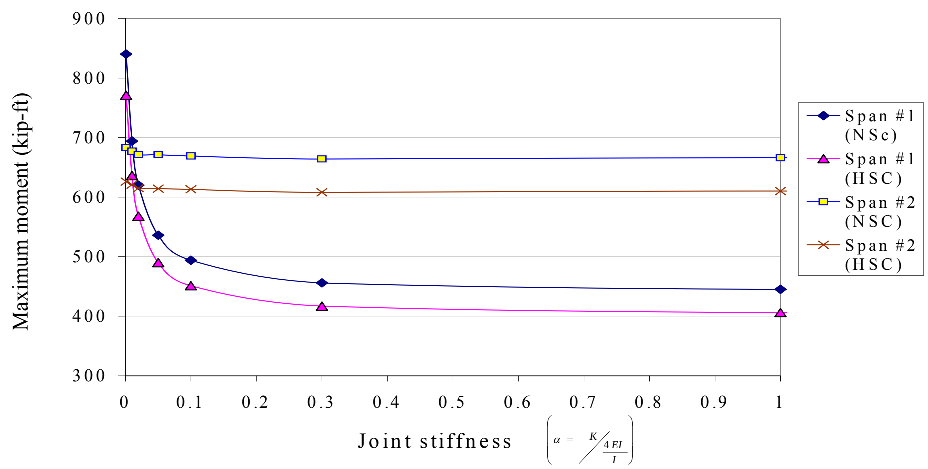


Figure 8. Maximum Moment vs. Joint Stiffness

Table 2. Distribution Factors from Live Load Testing (Joint Stiffness α = 0.012)

Section		Loading	Moment (kip-ft)	M_0 (kip-ft)	DFM
NSC	0.4L Span#1	2 trucks	473.9	676.5	0.701
		3 trucks	554.3		0.696
	Mid-span Span #2	2 trucks	494.0	696.3	0.709
		3 trucks	545.7		0.666
HSC	0.4L Span#1	2 trucks	462.3	619.6	0.746
		3 trucks	536.0		0.735
	Mid-span Span #2	2 trucks	492.5	638.3	0.771
		3 trucks	552.8		0.736

FINITE ELEMENT ANALYSIS

A three-dimension finite element bridge model was developed for the determination of live load distribution factor. The models were developed based on the following assumptions:
A small-deflection theory was used and full composite action between beams and deck slab was assumed. Roller and hinge supports are used in the finite-element models. All materials are elastic and homogenous.

The material properties from laboratory experiments were used in the modeling. The analysis only considered the effects of live load. Therefore, the stress or moment in the beams due to prestressing force and dead load were not considered. The stiffness effect of the parapet was not considered. In the model, the concrete slab was idealized as quadrilateral shell element with the element size of 2 ft × 2 ft, and the longitudinal beams were idealized as line beam element, as shown in Figure 9.

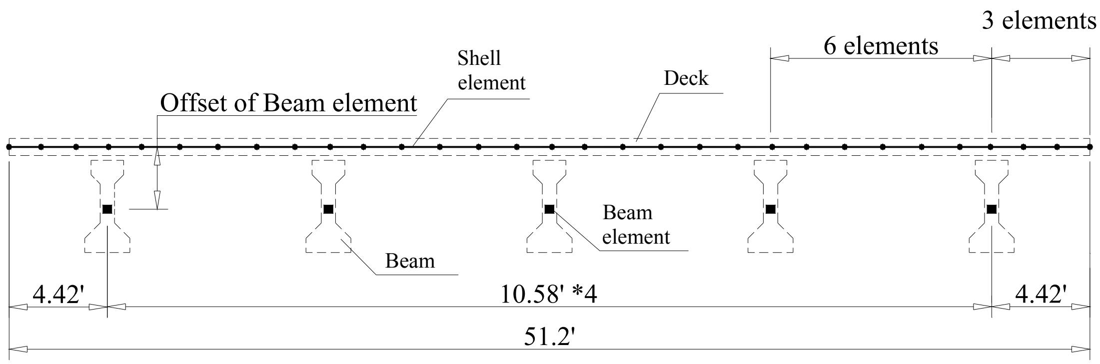


Figure 9. Cross-Section of the Bridge Model

For the convenience of comparison between theoretical analysis and experimental test, the moments at the gauge locations (midspan and 0.4L sections) were investigated. TDOT dump trucks were applied as live load for one-, two- and three-trucks loading cases. Trucks were placed at the pre-determined longitudinal positions and then moved transversely within the width of bridge. Analyses were performed for numerous loading cases and moments in beams were calculated. After analyzing all possible loading positions, the maximum moment and the critical loading cases could be determined. The moment distribution factors were calculated for every case, and results are shown in Table 3. For three-trucks cases, the calculated distribution factors were multiplied by a multiple presence factor of 0.85.

It can be seen from the table, when having the same number of truck loads, the DFM results at the different spans for two bridges are quite close, although the moments may be different. While comparing the results of two bridges, it was noticed that the distribution factors of the HSC bridge were higher than those of the NSC bridge by 1% to 2%. As the dimensions and loading positions of the two models were exactly the same, the difference of the DFM between the two bridges is attributed to the differences in the concrete properties. The concrete modulus of elasticity of the HSC beam was higher than that of the NSC beam. The higher stiffness of the HSC beam may result in its relatively larger DFM, which also agrees with the equation in AASHTO LRFD.

Table 3. Distribution Factor from Finite Element Analysis

Section	Loading	S_b (lb/ft^2)	I_c (in^4)	Y_{bc} (in)	Moment (kip-ft)	M_0 (kip-ft)	DFM
0.4L of Span #1 (NSC)	1 Truck	52,430	478,998	40.53	358.6	877.5	0.409
	2 Trucks	90,817			621.1		0.708
	3 Trucks	107,878			737.8		0.715
Midspan of Span #2 (NSC)	1 Truck	43,721			299.0	696.3	0.429
	2 Trucks	73,799			504.7		0.725
	3 Trucks	86,109			588.9		0.719
0.4L of Span #1 (HSC)	1 Truck	49,427	463,404	39.67	334.1	805.3	0.415
	2 Trucks	85,318			576.8		0.716
	3 Trucks	102,310			691.6		0.730
Midspan of Span #2 (HSC)	1 Truck	41,215			278.6	638.3	0.436
	2 Trucks	69,328			468.7		0.734
	3 Trucks	80,574			544.7		0.725

S_b = stress at beam bottom fiber

COMPARISON OF TEST RESULTS AND DISTRIBUTION FACTORS EQUATIONS

The field test results were compared with the finite element analysis (FEA) results, AASHTO LRFD method and a set of newly proposed equations. In this section, only the results of interior beams are discussed.

COMPARISON OF FEA AND FIELD TEST RESULTS

The moment distribution factors obtained from finite element analysis and live load field test are shown in Table 4. For field test results, two-truck loading introduced larger distribution factors than three-trucks loading cases when a multiple presence factor of 0.85 was used for three-trucks loading. For NSC bridge, moment distribution factors from live load testing are smaller than those from Finite element analysis. However, moment distribution factors from live load testing for HSC bridge are larger than those from Finite element analysis. In general, the distribution factor for NSC bridge and the HSC bridge from these two methods were quite close, usually less than 5%. The distribution factors from the NSC bridge were all smaller than those from the HSC bridge, which indicated the higher stiffness of beams could increase the distribution factor of the moment.

Table 4. Moment Distribution Factors of FEA and Live Load Testing

Section		Loading	Moment Distribution Factor	
			Field test	FEA
NSC	0.4L Span #1	2 trucks	0.701	0.708
		3 trucks	0.696	0.715
	Mid-span Span #2	2 trucks	0.709	0.725
		3 trucks	0.666	0.719
HSC	0.4L Span #1	2 trucks	0.746	0.716
		3 trucks	0.735	0.730
	Mid-span Span #2	2 trucks	0.771	0.734
		3 trucks	0.736	0.725

AASHTO LRFD EQUATIONS

The AASHTO LRFD equations for live-load distribution considered variations in beam spacing, beam stiffness, span length, skew, and slab stiffness. The following are equations of moment distribution factor for interior beams. The distribution factors of live load moment for interior beams in HSC and NSC bridges are calculated using these equations.

<u>For one design lane loaded:</u>

$$\mathrm{DFM}=0.06+\left(\frac{S}{14}\right)^{0.4}\times\left(\frac{S}{L}\right)^{0.3}\times\left(\frac{K_g}{12.0\times L\times t_s^3}\right)^{0.1} \tag{3}$$

<u>For two or more design lanes loaded:</u>

$$\mathrm{DFM}=0.075+\left(\frac{S}{9.5}\right)^{0.6}\times\left(\frac{S}{L}\right)^{0.2}\times\left(\frac{K_g}{12.0\times L\times t_s^3}\right)^{0.1} \tag{4}$$

Where S = beam spacing, ft; L = beam span, ft; t_s = depth concrete slab, in; K_g = = n× (I +A e_g^2), longitudinal stiffness parameter, in^4; and n = modular ratio between beam and slab concrete, n = E_b/E_{cs}.

PROPOSED EQUATIONS

Simplified distribution factor equations are proposed based on a study of over 500 bridges. The proposed equations include variables of beam spacing, span length, bridge clear width and number of girders. Because the concrete strength is not required for the proposed equations, the distribution factors of beams in the HSC and NSC bridges are the same.

<u>For one design lanes loaded:</u>

$$\mathrm{DFM} = m\times\left(\frac{3\,S}{L}\right)^{0.35}\times\left(g_{\text{lever rule}}\right) \geq m\times\left(\frac{N_L}{N_g}\right) \tag{5}$$

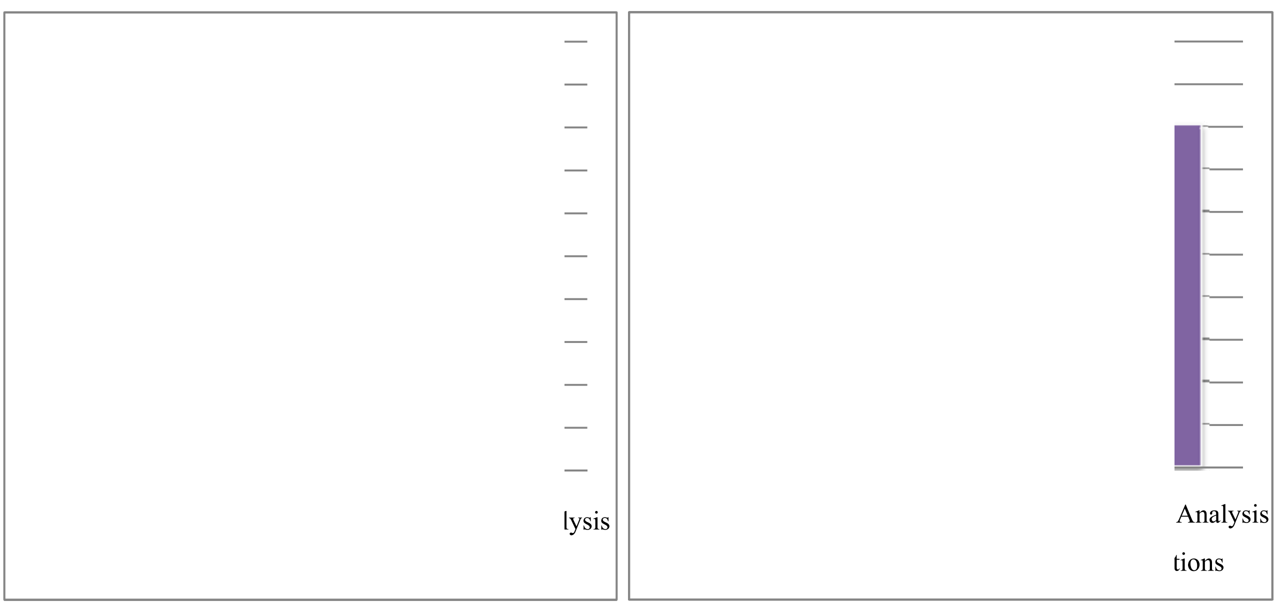
Where, m = multiple presentation factor; S = beam spacing, ft; L = beam span, ft; W_c = clear road width, ft; N_g = number of girders in the bridge cross section; $g_{\text{lever rule}}$ = distribution factor computed with the lever rule method, and N_L = Maximum number of design lanes considered in an analysis. Use integer part of $\frac{W_c}{12}$ to determine number of loaded lanes N_L for multiple presence factor and m shall be greater than or equal to 0.85.

For two or more design lanes loaded:

$$DFM = m \times \left(\frac{3\,S}{L}\right)^{0.01} \times \left(\frac{W_c}{10 \times N_g}\right) \geq m \times \left(\frac{N_L}{N_g}\right) \tag{6}$$

COMPARISON OF THE RESULTS FROM VARIOUS METHODS

Figure 10 shows the comparison of moment distribution factors of interior beams from live load testing, finite element analysis, AASHTO LRFD, and proposed equations. As expected, the distribution factors obtained from FEA and the live load testing are smaller than those calculated from equations. The distribution factors from AASHTO LRFD method are larger than all the other results, 12.0% higher than the FEA results for the NSC bridge beam and 14.1% higher than the FEA results for the HSC bridge beam. Likewise, LRFD results were 14.5% higher than the live load testing results for the NSC bridge beam and 8.7% higher for the HSC bridge beam. The distributions factors from proposed equations are very close to the live load testing results and FEA results, which was about 10 % higher than the FEA results and about 13.4% higher than the live load testing results for NSC bridge and 4.3% higher for the HSC bridge. The comparison study indicates that proposed equations are a reliable method to determine the distribution factor of live load moment for this twin bridge.



(a) NSC Bridge (b) HSC Bridge

Figure 10. Moment Distribution Factor of Interior Beam in NSC and HSC bridges

CONCLUSIONS

Based on the results of this limited study, following conclusions have been made:

- Live load testing results showed that the distribution factors of the HSC bridge are higher than those of the NSC bridge. This agreed with the finite element results. The high modulus of elasticity of HSC introduces a high stiffness of the HSC beams. As a result, live load distribution factor of HSC beams is increased.
- Finite element analysis is a good way to calculate live load moment distribution factor. The results from finite element analysis are close to those from the field test.
- The moment distribution factors calculated from AASHTO LRFD equations were higher than the live load testing results and finite element analysis results, which indicated that the LRFD equations are conservative.
- The proposed distribution factors equations are simpler than AASHTO LRFD method in the way that includes less bridge parameters. Meanwhile these equations maintain the accuracy of calculation. The comparison showed that the results from the proposed equations are closer to the field test results and finite element analysis results.

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