EVALUATION OF LIVE LOAD DISTRIBUTION FOR MOMENT IN NEXT BEAM BRIDGE

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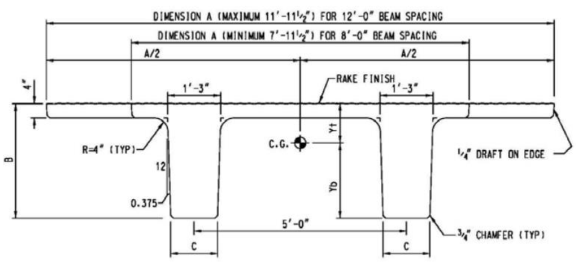
ABSTRACT
Currently, the live load distribution factors (LLDF) for the northeast extreme tee (NEXT) beam bridges are calculated on the basis of AASHTO equation for each beam stem, then two LLDFs from each stem are added together and applied to the entire beam, which may lead to an over-conservative design. This paper employed a 3-D finite element (FE) modeling of an entire NEXT beam bridge to evaluate the live load distribution for moment in each beam. FE results were compared to the manual solution based on the AASHTO LRFD Bridge Design Specifications, which indicated that using AASHTO type ”k” live load distribution factor for moment in interior beam could lead to a safe design of 8ft-wide NEXT beams, for both exterior and interior girders.

Keywords: NEXT Beam, Live Load Distribution Factor, Finite Element (FE).

INTRODUCTION

Adjacent-box-beam bridges gained popularity for medium span (i.e., about 40ft-120ft) in the northeast United States in past decades, however, various durability problems were identified for adjacent-box-beam bridges in service in a recent study by Russell (2011)[1]. It becomes important to have a new bridge system that could substitute the adjacent-box-beam bridge system to enhance the sustainability of highway bridges[2]. The PCI northeast bridge technical committee proposed and developed the northeast extreme tee (NEXT) beam sections that could span from about 45ft to 90ft[2,3], which would not compete with northeast bulb-tee sections that could span from about 70ft to 148ft as indicated by Culmo and Seraderian[2]. Fig.1 shows the NEXT beam sections and their section properties[2,3]. As can be seen, section depth varies from 24in. to 36in., while the beam width varies from 8ft to 12ft.



NEXT BEAM - SECTION PROPERTIES

BEAM DESIGNATION	BEAM WIDTH INCHES	BEAM DEPTH INCHES	BASE STEM WIDTH INCHES	AREA IN2	I IN4	Yb INCHES	Yt INCHES	St IN3	Sb IN3	WEIGHT PLF
	A	B	C			D	E			
MINIMUM WIDTH BEAMS										
NEXT 36 F	95.50	36.00	13.00	1287	160240	21.77	14.23	11261	7361	1341
NEXT 32 F	95.50	32.00	13.25	1182	115813	19.51	12.49	9272	5936	1231
NEXT 28 F	95.50	28.00	13.50	1075	79901	17.24	10.76	7426	4635	1120
NEXT 24 F	95.50	24.00	13.75	966	51823	14.95	9.05	5726	3466	1006
MAXIMUM WIDTH BEAMS										
NEXT 36 F	143.50	36.00	13.00	1479	185525	23.36	12.64	14678	7942	1541
NEXT 32 F	143.50	32.00	13.25	1374	134258	20.98	11.02	12183	6399	1431
NEXT 28 F	143.50	28.00	13.50	1267	92661	18.57	9.43	9826	4990	1320
NEXT 24 F	143.50	24.00	13.75	1158	60045	16.12	7.88	7620	3725	1206

Fig. 1 Section Properties of NEXT Beams[2,3]

The NEXT beams offer several advantages over other types of beams[2] : (1) no installation or stripping of formwork is required in the field; (2) no intermediate diaphragms are included. Thus, using NEXT beams can accelerate the construction process[2]. However, as a newly developed bridge beam section, the calculation of live load distribution factor for NEXT beam with a composite concrete deck is not addressed in current AASHTO LRFD Bridge Design Specifications[3]. The PCI Northeast Bridge Technical Committee suggested a conservative approach for calculation of live load distribution factors[3]. For calculating live load distribution factor for NEXT Beam F, equations for cross section type "k" in AASHTO LRFD Table 4.6.2.2.2b-1 is used with the following modifications[3]:

1. *Treat each stem as an individual beam and calculate the distribution factor for each stem based on the average stem spacing;*
2. *Multiply the above factor by two and apply it to the entire NEXT beam section;*

As stated in the guideline for NEXT beam, the above method produces more conservative LLDF when compared to calculations using the full beam width as girder spacing[3]. Note that this approach might be over-conservative. Therefore, this paper intends to evaluate the live load distribution for moment in each beam by employing a 3-D finite element (FE) modeling of an entire NEXT beam bridge. FE results will be compared to the manual solution based on the AASHTO LRFD Bridge Design Specifications[4].

VALIDATION OF 3-D FINITE ELEMENT MODELING IN SAP2000

In this study, a 3-D FE modeling was employed to investigate the distribution of live loads in a NEXT beam bridge, using SAP2000 program. Firstly, the modeling technique was verified with manual solution for a one-beam bridge model. Fig. 2 shows the cross section of one-beam bridge model. In SAP2000, an 8ft wide non-composite NEXT beam with an 8'' concrete slab on top of the beam is created. On the basis of the design chart of NEXT beam[2,3], the precast beam length of 70ft is chosen with a 68ft bearing-to-bearing length. Concrete compressive strength for the NEXT beam and the top concrete slab are 8.0ksi and 4.0ksi, respectively. AASHTO LRFD design loading[4], i.e., HL-93, was assigned on the bridge model to obtain the structural response per lane loading. Note that the HL-93 loading consists of a design truck, HS-20, and a 0.64k/ft design lane load[4]. In FE model, AASHTO design truck with dynamic impact was treated as a set of point loads, as shown in Fig.3, while the AASHTO design lane load was uniformly distributed as an area load over the entire beam, as shown in Fig.4.

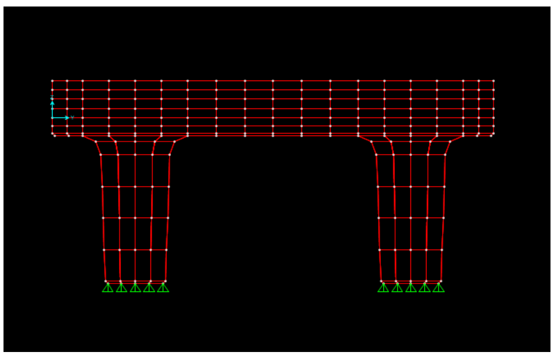


Fig. 2 Bridge section of one-beam model

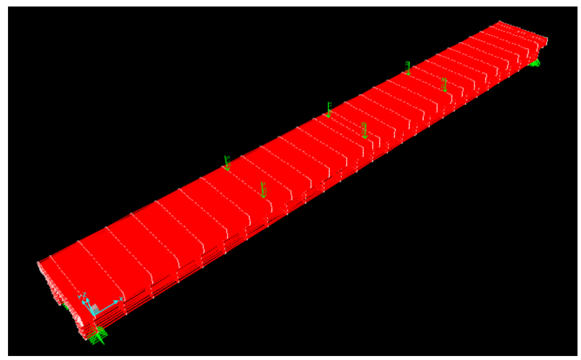


Fig. 3 Truck load HS-20 with Dynamic impact

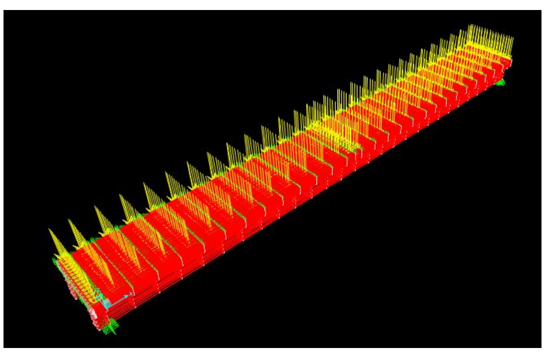


Fig. 4 Lane load (0.64kip/ft)

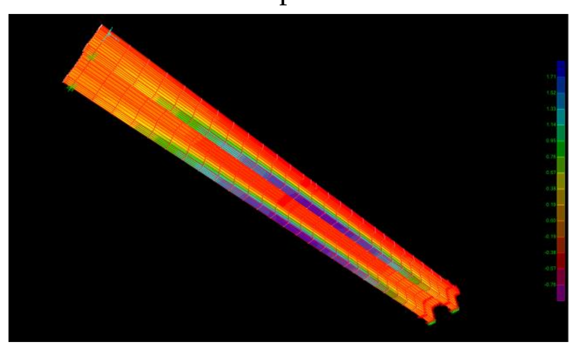


Fig. 5 Stress contour under HL-93 loading

Under the HL-93 loading, the normal stress, S_{11}, along the beam length direction can be obtained. Fig. 5 shows the stress contour of S_{11} in the beam under the combined loadings. The maximum stress is 1.80ksi and it is the same in each beam stem because of the symmetry of the beam section and the loadings. For a simply-supported beam under HL-93 design loading with dynamic impact, the maximum bending moment can be calculated manually as of 1633.3kip-ft. The section modulus for extremely bottom fiber, S_b, of the composite section is calculated as of 10672.6in^3. Thus, the maximum tensile stress at extremely bottom fiber can be determined as of 1.84ksi. As can be seen, the difference between the FE results and manual solution is approximately 2%. Similarly, the maximum stress under separate load is extracted from SAP2000, as shown in Table 1. The manual solutions of the maximum S_{11} stresses at extremely bottom fiber are 0.42kais and 1.42ksi under design truck (with dynamic impact) and design lane load, respectively, as shown in Table 1 as well.

Table 1 Comparison of maximum tensile stress from FE modeling and hand calculation

	Under design lane load	Under design truck with dynamic impact	Combined loading, HL-93
FE result	0.41ksi	1.39ksi	1.80ksi
Manual solution, LRFD	0.42ksi	1.42ksi	1.84ksi

As can be seen from Table 1, the differences between all the FE results and manual solutions are approximately 2%, which indicated an excellent accuracy of FE modeling by SAP2000 in this research. Therefore, with confidence SAP2000 was employed for the modeling of an entire NEXT beam bridge, as discussed below.

LIVE LOAD DISTRIBUTION FACTOR FOR NEXT BEAM BRIDGE BY 3-D FINITE ELEMENT MODELING

Culmo and Seraderian (2010) presented the development of the northeast extreme tee (NEXT) beam for accelerated bridge construction[2]. Fig.6 shows one of the typical bridge sections that uses NEXT beams[2,3], which was selected in this study for the investigation of live load distribution. As can be seen, the curb to curb width of the bridge is 28'-11 ½'' with two 12ft design lanes. Bridge girder spacing is 8ft. Note that for design purpose, this bridge can be loaded by one design lane or by two design lanes[4]. The design lanes can be placed anywhere within the bridge width to obtain the most critical scenario[4].

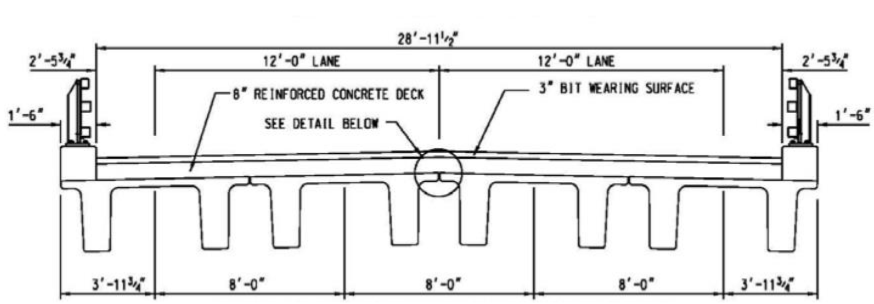


Fig. 6 Bridge section with minimum width beams[2,3]

ONE DESIGN LANE LOADED CASES

Fig.7 shows load case 1-1 that have only one design lane loaded, in which the design lane load was place right next to the left curb. This scenario will give the maximum loading effects on the exterior beam. Fig. 8 showed the design lane load on the bridge for load case 1-1 in SAP2000, while Fig. 9 showed the design truck load (with dynamic impact) on the bridge. The results of structural responses under the loadings can be obtained by displaying the stress contour in SAP2000. Fig. 10 showed the stress contour of S_{11} in concrete bridge beams and concrete slab. It can be seen that exterior stem exhibited larger S_{11}stress than all the others with a maximum stress of 0.817ksi at stem 1. For interior beams, the maximum S_{11}stress of 0.627ksi is observed at stem 3 under load case 1-1.

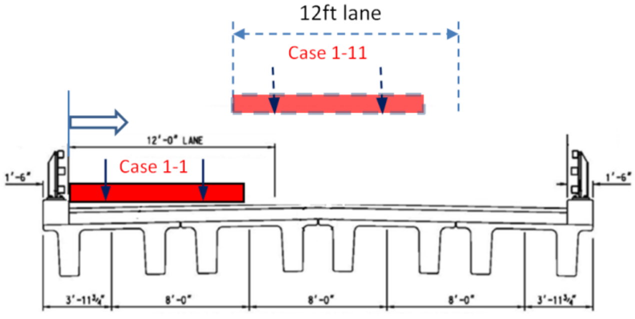


Fig. 7 Loading profile for Case 1-1
(note: bridge section adapted from [2])

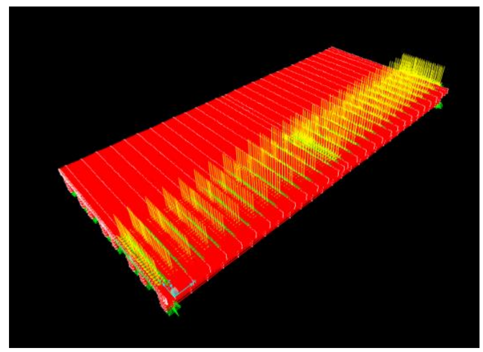


Fig. 8 Design lane load for Case 1-1

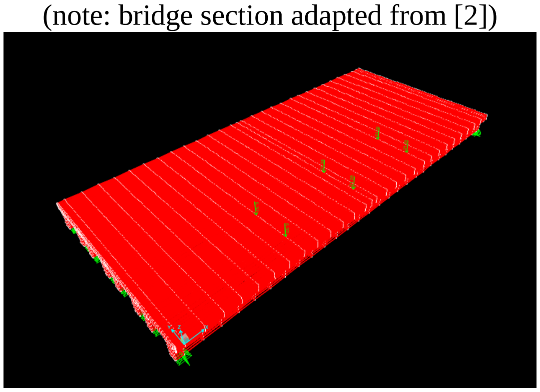


Fig. 9 HS-20 truck load with dynamic impact for Case 1-1

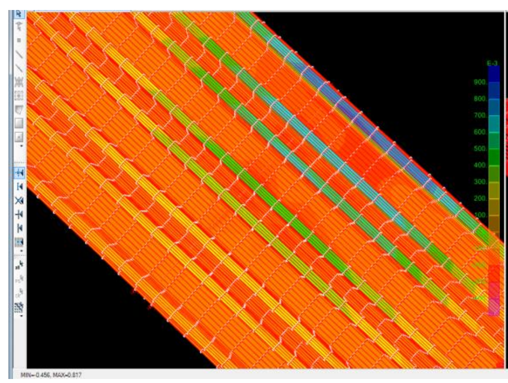


Fig. 10 Stress contour of S_{11} in bridge beams under combined loadings for Case 1-1

In order to determine the maximum loading effect on the interior beams, additional ten (10) load cases were investigated by moving the load case 1-1 transversely by every one foot to the right curb direction. Table 2 showed the results of maximum S_{11} stress under the above eleven one-lane loaded cases.

Table 2 Maximum tensile stress in each stem under one-lane loaded cases

	Exterior Beam		Interior Beams				Exterior Beam	
	Beam 1		Beam 2		Beam 3		Beam 4	
	Stem 1	Stem 2	Stem 3	Stem 4	Stem 5	Stem 6	Stem 7	Stem 8
Case 1-1	**0.817**	0.701	**0.627**	0.480	0.397	0.281	0.222	0.157
Case 1-2	0.758	0.675	0.618	0.492	0.414	0.300	0.247	0.174
Case 1-3	0.698	0.650	0.607	0.503	0.430	0.320	0.267	0.190
Case 1-4	0.639	0.622	0.588	0.518	0.445	0.338	0.285	0.209
Case 1-5	0.586	0.587	0.575	0.526	0.466	0.356	0.300	0.230
Case 1-6	0.536	0.556	0.566	0.526	0.472	0.375	0.324	0.250
Case 1-7	0.496	0.529	0.549	0.525	0.485	0.399	0.348	0.274
Case 1-8	0.454	0.500	0.533	0.520	0.505	0.418	0.370	0.297
Case 1-9	0.418	0.476	0.511	0.521	0.512	0.441	0.395	0.326
Case 1-10	0.387	0.446	0.480	0.523	0.518	0.462	0.421	0.355
Case 1-11	0.348	0.417	0.450	0.505	0.515	0.477	0.444	0.381

As can be seen from Table 2, case 1-1 governed all the other cases, giving a maximum tensile stress of 0.817ksi at the extreme bottom fiber of the exterior beam, and also giving a maximum tension stress of 0.627ksi at the extreme bottom fiber of the interior beam. In accordance with AASHTO LRFD bridge design specifications, a multiple presence factor of 1.2 shall be applied to one-lane loaded case[4]. Therefore, for one-lane loaded cases, the maximum tension stresses will be 0.980ksi and 0.752ksi for exterior and interior beams, respectively.

TWO DESIGN LANE LOADED CASES

Fig.11 shows the loading profile of a two design lane loaded case, called “case 2-1”, in which the two design lanes are adjacent to each other. The design lane loads were placed to the left side of their design lanes to achieve maximum loading effects on the exterior beam. Additional 5 cases were explored by moving the load case 2-1 transversely to the right by every one foot to study the maximum loading effect on the interior beams. Note that the design lane load can appear anywhere within the design lane, as stated in AASHTO LRFD specification[4]. In this regard, load case 2-7, as shown in Fig. 12, was also adopted for further study as this load pattern may give more critical loading effects on the interiors beams than that from load case 2-1. Similarly, additional 3 cases were explored by moving the case 2-7 loading transversely to the right by every one foot to determine the maximum loading effect on the interior beams. Fig. 13 depicted the design lane load on the bridge for the load case 2-1 in SAP2000. Fig. 14 showed the stress contour of S_{11} in concrete bridge beams and concrete slab for load case 2-1. It can be seen that exterior stem exhibited larger S_{11} stress than all the others. The maximum S_{11} stress is of 1.098ksi in beam stem 1. For interior beams under load case 2-1, the maximum S_{11} stress is 1.022ksi located at stem 3.

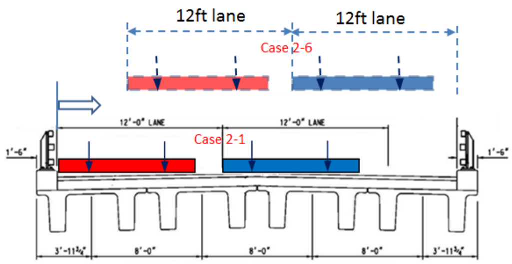


Fig 11 Loading profile for Case 2-1
(note: bridge section adapted from [2])

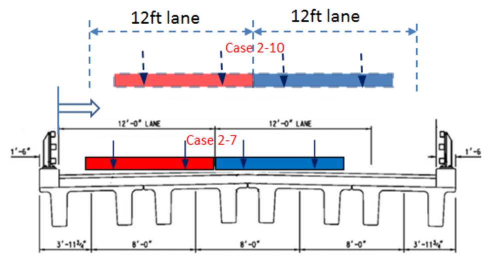


Fig. 12 Loading profile for Case 2-7
(note: bridge section adapted from [2])

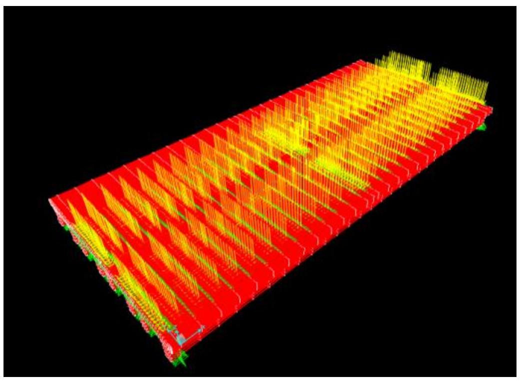


Fig. 13 Design lane load for Case 2-1

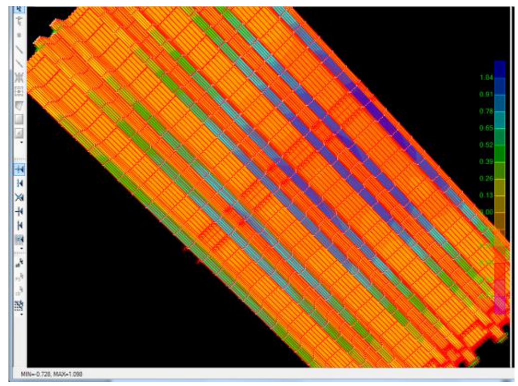


Fig. 14 Stress contour of S_{11} in bridge beams under combined loadings for case 2-1

Table 3 showed all the results of maximum S_{11} stress for each stem under the above 10 two-lane loaded cases.

Table 3 Maximum tensile stress in each stem under two-lane loaded cases

	Exterior Beam		Interior Beams				Exterior Beam	
	Beam 1		Beam 2		Beam 3		Beam 4	
	Stem 1	Stem 2	Stem 3	Stem 4	Stem 5	Stem 6	Stem 7	Stem 8
Case 2-1	**1.098**	1.050	**1.022**	0.953	0.908	0.800	0.731	0.610
Case 2-2	1.012	1.006	0.992	0.961	0.925	0.838	0.777	0.717
Case 2-3	0.932	0.958	0.962	0.955	0.933	0.871	0.824	0.733
Case 2-4	0.854	0.906	0.928	0.944	0.938	0.902	0.865	0.793
Case 2-5	0.792	0.863	0.901	0.938	0.945	0.930	0.911	0.855
Case 2-6	0.723	0.813	0.863	0.928	0.952	0.960	0.953	0.931
Case 2-7	0.978	1.000	1.006	0.984	0.942	0.864	0.766	0.644
Case 2-8	0.903	0.955	0.972	0.980	0.955	0.876	0.814	0.707
Case 2-9	0.835	0.907	0.940	0.972	0.962	0.909	0.859	0.767
Case 2-10	0.768	0.860	0.909	0.964	0.971	0.943	0.910	0.834

As can be seen from Table 3, case 2-1 governed all the other 2-lane loaded cases. The maximum tension stresses at the extreme bottom fiber of the beam are 1.098ksi and 1.022ksi for the exterior and interior beams, respectively. In accordance with AASHTO LRFD bridge design specifications, a multiple presence factor of 1.0 shall be applied to two-lane loaded case[4]. Therefore, for two-lane-loaded cases, the maximum tension stresses will be 1.098ksi and 1.022ksi for exterior and interior beams, respectively.

LIVE LOAD DISTRIBUTION FACTORS (LLDF) FOR MOMENT CALCULATED ON THE BASIS OF FE RESULTS

In AASHTO LRFD bridge design specifications, the flexural moment for the bridge girder under live loads is calculated by multiplying the lane moment with a live load distribution factor[4]. Thus, the LLDF for the beam can be determined through dividing the girder moment by lane moment. In this way, LLDFs for moment for both exterior and interior NEXT beams can be calculated with the above FE results, as shown in Table 4. Note 1.80ksi is the maximum S_{11} under lane moment as shown in Table 1 in this paper.

Table 4 Live load distribution factors for moment based on the FE results

	LLDFs (One-lane)	LLDFs (Two-lane)	Control LLDFs
Exterior beam	0.980/1.800=0.544	1.098/1.800=0.610	**0.610**
Interior beam	0.752/1.800=0.418	1.022/1.800=0.568	0.568

As can be seen, the live load distribution factor is governed by the exterior beam with all possible scenarios of design loadings on the bridge.

LIVE LOAD DISTRIBUTION FACTORS FOR MOMENT CALCULATED FROM LRFD SPECIFICATIONS

In AASHTO LRFD Bridge Design Specifications, the distribution of live loads per lane for moment in interior beams can be calculated from LRFD Table 4.6.2.2.2b-1[4], as shown in Table 5. “S” is the girder spacing, “L” is the span length, “t_s” is the deck thickness. $K_g = n(I + Ae_g^2)$[4].

Table 5 Distribution of live loads per lane for moment in interior beams[4]

<table>
<tr><td colspan="2">Live load distribution factors for moment in interior beams</td><td>Range of Applicability</td></tr>
<tr><td colspan="2">One design lane loaded: $0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$</td><td rowspan="2">$3.5 \leq S \leq 16.0$
$4.5 \leq t_s \leq 12.0$
$20 \leq L \leq 240$
$N_b \geq 4$</td></tr>
<tr><td colspan="2">Two or more design lane loaded: $0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$</td></tr>
<tr><td>Where,
$K_g = n(I + Ae_g^2)$,
$n = \frac{E_B}{E_D}$</td><td colspan="2">$10{,}000 \leq K_g \leq 7{,}000{,}000$
E_B = modulus of elasticity of beam materials, ksi
E_D = modulus of elasticity of deck materials, ksi
e_g = distance between the centers of gravity of the basic beam and deck, in.
I = moment of inertia of beam, in^4</td></tr>
</table>

In this study, S=8ft; L=68ft; t_s=8in.; N_b=4; the value of K_g can be calculated as of 831,490in^4 based on the section and material properties of the NEXT beam studied herein. Therefore the distribution for live loads per lane for moment in interior beams can be calculated based on the above equations, as shown in Table 6.

Table 6 Distribution factors of live loads per lane for moment in interior beams

	LLDFs (One-lane)	LLDFs (Two-lane)	Control
Interior beam	0.511	0.705	**0.705**

As can be seen from Table 6, the governed LLDF as determined from AASHTO equations is 0.705, which is larger than the control value of LLDF (i.e., 0.610) as calculated from FE modeling, as shown in Table 4. This indicated that treating the NEXT beam as Type "k" bridge shall give conservative LLDFs leading to a safe design.

The PCI Northeast Bridge Technical Committee suggested treating each stem as an individual beam and calculating the distribution factor for each stem based on the average stem spacing[3]. Then, the above factor is multiplied by two to calculate the LLDF for the entire NEXT beam section[3]. Table 7 shows the LLDFs per lane for moment in interior beams based on PCI northeast committee's method.

Table 7 LLDFs per lane for moment in interior beams based on PCI northeast committee

	LLDFs (One-lane)	LLDFs (Two-lane)	Control
Interior beam	0.638	0.825	**0.825**

As can be seen from Table 7, the governed LLDF is 0.825, which is much larger (approx. 35%) than the control value of LLDF as calculated from FE modeling. It appeared that the approach of treating each stem as an individual beam could lead to over-conservative LLDFs for the NEXT beam design.

CONCLUSIONS

In this paper, the live load distribution for moment in a NEXT beam bridge was investigated by 3-D finite element (FE) modeling in SAP2000. The FE results were compared to the manual solution based on AASHTO LRFD Bridge Design Specifications, indicating that both the exterior and interior beams of a bridge constructed with 8ft-wide NEXT beams can be designed with AASHTO type "k" LLDF for moment in interior beam. The current method recommended by PCI northeast bridge technical committee is too conservative for the case in this paper. Further study of the live load distribution factor for other type NEXT beams is under investigation by the author.

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