A NOTE ON SINGLE LANE LIVE LOAD DISTRIBUTION FACTORS FOR THE ALASKA STYLE BULB-TEE BRIDGES

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ABSTRACT

For the Alaska decked bulb-tee bridges, the AASHTO Specifications provide for one live load distribution factor (DF) equation regardless of number of loaded lanes. As a result, it is the practice of AKDOT&PF to use AASHTO multiple lane live load DFs for load rating. It seems that this practice results in a load rating penalty for Alaska Bulb-Tee girder bridges. A two-year research project has been initiated in Alaska with the objective of determining the appropriate distribution factors for load rating purposes. Similar to other slab-and-beam bridge systems, a single lane loaded DF formula for Alaska style decked bulb tee bridges should be specified in the AASHTO Specifications to consider the impact of loaded lanes on calculation of DF. The "Lever Rule" and "Slab Membrane Action" appear to explain the difference between single lane DF and multilane DF. And bridges for the field testing program have been identified.

Keywords: Load Distribution, Single Lane, Decked Bulb-Tee, Load Rating, In-situ Testing, AASHTO Specifications, Lever Rule, Slab Membrane Action

INTRODUCTION

The Alaska Department of Transportation (AKDOT&PF) uses AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications[1] for design and evaluation of Alaska's highway bridges. Most of the new bridges in the state are constructed from the Alaska decked bulb-tee girder. Because there is a longitudinal joint (hinge) between girders for this type of bridge, AASHTO lists this bridge under a different category when calculating live load distribution factors (DFs). According to the current AASHTO Specifications, there are two different live load DF equations for bulb-tee girder bridges other than the Alaska decked bulb-tee type. One equation is for single lane loaded, and the other for two or more lane loaded. For the Alaska decked bulb-tee bridges, the AASHTO Specifications provide for one live load DF equation. That equation was based on data from two or more lane loaded bridges. As a result, it is the practice of AKDOT&PF to use AASHTO multiple lane live load distribution factors for load rating. It seems that this practice results in a load rating penalty for Alaska Bulb-Tee girder bridges. So, a method for finding single lane distribution factors is needed for Alaska Bulb-Tee girder bridges.

REVIEW OF DISTRIBUTION FACTOR (DF) FORMULAS

Based on Newmark's research[2], the lateral wheel load distribution factors were determined by the expression:

$$g = \frac{S}{D} \tag{1}$$

where **g** = the wheel load distribution factor (DF); **S** = the center-to-center girder spacing (ft); and **D** = different constants for different bridge systems (ft).

Simple "S-over" live-load distribution factors have been used for bridge design since the American Association of State Highway Officials (AASHO) published its first edition of Standard Specifications for Highway Bridges in 1931. These factors allow the designer to uncouple transverse behavior from longitudinal behavior. However, live-load distribution provisions for multibeam precast concrete bridges (such as Alaska style decked bulb tee bridges) were not included in the specifications until 1965, when AASHTO published its ninth edition of Standard Specifications. In its ninth edition, the distribution criteria for multibeam bridges were only limited to a brief reference in the slab design section. Specifically, the distribution width per wheel is equal to 4.0+0.06L (L = Span) (ft) with a maximum of 7.0 ft.

AASHTO STANDARD SPECIFICATIONS, 12TH EDITION, 1977

In 1977, the distribution criteria for multibeam bridges was incorporated into "Distribution of Loads" section with other bridge systems. The DF formula for multibeam bridges took the same format (of Eq. (1)) as other bridge systems, with the following different definitions:

$$S = \text{effective girder spacing} = \frac{12N_L + 9}{N_g} \tag{2}$$

$$D = 5 + \frac{N_L}{10} + (3 - \frac{2N_L}{7})(1 - \frac{C}{3})^2 \qquad C \le 3$$
$$= 5 + \frac{N_L}{10} \qquad C > 3 \tag{3}$$

where N_L = total number of design traffic lanes; N_g = number of longitudinal beams; and C = a stiffness parameter that depends on the type of bridge, bridge and beam geometry, and material properties, calculated based on the following:

$$C = K\frac{W}{L} \tag{4}$$

$$K = \sqrt{\frac{E}{2G}\frac{I_l}{J_l + J_t}} \tag{5}$$

where W = the overall width of the bridge (ft); L = span length (ft); EI_1 = flexural stiffness of the transformed beam section per unit width; GJ_1 = torsional stiffness of the transformed beam section per unit width; and GJ_t = torsional stiffness of a unit width of bridge deck slab.

These DF formulas for multibeam bridges were proposed by Sanders and Elleby in NCHRP Report 83[3]. The multibeam criteria were, as most criteria, based on no reduction in load intensity (i.e., without considering the multiple presence factor).

UNIVERSITY OF WASHINGTON STUDY

The only stemmed members addressed in NCHRP Report 83 were channels. Considering sections such as double tees, bulb tees, single tees, as well as decked bulb tees have come into common use for bridges, the University of Washington performed the NCHRP 12-24 study on load distribution for precast stemmed multibeam bridges[4]. The specific objectives of that research were to investigate the distribution of truck wheel loads in the decks of bridges made from single-stem and multi-stemmed precast concrete tee-shaped members, and to make recommendations for their design in a form suitable for inclusion in the AASHTO Standard Specifications. The following DF formulas were proposed in the final NCHRP Report 287 [4]:

$$S = \text{width of precast member} \tag{6}$$

$$D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2 \qquad C \le 5$$
$$= (5.75 - 0.5N_L) \qquad C > 5 \tag{7}$$

$$C = K\frac{W}{L} \tag{8}$$

$$K = \sqrt{\frac{EI}{2GJ}} \tag{9}$$

where EI = flexural stiffness of each girder; GJ = torsional stiffness of each girder; and others are the same as before.

Comparing Eqs (6)-(9) with Eqs (2)-(5), the following changes are noted: (1) The former use the effective girder spacing while the later use the actual girder spacing. (2) There is a difference in calculating the stiffness parameter K. (3) The wheel load fractions from both sets of formulas give nearly identical results for small C values (i.e., long narrow bridges made from torsionally stiff members). D increases when C decreases. This is because torsionally stiff members deflect under load but twist little, thereby causing adjacent members to deflect as well, spreading the load into them. However, for large C values (i.e., short wide bridges made from stemmed members), the 1977 AASHTO relationships (Eqs (2)-(5)) predict significantly larger D values. Finally, (4) Eqs (6)-(9) consider bridges with skew angles up to 45 degrees while Eqs (2)-(5) do not take skew into account. For skewed bridges, bridge width W is measured perpendicular to the longitudinal girders and bridge span L is measured parallel to longitudinal girders in Eqs (6) – (9).

AASHTO STANDARD SPECIFICATIONS, 16TH EDITION, 1996

The current edition of Standard Specifications[5] has the same DF formulas as Eqs (6) – (9). The current specifications state that if the value of $\sqrt{\frac{I}{J}}$ exceeds 5.0, the live load distribution should be determined using a more precise method, such as the Articulated Plate Theory or Grillage Analysis.

It also states that for non-voided rectangular beams, channels, and tee beams, Saint-Venant torsion constant "J" may be estimated using the following equation:

$$J = \sum \{\frac{bt^3}{3}(1 - 0.630\frac{t}{b})\} \tag{10}$$

where b = the length of each rectangular component within the section; t = the thickness of each rectangular component within the section. The flanges and stems of stemmed or channel sections are considered as separate rectangular components whose values are summed together to calculate "J".

The current Standard Specifications also require full-depth rigid end diaphragms to ensure proper load distribution for channel, single- and multi-stemmed tee beams.

AASHTO LRFD SPECIFICATIONS, SECOND EDITION WITH 2001 INTERIM

The LRFD Specification[1] contains the same provisions for load distribution for "multi-beam decks which are not sufficiently interconnected to act as a unit," as appeared in recent editions of the Standard Specifications.

Some of the changes are as follows: (1) Instead of using wheel load fraction, as in Standard Specifications, LRFD Specs use lane load fraction. Thus, "D" value from LRFD is twice as much as the one in Standard Specs. (2) There is no range of applicability specified in LRFD Specs other than that the number of beams is not less than four, beams are parallel and have approximately the same stiffness, and the stem spacing of stemmed beams is more than 4 ft or less than 10 ft. (3) The multiple presence factors in LRFD Specs are different from those in Standard Specs. (4) The St. Venant torsional inertia, J, may be determined as:

$$J = \frac{1}{3}\sum bt^3 \qquad \text{For thin-walled open beam} \qquad (11)$$

$$J = \frac{A^4}{40.0 I_p} \qquad \text{For stoky open sections (such as T-beams)} \qquad (12)$$

where A = area of cross-section; and I_p = polar moment of inertia. (5) The load fraction formulas for the interior and exterior beams are the same in Standard Specifications, while the lane load fraction for exterior beams is based on "Lever Rule" in LRFD Specifications; (6) Similar to Standard Specs, there is no correction factor available for skewed bridges in LRFD Specs. (7) Distribution factor method for shear is recommended to use "Level Rule". Finally, (8) there are no correction factors for load distribution factors for support shear of the obtuse corners of the skewed bridges.

The AASHTO LRFD Specifications recommend using the "Lever Rule" – a method of determining the live-load shear carried by a single girder assuming that the deck acts as a simply supported span between girders. Using the "Lever Rule" results in two perceived problems: (1) The "Lever Rule" is invalid for Alaska Decked Bulb-Tee Girders. The deck formed by these girders has a longitudinal joint midway between adjacent girders. This longitudinal joint acts in a manner similar to a hinge. The assumption of hinges over the girders would result in an instability in the system using the "Lever Rule". And (2) the "Lever Rule" method may be overly conservative for analyzing Alaska Decked Bulb-Tee Girders.

IMPACT OF SINGLE LANE LOADING

One of the key issues in load rating of bridges is the realistic calculation of live load distribution factor for single lane loading conditions. The live load distribution factor for load rating purposes can be different from the distribution factor for bridge design. Use of the multiple lane distribution factor will over-estimate the live load carried by a girder due to single lane loading, resulting in a reduction in the allowable live load carried by the bridge, and the "operating" or maximum bridge live load capacity is reduced.

AASHTO SPECIFICATIONS

According to the current AASHTO Specifications, there are two different live load DF equations for most bridges. One equation is for single lane loaded, and the other is for two or more lane loaded. Regardless of number of loaded lanes, however, the same D value is used for precast concrete beams used in multibeam decks, including the Alaska style decked bulb-tees.

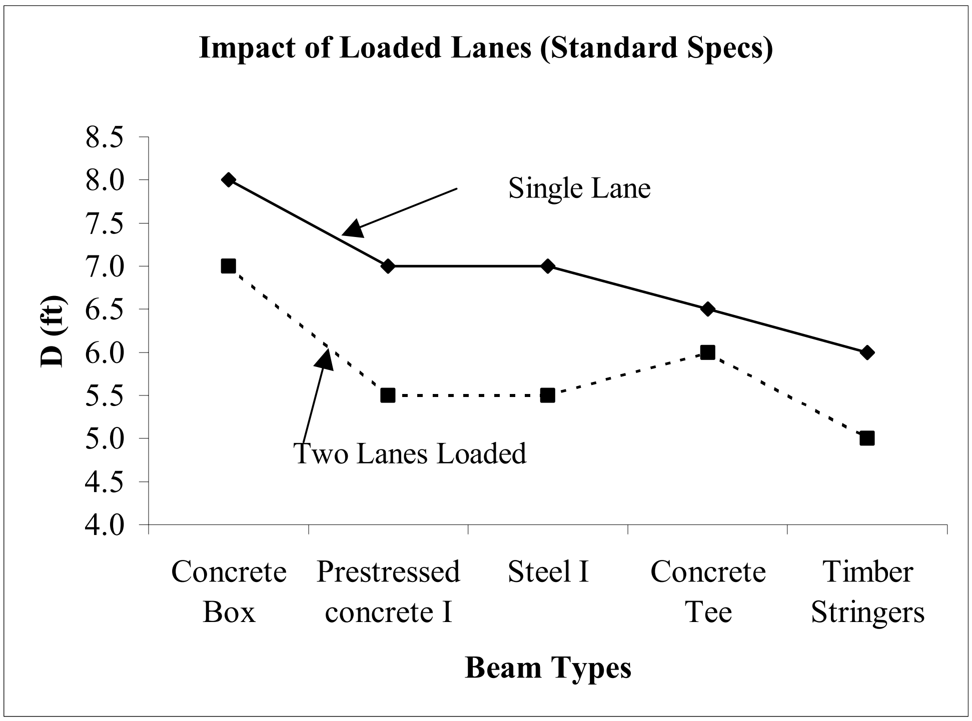


Fig.1 Impact of Loaded Lanes on "D" Values (AASHTO Standard)

Figure 1 shows the impact of the number of loaded lanes on "D" values in the "S-over" live-load distribution factor for different bridge systems based on AASHTO Standard Specifications. Several observations can be drawn from Figure 1. First, D increases for all five bridge systems considered when the same bridge is changed from two lanes loaded into single lane loaded. A larger D value suggests better live-load distribution. The degree of increase in D values for different bridge systems is different. The D value of single lane loaded prestressed concrete and steel I-beams is about the 127% of D value of the two lanes loaded counterpart. For a concrete tee beam system, the D value is only increased by 8%.

The second observation is that for both single lane loaded and two lanes loaded bridge systems, the multi-cell concrete box system has the highest D values and the timber stringer system, the lowest.

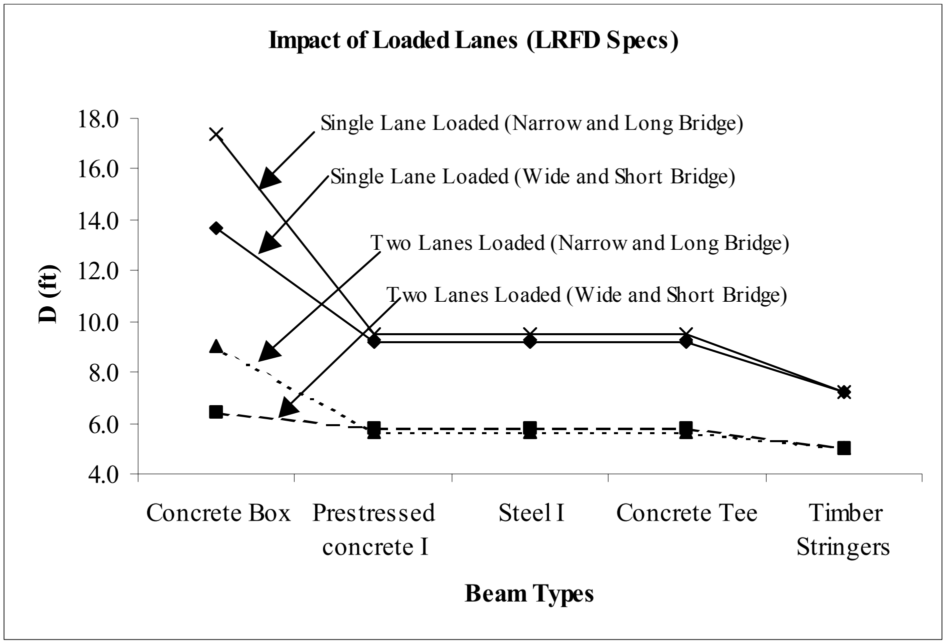


Fig.2 Impact of Loaded Lanes on "D" Values (AASHTO LRFD)

Figure 2 shows the impact of number of loaded lanes according to AASHTO LRFD Specifications. Based on NCHRP Project 12 – 26[6], new, more accurate, and more complex live-load distribution factor equations were developed and proposed to AASHTO as replacements for the simple "S-over" factors in AASHTO Standard Specifications. These equations are included in the LRFD Specifications. Other changes in LRFD Specifications include: (1) The multiple presence factor of "1.2" is applied to single lane loaded bridges. (2) The lane distribution factor is used in LRFD instead of the wheel load distribution factor. And (3) the lane distribution factor in LRFD depends on stiffness parameters, and width and span of bridges, as well as the girder spacing parameter, as in Standard Specifications.

In order to facilitate comparison, the lane load distribution factor has been converted to a common basis in the format of "S-over" formula. In calculating the equivalent "D" values shown in Figure 2, bridges are grouped into "wide and short" and "narrow and long" categories according to the range of applicability specified in LRFD Specs. In the first category, the high range of girder spacing and the low range of span are used. And the low range of girder spacing and the high range of span are applied in the second category. Other assumptions used are: (1) The number of cells is 6 for concrete box girder bridges. And (2) $\frac{K_g}{12Lt_s^3}$ is assumed to be equal to 1.0 for deck-and-slab bridges. In order to convert to a value free of multiple presence factors, the D values are multiplied by 1.2 for the single lane loaded bridges.

Comparing Figure 2 with Figure 1, some similar observations can be found. However, the following conclusions can be also drawn from Figure 2: (1) Except for concrete box bridge systems, differences in D values between "wide and short" and "narrow and long" bridges are not significant. And (2) improvement in load distribution for single lane loaded bridges is even better according to LRFD Specs (i.e., $D_{single\text{-}lane}$ value is much larger than $D_{two\text{-}lane}$ value), as shown in Figure 3.

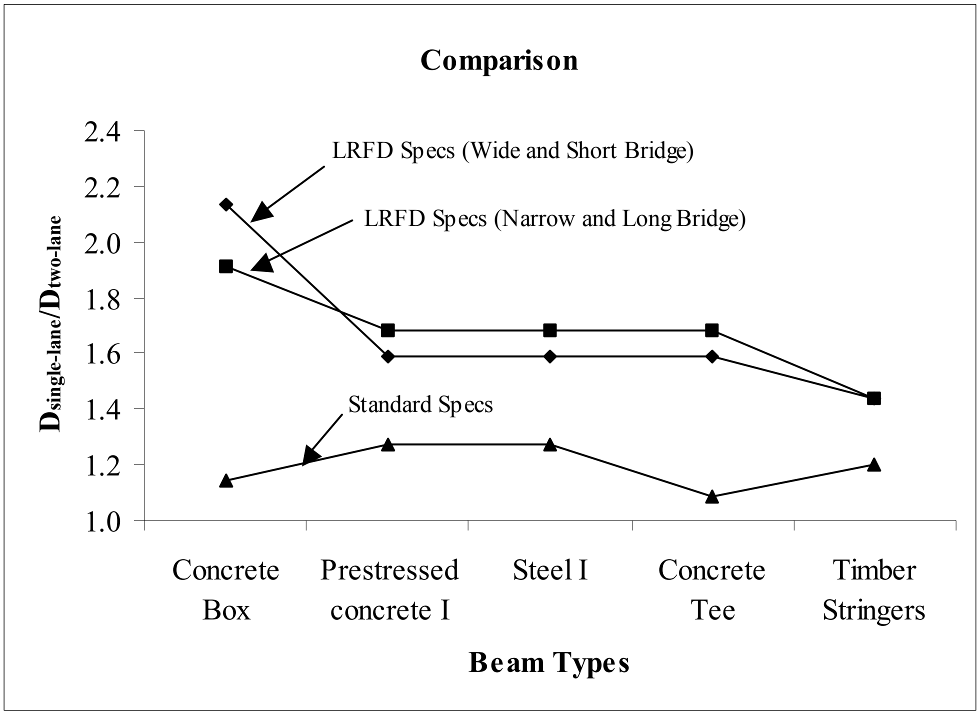
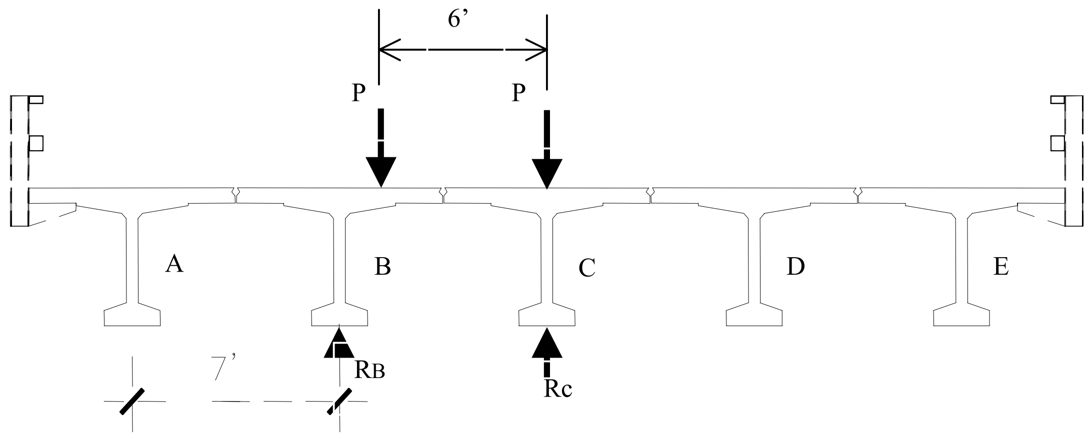


Fig. 3 Comparison between $D_{single\text{-}lane}$ and $D_{two\text{-}lane}$ according to AASHTO

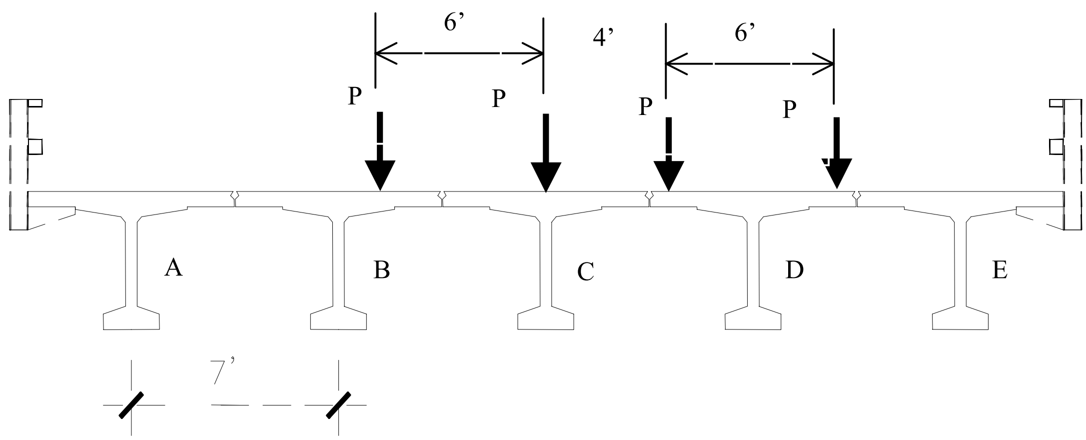
Obviously, there exists a difference between AASHTO Standard Specs and AASHTO LRFD Specs as shown in Figure 3. In general, LRFD Specs predict a higher ratio of D values, especially for the concrete box bridge system, than Standard Specs. It also appears that the bridge geometry plays a very important role in calculating the live-load distribution factor for single lane loaded bridges according to LRFD Specs.

Discussions

As shown in Figure 3, the single lane DF will be about 78% of the multiple lane DF factor based on AASHTO Standard Specifications, and even lower DF for single lane based on AASHTO LRFD Specifications. This seems reasonable from the perspective of the "Level Rule," as shown in Figure 4.



(a) Single Lane Loaded



(b) Two Lanes Loaded

Fig. 4 Free Body Diagram – Lever Rule Method

Consider Figure 4 (a). The deck is assumed to be simply supported by each girder except over the exterior girders A and E where the cantilever is continuous. If we consider one lane loaded, the reaction at C (Rc) is established by balancing the moment about B.

Rc (7) = P (7) + P (7-6)

which reduces to

Rc = P + P/7 = 1.143 P

The fraction of the single lane that is carried by the Girder C is 1.143P/(2P) = 0.572. Thus, the girder distribution factor is 0.572 (without the multiple presence factor).

The distribution factor for the same Girder C subjected to two loaded lanes is established by considering trucks positioned with axles on deck panels BC, CD, and DE, as shown in Figure 4 (b). Equilibrium requires that the reaction at C is

Rc = (P + P/7) + [(7-4)/7]P = 1.572P

And the distribution factor (without considering the multiple presence factor) is 1.572P/(2P) = 0.786, which is larger than the distribution factor for single lane loaded.

The above discussion is based on the “Level Rule” assumption, which may or may not apply to all bridge systems. The possible load distribution mechanisms between single loaded lane and multiple loaded lanes still need to be studied. In the traditional “S-over” approach, it is assumed that for consideration of longitudinal bending, the slab can be thought of as a series of strips, each forming a top flange of a T-beam. No check has been made to confirm that after notionally cutting up the deck the displacements of the parts are compatible, i.e. that the parts can in fact be joined together without additional forces and distortion hitherto not considered.

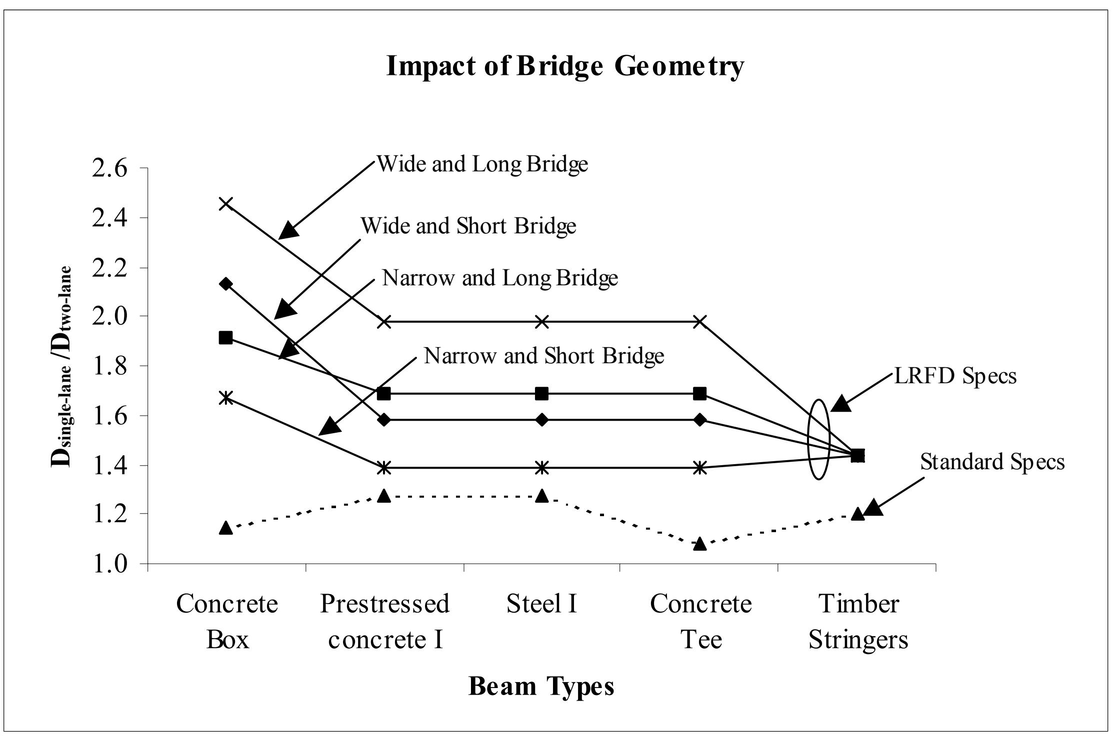


Fig. 5 Impact of Bridge Geometry on $D_{single\text{-}lane}/D_{two\text{-}lane}$

If all separated “T-beams” flex about a neutral axis passing through their centroids, the ends of the slab flanges are displaced relative to each other. In reality this step displacement cannot happen, and the relative movement of the tops of the “T-beams” is resisted and reduced by longitudinal shear forces in the connecting slab. This is also referred to as “slab membrane action”[7]. These shear forces are in equilibrium with axial tension/compression forces in beams near midspan. The forces have two effects on deck behavior. First, the axial tension forces in the beams with the largest deflections (i.e. under the load) cause the neutral axis to rise locally while compression forces elsewhere cause the neutral axis to move down. Secondly, the load distribution characteristics of the deck are improved. The longitudinal interbeam shear forces and axial forces are at different levels and thus form couples which reduce the moment in the loaded beams and increase moments elsewhere. This explains why single lane loaded bridges have better live-load distribution characteristics than multi-lane loaded ones. Single lane loads cause larger deflection differences between “separated T-beams” than multi-lane loads. Also, wider slabs have larger in-plane bending resistance, and thus larger interbeam shear forces. Longer span bridges tend to have larger deflection differences than short span bridges. Figure 5 shows the impact of bridge geometry on the ratio of D values based on LRFD Specs. It appears to support the above discussion.

MULTIBEAM (e.g. DECKED BULB TEE) BRIDGES

For the Alaska decked bulb-tee bridges, the AASHTO Specifications do not consider the impact of a single lane loaded case, and they only provide for one live load DF equation, regardless of the number of loaded lanes. As a result, it is the practice of AKDOT&PF to use AASHTO multiple lane live load distribution factors for load rating. It seems that this practice results in a load rating penalty for Alaska Bulb-Tee girder bridges. So a method for finding single lane distribution factors is needed for Alaska Bulb-Tee girder bridges.

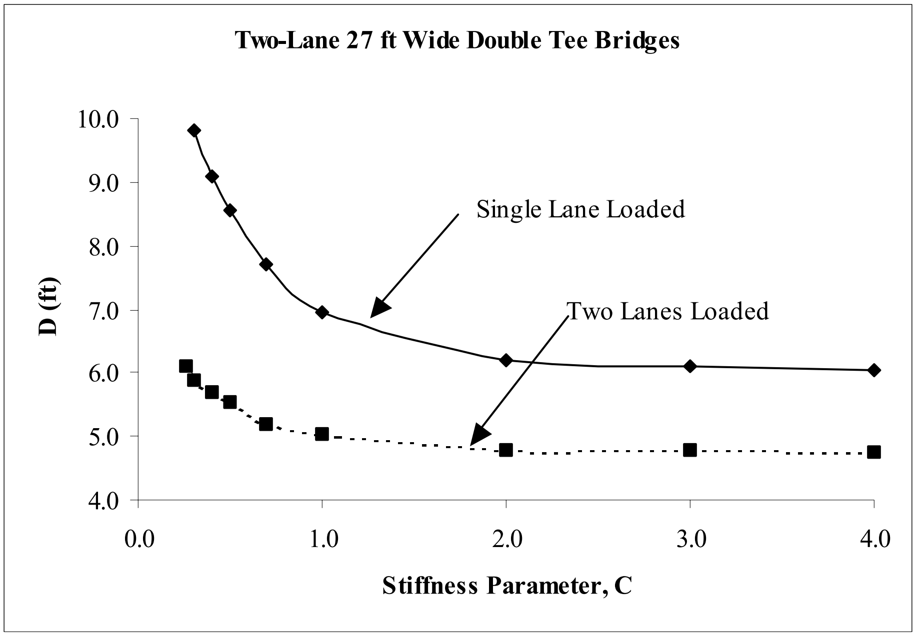


Fig. 6 Parameter Studies Considering Number of Loaded Lanes

As discussed before, the live-load distribution factor equations for multibeam bridges in AASHTO Specifications were based on study performed at the University of Washington (UW)[4]. In the UW study, six bridge widths were considered (27, 36, 39, 48, 51, and 60-ft). The final equations were based on the lowest D values from the multi-lane loaded cases.

By re-assembling the parameter studies performed in the original UW study, we have found that the D value for a single lane loaded 27 ft wide double tee bridge is about 1.28 times the D value of the same bridge with two lanes loaded, as shown in Figure 6.

FIELD TESTING PROGRAM

Field loading testing generally gives a realistic determination of the distribution. Most of tests were conducted on beam-and-slab bridges. The most extensive single effort of field testing was conducted at the AASHO Test Road[8]. However, only three full-scale tests of the type of multi-beam bridges studied have been reported[9,10,11]. The first test[9] was conducted on a bridge consisting of channel sections; the second[10] on a bridge with solid sections with holes; and the third[11] on a bridge composed of solid sections.

In order to check the analysis results, a field testing program has been designed under the current research project at the University of Alaska Fairbanks (UAF). Table 1 shows the selected eight Alaska style decked bulb-tee bridges. In selecting the bridges to instrument, UAF researchers considered the following factors. (1) They are all located in or near Anchorage, Alaska. (2) Traffic can be closed during late night hours for all these bridges. (3) They are all accessible to instrument. And (4) they represent different geometry of the bridges in Alaska in terms of skew angles and aspect ratio (length/width). Researchers have also decided to test paired structures to provide verification of the instrumentation and modeling procedures. Based on these factors, four pairs of bridges have been identified, as shown in Table 1. Figure 6 shows the twin bridge structures on Huffman Road in Anchorage, Alaska.

Fig. 5 Huffman Road Bridges, Anchorage, Alaska

Current field testing plans are: (1) A finite element model will be used to simulate field tests. This will enable the researchers to plan the field test, predetermine number and type of sensors needed, and select the number of tests. (2) Use the laboratory to wire, calibrate and test sensors, electronic cables, and data acquisition equipment. (3) Once in the field, sensors to measure strain and deflection will be mounted to the bridge structure. The BDI Intelliducer^{TM} 370 will be used for this study. Web gauges will be needed to determine what portion of the load distribution is taken by shear. It is anticipated that the BDI Automatic vehicle position indicator will be used to record vehicle location. Finally, (4) after tests are run, influence lines will be plotted and the distribution factors will be evaluated.

CONCLUSIONS

For the Alaska decked bulb-tee bridges, the AASHTO Specifications provide for one live load distribution factor (DF) equation regardless of number of loaded lanes. Based on this study, researchers have found that AASHTO provisions result in a load rating penalty for Alaska decked bulb-tee girder bridges. Similar to other slab-and-beam bridge systems, a single lane loaded DF formula for Alaska style decked bulb tee bridges should be specified in the AASHTO Specifications to consider the impact of loaded lanes on calculation of DF. The "Lever Rule" and "Slab Membrane Action" appear to explain the difference between single lane DF and multilane DF. And bridges for the field testing program have been identified.

Table 1 Selected Bridges to Test

Bridge Name	Bridge No.	Length (ft)	O-O Width (ft)	Aspect Ratio	Road Width (ft)	No. Girders	~Bf width	Skew Angle (deg)	No. Spans	Girder Depth (in)	Depth/ Width
Campbell Creek SB	1443	140	37	3.8	36	5	88.4	4.3	1	66	0.75
Campbell Creek NB	1694	140	37	3.8	36	5	88.4	4.3	1	66	0.75
Huffman NB	1441	128	37	3.5	36	5	88.4	27.5	1	54.5	0.62
Huffman SB	1442	128	37	3.5	36	5	88.4	27.5	1	54.5	0.62
West 100 Ave NB	1695	116	37	3.1	36	5	88.4	0	1	54	0.61
West 100 Ave SB	1603	116	37	3.1	36	5	88.4	0	1	54	0.61
Dimond Blvd	1325	110	105	1.0	100	14	89.5	0	1	53	0.59
Dowling Road	1324	110	105	1.0	100	14	89.5	0	1	53	0.59
TOTAL AVERAGE		124	54.0	2.9	52	7	88.7	8.0		56.9	0.64

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