Evaluation of Live Load Distribution Factors in Integral Bridges Using the Finite Element Method

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Abstract— *This paper investigates the adequacy of both AASHTO Standard Specification and AASHTO LRFD girder distribution factors (GDF) for use in the design of integral abutment bridges (IABs). A three-dimensional finite element model (FEM) was developed of an integral bridge to assess its GDFs. Vehicular live loading was applied in one, two, and three lanes in the FEM. The stresses corresponding to each case was utilized to compute the GDFs for each girder. The GDFs obtained from one lane loading were closer to those obtained from the AASHTO LRFD code than to AASHTO standard equation, which was overly conservative in all cases. The AASHTO GDF equations were excessively conservative for multiple lanes. The effect of several parameters such as, the number of piles, girder spacing, and boundary conditions on the GDF ratios was studied. The GDF ratios decreased in accuracy as the girder spacing decreased for one lane loaded. Both AASHTO 1996 and 2012 are more conservative in estimating simply supported bridge GDFs compared to an equivalent IAB.*

Keywords— Girder Distribution factors; Integral Bridges; Steel girders; finite element modeling; vehicular live load.

I. INTRODUCTION

In early bridge construction, up until the 1960's, bridges were primarily designed using expansion joints (Arsoy, 1999). This was the best known way to account for thermal expansion/contraction of bridges, but after some time the bridge joints began to become ridged due to expansion joint failures such as corrosion. One of the primary reasons IABs have seen an increase in use is due to the common problems found in the expansion joints of standard bridges. These problems can include: 1)leaky expansion joints and seals letting in roadway water runoff, thus corroding the expansion joint bearings (Mistry, 2005). As engineers observed these improperly functioning bridges, they realized that a bridge could still adequately perform without the use of expansion joints (Mourad, 1999).

The IAB state design procedures vary widely while the majority of states are increasingly building and retrofitting bridges to function as IABs (Olson, 2009). IABs are designed and built using no expansion joints to account for thermal changes. The abutments are rigidly attached to the girders and deckcausing the entire bridge to move as one unit as external thermal and vehicular loads are applied.The most common problem associated with IABs is the abutment movement due to temperature changes causing backfill settlement and cracks in the approach slabs. This has created many research opportunities to study approach fill settlement along with soil-pile interaction. Since this has been extensively studied, the following research was done to better understand the accuracy of both the 1996 AASHTO Standard Specifications and the 2012 AASHTO LRFD specifications in calculating girder distribution factors (GDF) in a steel girder IAB. Suksawang et al. (2013) developed a simplified S-over load distribution factor (LDF) for shear in steel and prestressed concrete I-girder bridges. Three-dimensional finite element models of the studied bridges were verified using field testing of seven I-girder bridges. A parametric study with various girder spacing and lengths was conducted to develop the proposed equations. It was concluded that the proposed equations were able to accurately predict the GDFs predicted by the finite element analysis. Sukswang and Nassif (2007) initiated an experimental program that consisted of five steel I-girder bridges. Additionally, a detailed parametric study using the finite element analysis was performed. The researchers developed new simplified GDFs equations for various types of bridges including steel, prestressed, and spread box girder bridges. Barr and Amin (2006), used a full scale, single lane test bridge to study the shear response for a typical slab-on-girder bridge's response. The results from the bridge testing were used to validate the finite element model. Several models were used to investigate several design parameters such as girder spacing, span length, overhang distance and skew angle on shear live-load distribution factors. It was concluded that shear GDFs calculated by AASHTO LRFD equations are accurate.

https://dx.doi.org/10.24001/ijcmes.3.4.2 ISSN: 2455-5304

Although the accuracy of the new equations adopted by AASHTO has been investigated by many researchers ((Mabsout andTarhini, 1997a; Mabsout and Tarhini, 1997b; Barr *et al*., 2001; Schwarz and Laman, 2001; Eamon and Nowak, 2002; Nassif *et al*., 2003; Barr and Amin, 2006; and Huo*et al.*, 2005) who concluded that the use of these equations is conservative compared to field testing and finite element analysis, there has not been much research done on integral abutment bridges.

The objective of this study is to evaluate the accuracy of both AASHTO Standard Specifications and AASHTO LRFD in predicting girder distribution factors in integral abutment bridges. A parametric study was performed to study the sensitivity of crucial bridge parameters on the values of GDFs in integral bridges.

II. BRIDGE DESCRIPTION

The Scotch Road Bridge is a two-span 90.9 m integral abutment bridge located in Trenton, NJ. The bridge was fully instrumented and monitored from April, 2003 to May, 2006 by Hassiotis et al., 2006. Figs.1 and 2 are schematic diagrams showing the bridge layout and geometrical configurations. The bridge has six lanes, three in each traffic direction.

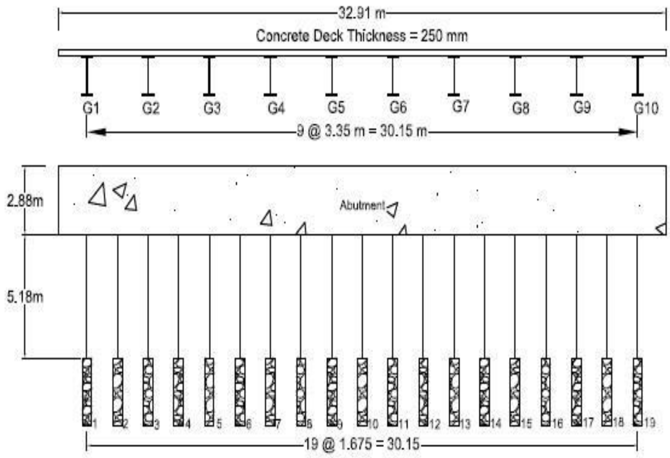

Fig.1: Cross sectional layout of the Scotch Road Bridge(Brendler and Khodair2016)

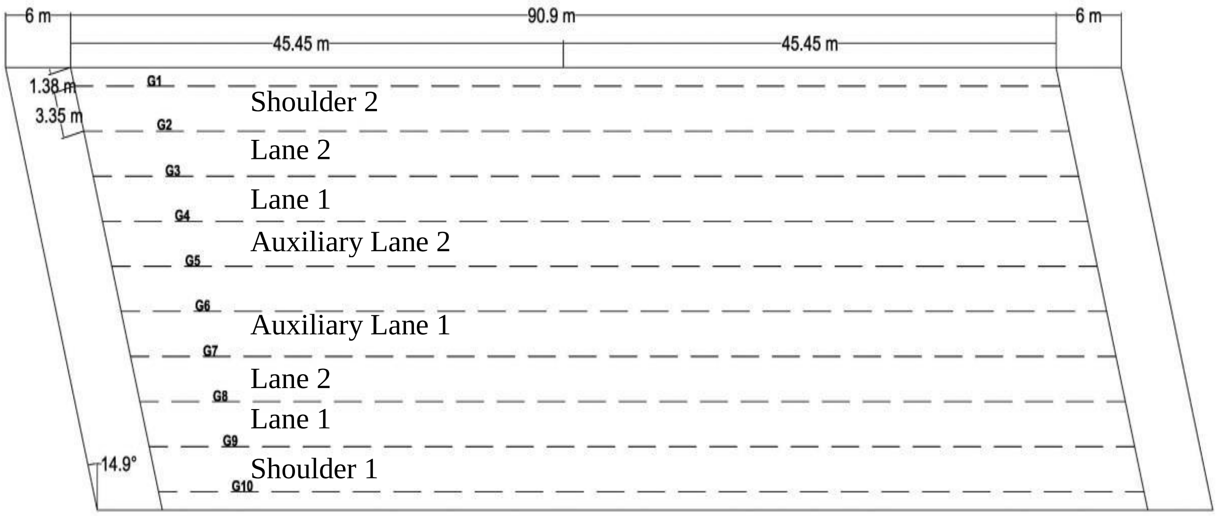

Fig.2: Plan view of the Scotch Road Bridge.

III. FINITE ELEMENT MODEL

A three-dimensional finite element model (FEM) was developed using Abaqus/Cae as shown in Fig. 3. Shell elements (S8R5) were used to model the bridge deck, approach slab, and abutments. The deck, approach slab, and abutment also included a smeared rebar layer internally embedded in the shell elements (Abaqus, 2013).Although the girders and piles have different cross sectional dimensions, they wereboth modeled using B32 elements where B represents a beam element, 3 = the number of nodes per element, and2 indicates quadratic analysis (Abaqus, 2013). Another component of the modelwasthe connections used to achieve composite action of the deck and girder in addition to fixity atthe abutment, deck, and

https://dx.doi.org/10.24001/ijcmes.3.4.2 ISSN: 2455-5304

girder intersections. A beam multi point constraint (MPC) was used to connect the girder to the deck spaced every 0.505m along the length of the bridge as dictated by the model meshing.

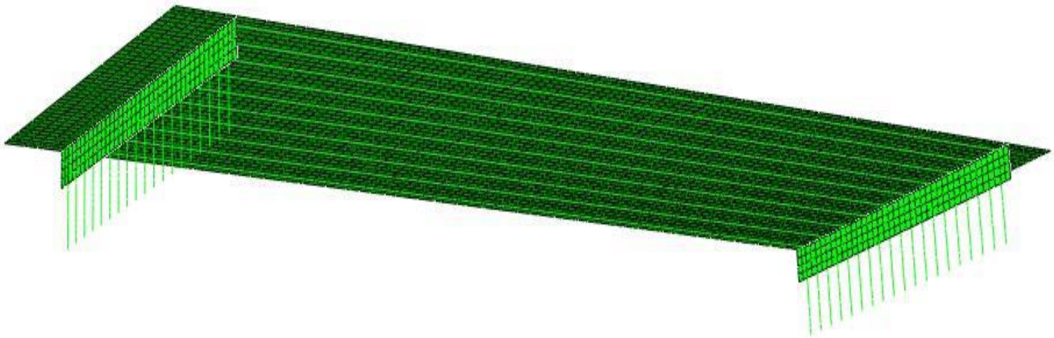

Fig.3: Undeformed shape of the FEM of the Scotch Road Bridge using Abaqus/Cae.

The beam MPC creates a rigid link between the two nodes being connected, thus constraining both displacement and rotation of a master node, the deck, to a slave, the girder (Abaqus, 2013), to model the composite action between the deck and supporting girders. The connection used to achieve fixity at the location of the girder ends, deck, and top of the abutments was a combination of beam MPC and non-MPC tie constraints. Beam MPCs were used to connect the deck directly to the abutment at all locations without a girder intersection. Tie constraints were then used to rigidly connect the girders to the abutment. Similar to the beam MPC, the tie constraint rigidly connected all degrees of freedom of a master and slave node, but behaves well where there is no space modelled between the elements as with the abutment girder intersection. The same tie constraints were used to attach the tops of the piles to the bottom of the abutments. Finally the springs and boundary conditions were added to the model before validation testing. The boundary condition at the center of the two-span bridge was treated as a roller, and the bottom of the piles were fixed. The pile length exceeded the typically accepted minimum distance of 4.5 m below the soil surface to behave as a fixed base (Davids, 2010). Springs were located along the piles, behind the abutment, and under the approach slabs to model the soil. The springs were adjusted based on the thermal validation of the Scotch Road Bridge to match the field data.

IV. VALIDATION OF THE FINITE ELEMENT MODEL

Hassiotis et al. (2006) recorded the temperature variation at the top of girder 5 and the displacement at the sleeper slab over a three-years period. The temperatures recorded from the Scotch Road Bridge were applied to the Abaqus FEM and the resulting displacements were analysed. Several iterations were implemented by adjusting the spring coefficients of the FEM model for each iteration until the displacement obtained from the model closely matched those of the field data using a k = 10,000 kN/m^{2}as shown in Fig. 4.

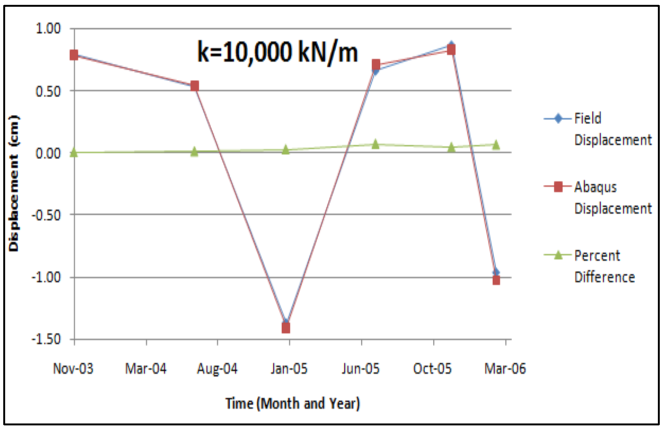

Fig. 4: Comparison between measured and FEM displacements versus time (Brendler and Khodair, 2016).

V. DISTRIBUTION FACTORS

The objective of this research is to compare the GDFs calculated based on both the 1996 AASHTO Standard Specifications and the 2012 AASHTO LRFD Bridge Design Specifications to ensure that the formulas adopted by AASHTO are adequate for IABs. The original AASHTO Standard equation was simple and neglected many factors associated with bridge design. The original GDF for one-lane loaded is

$$GDF_{1\text{-lane}} = \frac{S}{4.27} \qquad (1)$$

while two or more lanes loaded is

$$GDF_{multiple} = \frac{S}{3.36} \qquad (2)$$

where S = the girder spacing (m).

AASHTO LRFD (2012) adopted GDFs equations incorporating additional factors for span length, overall bridge stiffness, deck thickness, and a correction factor for skew over 30°. The equation developed for one lane loaded is

https://dx.doi.org/10.24001/ijcmes.3.4.2 ISSN: 2455-5304

$$\mathrm{GDF} = 0.06 + \left(\frac{S}{4.3}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s{}^3}\right)^{0.1} \qquad (3)$$

and for two or more lanes

$$\mathrm{GDF} = 0.075 + \left(\frac{S}{2.9}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s{}^3}\right)^{0.1} \qquad (4)$$

with skew correction factor for both equations of

$$c_1 = 0.25 + \left(\frac{S}{L}\right)^{0.5}\left(\frac{K_g}{Lt_s{}^3}\right)^{0.25} \text{for } 30^o < \theta < 60^o \qquad (5)$$

$$c_1 = 0 \text{ for } \theta < 30^o \qquad (6)$$

where S = girder spacing (m), L = span length (m), t_s = slab thickness (m), K_g = longitudinal bridge stiffness = $n(I + Ae_g^2)$ (m^4), n = ratio of the beam to deck modulus of elasticity, I = moment of inertia of girder, A = girder area, and e_g = distance between the center of gravity of the deck and girder.

Table 1 illustrates the extreme variation between the AASHTO 1996 and 2012 calculation techniques. The 1996 AASHTO standard specification is significantly more conservative than the current 2012 AASHTO LRFD.

VI. LOADING

The Scotch Road Bridge was loaded using the standard HS20-44 design truck. The load was applied to the bridge lanes of the three-dimensional (3D) FEM resulting in the deformation of the bridge as shown in Fig.5. A total of three cases were analysed: (1) one truck in lane one, (2) two trucks in lanes one and two, and (3) three trucks in lanes one, two, and the auxiliary lane. The critical truck location causing the maximum stress summed across all ten girders was found and used in calculating the GDFs for all girders.

Table.1: GDFs Calculated for the Scotch Road Bridge

Interior Moment Distribution Factors		
AASHTO Version	**Lanes Loaded**	**Factor**
1996	1	0.7588
	2	0.9643
2012	1	0.4761
	2	0.7236

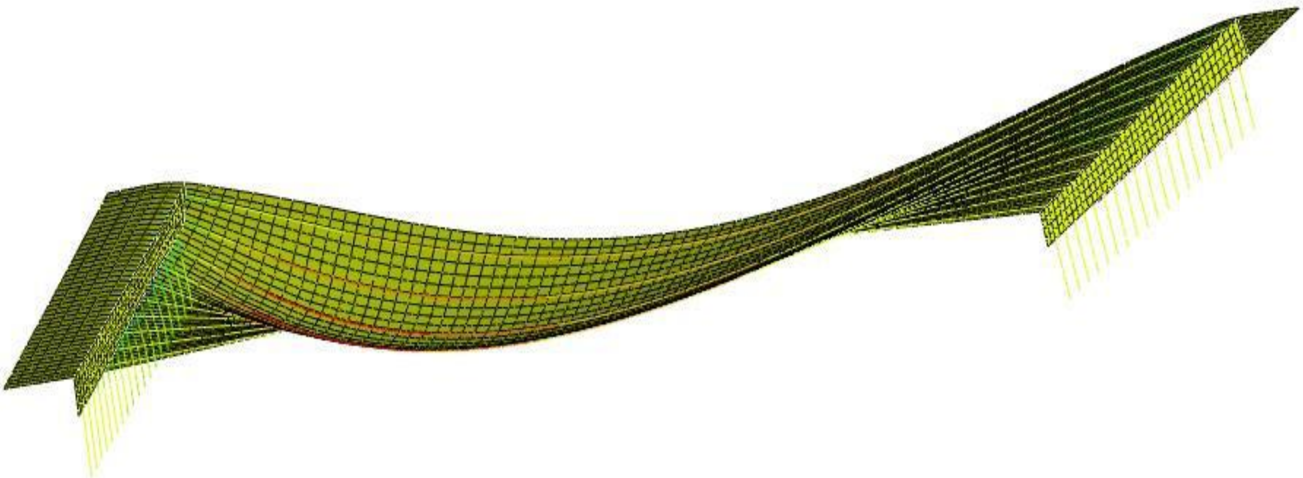

Fig.5: Deformed shape of the FEM showing the stress variations due to the application of the HS-20-44 design truck.

VII. FE GIRDER DISTRIBUTION FACTORS CALCULATIONS

The GDFs were calculated by dividing the stress at a single girder by the sum of themaximum flange stresses (Eom, 2001), Equation 8.

$$GDF_i = \frac{f_i}{\sum_{j=1}^{k} f_j} \qquad (8)$$

Where GDF_i is the distribution factor at the Ith girder, f_i = the stress at the girder under consideration, k = the total number of girders.

VIII. RESULTS

After loading the bridge the stress data was collected at four integration points, two at the top and two at the bottom of each girder flange. The top and bottom points were then averaged respectively to obtain the axial stress at the top and bottom of the girder.

Fig6 shows GDFs ratios for individual girders for one lane loaded. The FEM calculated GDFs were divided by both the AASHTO 1996 and AASHTO 2012 GDFs to obtain the calculated ratios. As expected, the distribution factor was essentially zero in girders far from the truck loading. Moreover, the closer the individual girder ratio is to one, the more accurately AASHTO represents the actual loading distribution calculated by Abaqus/Cae.

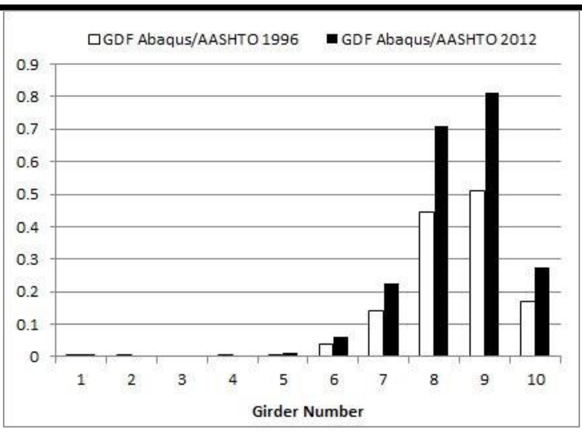

Fig. 6: GDFs ratios for one truck loaded in lane one(Brendler and Khodair, 2016).

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Lane one loading provided the most reasonable results for AASHTO LRFD, while the AASHTO standard equation was overly conservative in all cases. The Abaqus calculated GDF was only about 20% lower for one lane loaded compared to AASHTO LRFD, while being 50% lower compared to AASHTO Standard Specifications. A similar trend was observed for multiple lanes loading. Both AASHTO Standard Specifications and AASHTO LRFD GDFs equations were overly conservative for two and three lanes loaded. The Abaqus calculated GDFs were approximately 53% and 62% less for two and three lanes loaded respectively, compared to AASHTO LRFD.

A limited parametric study was conducted to study the effect of changing crucial bridge design parameters on GDFs ratios. The Scotch Road Bridge was modelled with ten piles spaced at 3.35m, opposed to the original nineteen piles spaced at 1.675m. All loading cases yielded similar results to the full model analysis with one truck in lane one being the most accurate and 2 and 3 trucks being overly conservative. This indicates that changing the number of piles didn't have any significant effect on the GDFs calculations.

Additionally, the effect of changing girder spacing on the GDF ratios was studied. The three bridges used were seven, ten, and thirteen girder bridges with 3.5 m, 3.35 m, and 2.51 m spacing respectively. All considered bridges were within the AASHTO LRFD maximum and minimum girder spacing requirements.

Fig. 7 shows a comparison between the FEM GDFs values, AASHTO LRFD, and AASHTO Standard Specifications for one lane loaded, and varying girder spacing. All GDFs values decreased as girder spacing decreased. The AASHTO LRFD GDFs closely corresponded to the FEM calculated GDFs for 3.5 m spacing. However, the 1996 AASHTO distributed approximately 40% more load than the model predicted per girder for this girder spacing. In general, The 1996 AASHTO standard specifications were overly conservative for all cases, especially the 3.5 m spacing.

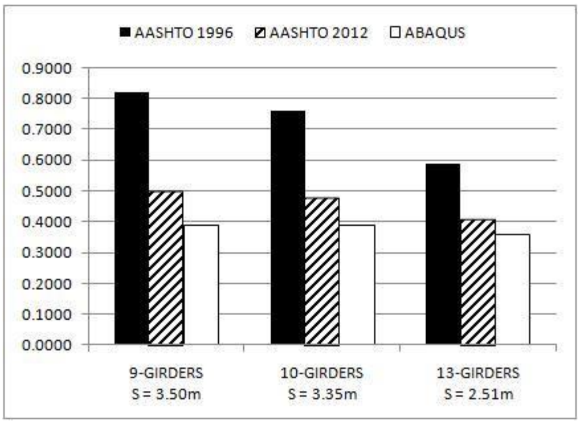

Fig. 7: Comparison between FEM, AASHTO LRFD, and AASHTO Standard GDFs values for one-lane loaded with varying girder spacing

AASHTO standard and AASHTO LRFD were again compared to the FEM GDFs results for two and three lanes loaded with varying girder spacing, Fig. 8. The resulting comparison yielded both AASHTO codes results were conservative for all cases, larger spacing in particular. AASHTO Standard was approximately 70% higher for 3.5 m spacing and AASHTO LRFD was 48% higher than the FEM for two lanes loaded.

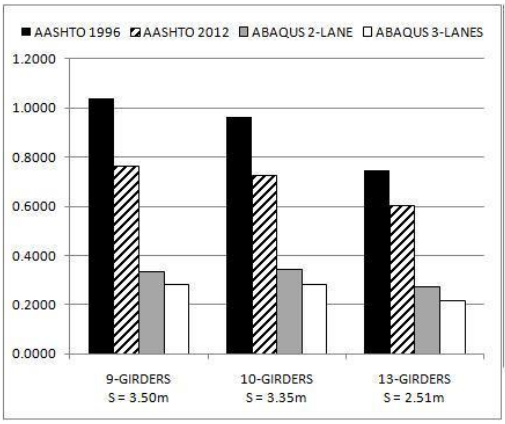

Fig. 8: Comparison between FEM, AASHTO LRFD, and AASHTO Standard GDF values for two and three-lanes loaded with varying girder spacing

The effect of the boundary conditions on GDFs ratios was also investigated. The Scotch Road Bridge FEM was adjusted to function as a simply supported bridge. Figs. 9 and 10 represent a comparison of the AASHTO GDFs ratios with a simply supported Scotch Road Bridge and the original IAB.For all loading cases both AASHTO 2012 and 1996 GDF ratios (Abaqus GDF/2012 AASHTO-LRFD GDFs or Abaqus GDF/1996 AASHTO Standard Specifications GDFs) are more conservative for simply supported bridge than IAB for one lane loading.

Additionally, both two and three lanes loaded are even more conservative for both bridge types compared to one lane loading.

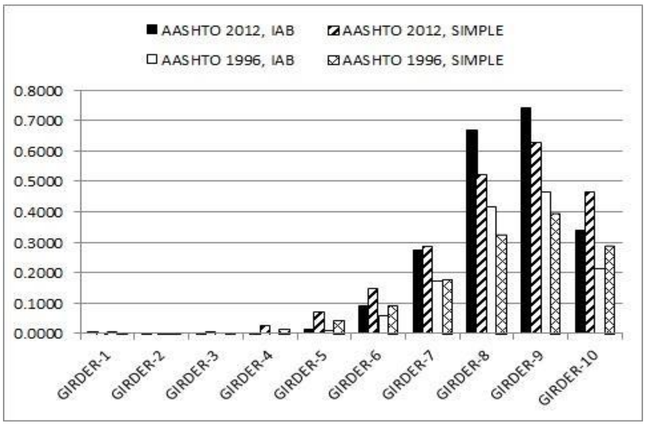

Fig. 9: Comparison of GDFs ratios for simply supported Scotch Bridge to the actual Scotch Road Bridge loaded for one-lane loading

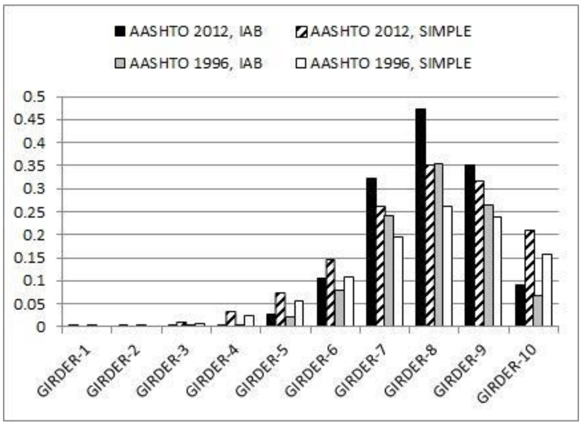

Fig.10: Comparison of GDFs ratios for simply supported Scotch Bridge to the actual Scotch Road Bridge loaded in two lanes

IX. SUMMARY AND CONCLUSIONS

The integral abutment Scotch Road Bridge was modelled in the finite element software Abaqus/Cae. The model was verified using temperature-displacement data recorded by Hassiotis et al., 2006 from April, 2003 to May, 2006. After validation of the FE model, three loading cases, including one truck in lane one, two trucks in lanes one and two, and three trucks in lanes one, two, and the auxiliary, were run in Abaqus/Cae. The stress data was used from each case to calculate the GDF for each girder. Moreover, a parametric study was conducted to study the sensitivity of the model to crucial design parameters such as girder spacing, number of piles, and the effect of boundary conditions at the end of the bridge (simply supported versus integral abutment). The following conclusions were found in this study:

- The 1996 AASHTO Standard Specifications GDF values are significantly more conservative than current 2012 AASHTO LRFD Bridge Design Specifications.
- GDFs for the 2012 AASHTO LRFD Bridge Design Specifications are reasonable for one lane loaded (approximately 20% less compared to AASHTO LRFD GDFs and 50% compared to 1996 AASHTO Standard Specifications), while overly conservative for multiple lanes loaded.
- Reducing the number of piles in a bridge does not significantly change the girder distribution factor.
- Both AASHTO design manuals GDFs values decreased in conjunction with a decrease in girder spacing. Additionally, both codes are more conservative in predicting GDFs for multiple lanes versus one lane loading.
- The 1996 AASHTO Standard Specifications GDFs were26% more than the FEMfor the Scotch Road Bridge (3.35 m girder spacing) in case of one lane loading, while it was only 9% more for AASHTO LRFD Specifications.
- Both AASHTO 1996 and 2012 are more conservative in predicting simply supported bridge GDFs than an equivalent IAB.

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