강거더 연속교의 횡방향 활하중분배계수 검증

Girder Distribution Factors for Continuous Steel Girder Bridges

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Abstract

Current bridge design codes do not clearly specify the girder distribution factors for continuous bridges. The objective of the paper is to validate the use of code-specified girder distribution factors for the continuous steel girder bridges, and to provide a basis for recommended girder distribution factors (GDF) for interior girders, suitable for evaluation of existing continuous steel girder bridges. This paper presents the procedure and results of 3-dimensional finite element analysis that were performed on five of continuous steel girder bridges to verify girder distribution factors. The analysis results showed that the live load moment distribution at the negative moment region is very similar to those at the positive moment region in continuous steel girder bridges. It was also found that the GDF's based on the strain values are similar to those based on the deflection. GDF's based on the deflection show marginally better distribution. The analysis results confirmed that the code specified GDF's for continuous steel girder birdges are very conservative.

요 지

현재의 교량설계기준에서는 연속거더교량에 대한 활하중 분배계수 규정에 대해 명확한 언급이 되어있지 않다. 따라서 이 논문의 목적은 단순교에 사용되는 시방서 규정 활하중 분배계수가 연속거더교에도 적용이 가능한지를 유한요소해석에 의해 검증하는 것이다. 연속거더교에 대한 활하중 분배의 영향을 검증하기 위해 5개의 공용중인 연속교에 대한 유한요소해석을 실행하였으며 그 결과 연속교에서 정모멘트 구역과 부모멘트 구역에서 활하중의 분배 형태는 매우 비슷하다는 결론을 내릴 수 있었다. 또한 처짐값의 비교에서 얻어진 활하중의 분배와 변형율에 기본한 활하중의 분배 양상의 차이가 거의 없는 것으로 나타났다. 해석 결과 현재 사용되고 있는 설계기준의 활하중 분배에 대한 규정은 연속거더교에 대해 매우 보수적이라고 판단된다.

Keywords : Live-Load Distribution, Continuous Girder Bridges, Finite Element Analysis

핵심 용어 : 활하중 분배, 연속거더교, 유한요소해석

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1. Introduction

A considerable number of bridges show signs of deterioration. In particular, there is a severe corrosion on many steel bridges. Therefore, there is a need for accurate and inexpensive methods for diagnostics, particularly, verification of load distribution.

Load test is an increasingly important topic in the effort to deal with the deteriorating infrastructure. However, the load test is not always feasible due to high traffic volume, inaccessibility. In this situation, finite element analysis can be a valuable alternative to accurately verify the actual load carrying capacity of existing bridges.

The objective of this study is to validate the code specified girder distribution factors (GDF) for in-service continuous steel girder bridges by 3-D finite element analysis.

Nowak et al. (1999) carried out load tests on five simply supported bridges to verify the code specified GDF′s. An extensive experimental and analytical study by Eom and Nowak (2001) confirmed that for the simply supported girder bridges, the GDF′s specified by the AASHTO Standards (2002) and AASHTO LRFD (2004) are often too conservative. However, the GDF′s for the continuous girder bridges were not considered in the previous study. Knowledge of the accurate GDF′s is needed to determine the actual value of live load (design truck load) for bridge girders. Continuous bridges can have different girder distribution characteristics.

However, current bridge design codes such as AASHTO Standards (2002) and AASHTO LRFD (2004) do not clearly specify the girder distribution factors for continuous bridges.

Korean Standard Highway Bridge Codes (1996, 2005) use the same GDF formulas as AASHTO formulas. The GDF formula specified in the AASHTO Standards (2002) was originally proposed by Newmark (1948) based on lab tests, and later developed by Sanders and Elleby (1970) by modeling bridges as orthotropic plates, and in these studies, continuous support conditions were not considered.

Therefore, this study aims to verify the girder distribution factors for continuous steel girder bridges. To accomplish this, finite element analysis was carried out on five of in-service highway bridges. The girder distribution factors are calculated both at positive moment region and negative moment region. The results are compared to the specified values in AASHTO Standards (2002) and AASHTO LRFD (2004).

2. Girder Distribution Factors for Continuous Bridges Specified in the Codes

For the bending moment in interior girders, the AASHTO Standards (2002) and the Korean Highway Bridge Specifications (2005) specify GDF′s as follows. For steel girder bridges and prestressed concrete girder bridges, with multiple lanes loaded,

$$GDF = \frac{S}{1.65} \qquad (1)$$

where $S=$ girder spacing (m).

In AASHTO Standards (2002) and the Korean Standard Highway Bridge Specifications (2005), there is no distinction between the girder distribution factors for positive moment region and negative moment region.

The girder distribution factors specified in the AASHTO LRFD code (2004) and the Standard Specifications of Korean Highway Bridges, Supplements (1996) specify the GDF as a function of girder spacing, span length, stiffness parameters, and bridge skewness. For the bending moment in interior girders with multiple lane loaded, GDF is:

$$GDF = \left(0.075 + \left(\frac{S}{2900} \right)^{0.6} \left(\frac{S}{L} \right)^{0.2} \left(\frac{K_g}{Lt_s^3} \right)^{0.1} \right) \quad (2)$$

for $\theta < 30°$

where S=girder spacing (mm); L=span length (mm); $K_g = n(I + Ae_g^2)$, t_s=thickness of concrete slab (mm); n=modular ratio for the girder and slab materials; I=moment of inertia of the girder (mm^4); A=cross section area of the girder (mm^2); e_g=distance between the centers of gravity of the girder and the slab (mm); and θ=skew angle in degrees.

The AASHTO LRFD (2004) formulas were developed based on a NCHRP Project 12-26 by Zokaie (1991). However, the correction factors for continuity have been deleted in AASHTO LRFD formulas.

In the AASSHTO LRFD (2004) GDF formulas for continuous bridges, for the positive moment, the span length, L, is the actual span length, and for the negative moment, the span length is the average of two adjacent spans.

3. Bridge Selections

This study is focused on existing continuous steel girder bridges. These structures constitute about 11% of all steel girder bridges in Michigan, the United States. In the bridge selection process, the consideration was focused on that the results of the analysis can reflect the majority of steel continuous girder bridges. Therefore, the selection of bridges was based on the following criteria:

· Structural type and material; steel continuous girder bridges.
· Skewness; bridges with skew angle of more than 30 degrees were excluded.
· Year of Construction; bridges built more than 20 years were considered to represent the majority of continuous steel bridge population
· Number of Lanes; two-lane bridges were selected.
· Traffic volume and accessibility; field testing can be a very effective means to verify the results of finite element analysis. Therefore, the field testing was planned after the analysis is finished. By this reason, the bridges that can be tested afterward were considered. Very busy bridges were not considered because of the expected difficulties with traffic control. Also, the bridges with easy accessibility for instrumentation were selected for the analysis.

Finally, five bridges were selected for the analytical study. The detailed descriptions of the selected bridges are shown in Table 1.

4. Procedures used in Finite Element Analysis

For the purpose of finite element analysis, the geometry of the bridge superstructure can be idealized in many different ways.

Table 1 Selected Bridges

Bridge	No. of Spans	No. of Lanes	Total Bridge Length (m)	Individual Span Length (m)	No. of Girders	Girder Spacing(m)	Year Constructed	Skew Angle
Bridge A	3	2	104.2	36.0+30.2+38.1	7	2.18	1982	1
Bridge B	3	2	113.2	42.7+34.4+36.0	7	2.11	1983	17
Bridge C	2	2	65.8	32.9+32.9	4	2.59	1967	0
Bridge D	3	2	106.5	38.9+28.8+38.9	6	2.54	1980	19
Bridge E	4	2	138.9	36.9+32.5+32.5+36.9	5	2.69	1979	7
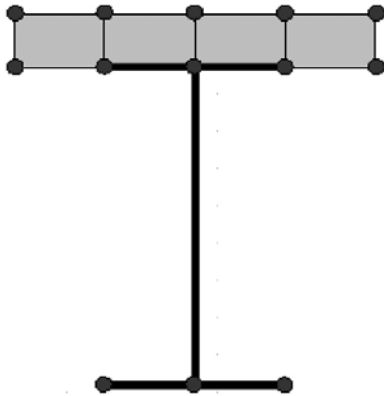
Fig. 1 Typical FEM Modeling Scheme

Table 2 Total No. of Elements and Nodes used in the FE Models

Bridge	Elements	Nodes
Bridge A	18,641	24,629
Bridge B	24,462	32,430
Bridge C	12,465	17,193
Bridge D	18,850	24,510
Bridge E	23,072	30,969

For this study, a three-dimensional finite element method was applied to investigate the structural behavior of the considered bridges. The concrete slab is modeled using isotropic, eight node solid elements, with three degrees of freedom at each node. The girder flanges and web are modeled using three-dimensional, quadrilateral, four node shell elements with six degrees of freedom at each node, as used in the study of Tarhini and Frederic (1992). The typical modeling scheme is shown in Fig. 1.

The structural effects of the secondary members such as sidewalk and parapet, are also taken into account in the FE analysis models. An example of a FEM mesh for a bridge with five girders spaced at 2.69m is shown in Fig. 2.

The analysis was performed using the commercially available ABAQUS finite element program. Material and other structural parameters are based on the AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges (1994), as well as the collected information about the bridge supplemented with engineering judgment. The generated models for all bridges used very fine meshes. The number of elements and nodes used for each model is shown in Table 2.

5. Loadings in the Fe Models

In Michigan, the United States, the maximum mid-span moment in medium span bridges is caused by 11-axle trucks, with gross vehicle weight (GVW) up to 730 kN depending the axle configuration. The vehicles used
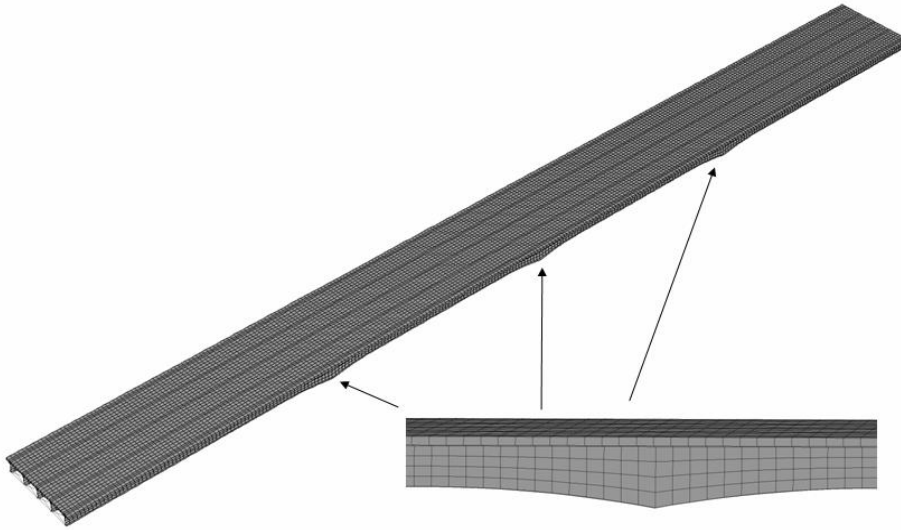
Fig. 2 An Example of FEM Mesh used in the Analysis

Fig. 3 Typical Side-View of Trucks used in the Analysis

in the analysis were fully loaded, three-unit, 11-axle trucks. A typical side-view of a truck used in the tests is shown in Fig. 3. Analysis was performed under side-by-side truck loading condition.

6. Girder Distribution Factor Calculation from Analysis Results

Girder Distribution Factors (GDF) are calculated from the maximum static strains obtained from the static loading at each girder at the selected cross section along the length of the bridge. The maximum negative bending moment occurs right over the piers. However, the strain concentration right over the supports prevents from accurate strain calculation. Therefore, strain values were recorded at 30cm apart from the center of the supports.

Ghosn et al. (1986) assumed that GDF for a girder was equal to the ratio of the static strain at that girder to the sum of static strains for all the girders.

Stallings and Yoo (1993) used the weighted strains to account for different section moduli of the girders. Accordingly, GDF for the i-th girder, GDF_i, can be derived as follows:

$$GDF_i = \frac{M_i}{\sum_{j=1}^{k} M_j} = \frac{ES_i \varepsilon_i}{\sum_{j=1}^{k} ES_j \varepsilon_j} = \frac{\dfrac{S_i}{S_\lambda} \varepsilon_i}{\sum_{j=1}^{k} \dfrac{S_j}{S_\lambda} \varepsilon_j} = \frac{\varepsilon_i w_i}{\sum_{j=1}^{k} \varepsilon_j w_j} \quad (3)$$

where M_i=bending moment at the i-th girder; E=modulus of elasticity; S_i=section modulus of the i-th girder; S_ℓ=typical interior section modulus; ε_i=maximum bottom-flange static strain at the I-th girder; w_i=ratio of the section modulus of the ith girder to that of a typical interior girder; and k=number of girders.

7. Results of Fe Analysis

For each bridge, the collected strain and deflection data obtained from the 3-D finite element analysis served as a basis for the calculation of the girder distribution factors. Analysis was performed for five steel continuous girder bridges that represent the majority of Michigan steel continuous bridge populations.

The girder distribution factors for the positive moment region, calculated from the strains and the deflections under full lane loading are shown in the Figs. 4 to 8. They are compared with code-specified values. As can be seen in the Figs. 4 to 8, for typical two lane steel girder bridges, the code-specified GDF's are conservative. Particularly, GDF's specified in the AASHTO Standards (2002) is very conservative. In some cases, the GDF from the analysis is only about 60 percent of the value specified in AASHTO Standards (2004). In the Table 3, the ratios for each bridge are shown. For tested bridges, the ratios range from 0.57 to 0.71 for positive GDF values, and from 0.55 to 0.74 for negative GDF values. It indicates, for tested bridges, that there are at least 30 percent of safety margin from the live load calculation by using AASHTO Standards (2002).

The GDF's specified in the AASHTO LRFD (2004) are less conservative than those of AASHTO Standards (2002). Still, AASHTO LRFD (2004) GDF's have, without exception, plenty of safety margin, as shown in the Table 3.

Figs. 4 to 8 indicate, in all bridges, that GDF's obtained from deflection data are a little less than those obtained from strain data. However, the differences are minimal. In all cases, the differences are less than 5 percent, as shown in the Table 4.
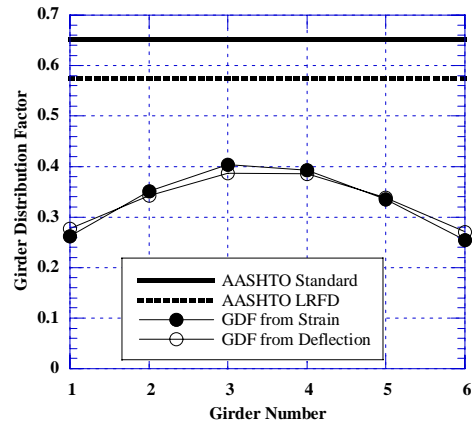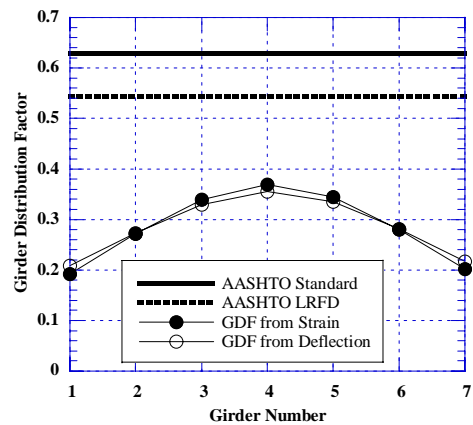

Fig. 4 Positve GDF at Midspan (Bridge A)

Fig. 5 Positve GDF at Midspan (Bridge B)
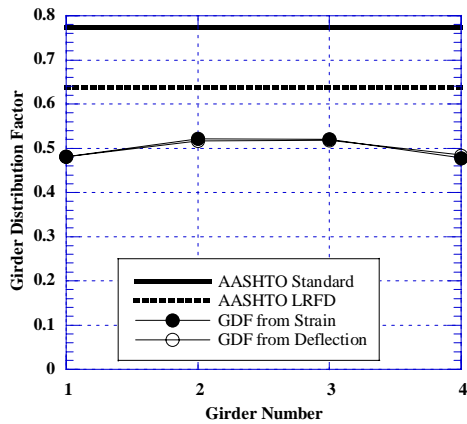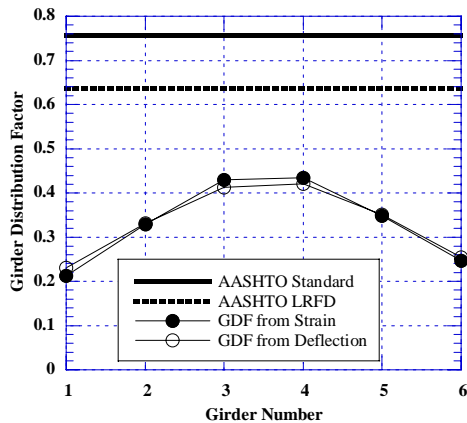
Fig. 6 Positve GDF at Midspan (Bridge C)

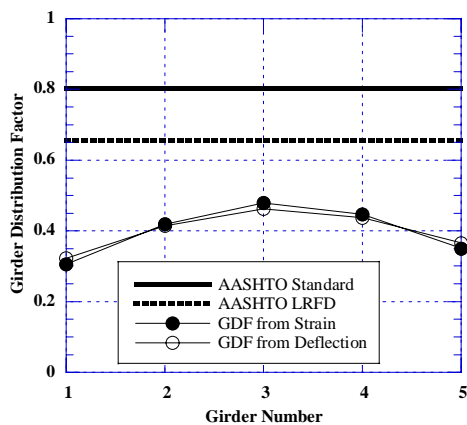Fig. 7 Positve GDF at Midspan (Bridge D)

Fig. 8 Positve GDF at Midspan (Bridge E)

Table 3 Ratio of Calculated GDF Vs. Code GDF

Bridges	Comparison of Calculated GDF's and Code Specified GDF's			
	Positue GDF / Code Values		Negative GDF / Code Values	
	AASHTO Standards	AASHTO LRFD	AASHTO Standards	AASHTO LRFD
Bridge A	0.62	0.70	0.57	0.66
Bridge A	0.59	0.68	0.57	0.67
Bridge A	0.71	0.82	0.74	0.89
Bridge A	0.57	0.68	0.55	0.67
Bridge A	0.60	0.73	0.67	0.82
Average	0.62	0.72	0.62	0.74

Table 4 Ratio of GDF based on Strain Vs. Deflection, and Positive Strains Vs. Negative Strains

Bridges	Comparison of Calculated GDF's (Ratios)	
	GDF from Positive Strain / GDF from Deflection	GDF from Positive Strain / GDF from Negative Strain
Bridge A	1.05	1.08
Bridge A	1.04	1.04
Bridge A	1.01	0.96
Bridge A	1.03	1.05
Bridge A	1.04	0.92
Average	1.03	1.01

In the Figs. 9 to 13, the GDF's obtained from strains at both positive moment regions and negative regions are presented and compared with code specified values. The results indicate that the girder distribution at the negative moment region can be different from those at positive moment region. For 3 bridges(Figs. 9, 10 and 12), there is a better load distribution at the midspan, but others have better distribution at negative moment region. However, the differences are very small.

For all bridges, the differences in the GDF values are documented in Table 4. In all cases, it is confirmed that the GDF's specified in AASHTO Standards (2002) are conservative,
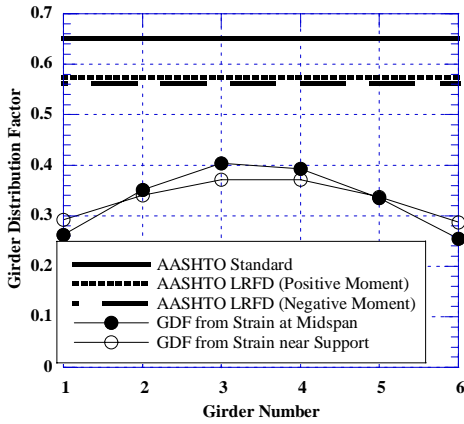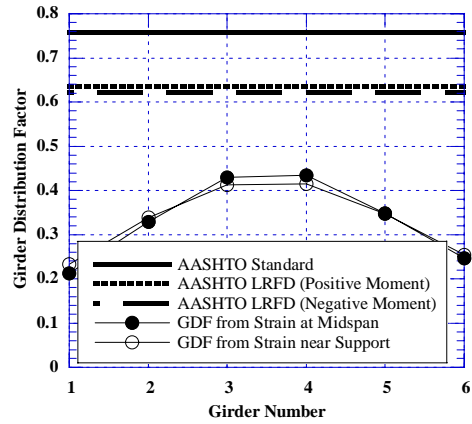
Fig. 9 Positve GDF vs. Negative GDF (Bridge A)
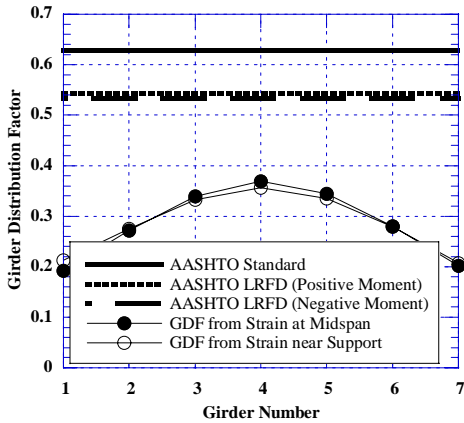

Fig. 10 Positve GDF vs. Negative GDF (Bridge B)
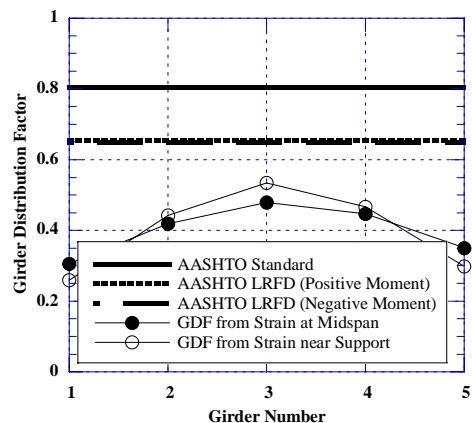

Fig. 11 Positve GDF vs. Negative GDF (Bridge C)

Fig. 12 Positve GDF vs. Negative GDF (Bridge D)

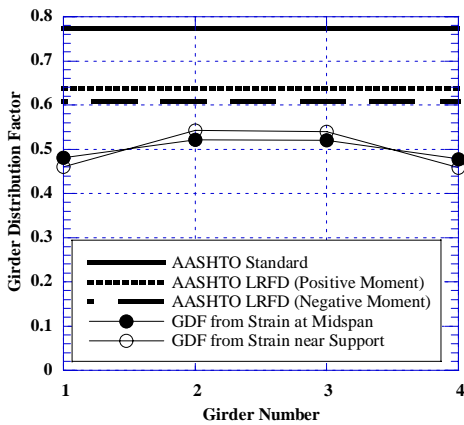

Fig. 13 Positve GDF vs. Negative GDF (Bridge E)

and in some cases, very conservative for moment distribution at both positive and negative moment region. The GDF's in AASHTO LRFD (2004) are also conservative.

8. Conclusions

3-D finite element analysis was performed for five continuous steel girder bridges with two traffic lanes, for the verification of the code specified girder distribution factors.

The analysis confirmed that the girder dis-

tribution factors for continuous steel girder bridges are, in all cases, significantly smaller than those specified by AASHTO Standards (2002). Also AASHTO LRFD (2004) GDF's are conservative for all selected continuous girder bridges, both for positive moment and negative moment regions.

Therefore, it can be concluded that the code specified GDF's, for multi-lane traffic, are very conservative, for both AASHTO LRFD (2004) and AASHTO Standards (2002). AASHTO Standards (2002) provides more conservative GDF's.

This conservatism assures that the continuous bridges designed using the code-specified GDF formulas have some level of extra safety reserve. It other words, bridges sometimes can safely resist the rare occasions of the unexpectedly high load passages. In some cases, this extra safety reserve in the load carrying capacity can be utilized to prove that a deficient bridge is actually adequate to carry normal traffic, based on very careful calculation and field monitoring.

Since the girder distributions are very similar for both positive moment region and negative moment region, it is acceptable to use same girder distribution factors for both positive moment region and negative moment region.

9. Acknowledgement

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