Behavior of Horizontally Curved Multi-Spans Continuous Composite Bridges under AASHTO LRFD Loading

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Abstract

The purpose of this paper is to study the effects of bridge curvature and spans continuity on the lateral distribution of the flexural and warping longitudinal stresses among girders in a horizontally curved continuous composite bridge under AASTHO LRFD loading. To achieve this goal, numerical analysis is conducted by using finite element program CSI Bridge. The bridge prototype adopted comprises a continuous three equal spans bridge with 120m total length. Bridges with different AASHTO live load cases and curvature ratios (L/R) are investigated. The study has revealed that the bridge curvature is the most influential factor that affect lateral stress distribution among girders. Generally, as curvature ratio increases, warping stresses increased, and thus stress distribution among girders became vastly non-uniform and extremely deviate from straight bridge behavior. Moreover, as (L/R) ratio increased the stress share for the outermost girders increased and for the innermost girders decreased and for high curvature ratio moment and stress reversal occur at mid span for the interior girders. The study shows that girders maximum stress values occurred for the case of a bridge loaded with AASHTO lane load and truck load on the exterior lane only.

Keywords: AASHTO LRFD Load, Composite Deck, Curved Bridge, Finite Element, Girders Distribution Factors

1. Introduction

In the past, the alignment of horizontally curved bridges was provided by straight girders or chords that met the bridge curvature. Most recently, the trend is toward the construction of such curved bridges using actually curved steel girders. The benefits of actual curved girders give allowance for increasing span length, fewer piers, more aesthetically pleasing structures, simplicity of construction and thinner sections can be designed [4]. However, Curved bridges have created new design problems for the engineers. Because of the curved profile of the girders, the eccentricity of the mid span with respect to the supports, the girders under load will twist in addition to distorting vertically and additional stresses result from twisting action. Generally, the effects of twisting on curved girder are large and need to be addressed by specifications and regulations related to the design of curved bridges.

The bridge designer can either select a series of simple spans or it can be designed as a bridge that is continuous over the piers. The advantages of continuous spans if compared with simple spans are [4]:

1. Lower cost because of material reduction in the superstructure, or longer spans for the same material and thus fewer piers.
2. Deflection and vibration lesser than simple span.
3. Longitudinal forces on the superstructure can be transmitted to the abutments instead of partially carried by the piers.
4. Needing of expansion devices have been less.
5. A more pleasing appearance can be achieved because of possible variation in span length and depth of girders.

If foundation conditions are good, and other site characteristics indicate medium or long spans, the continuous structure shows the least cost. For short spans, there is little cost difference and the speed and simplicity of erection may favor the simpler spans. Where precast, pre-stress concrete beams are used, the simple span is most often favored. Cast-in-place concrete beams can be easily formed as continuous beams and the saving in weight and more pleasing appearance are definite advantages.

2. Geometric Modeling

To study lateral stress distribution among bridge girders a three-equal spans continuous composite bridge with 120 meters total span length was analyzed. The adopted bridge model consists of a two lane concrete deck carriageway supported by 4 steel I-girders. X-type cross frames with top and bottom cords were used to laterally connect girders. **Fig. 1** shows the details of the typical composite concrete steel I-girder bridge cross-section used in this study.

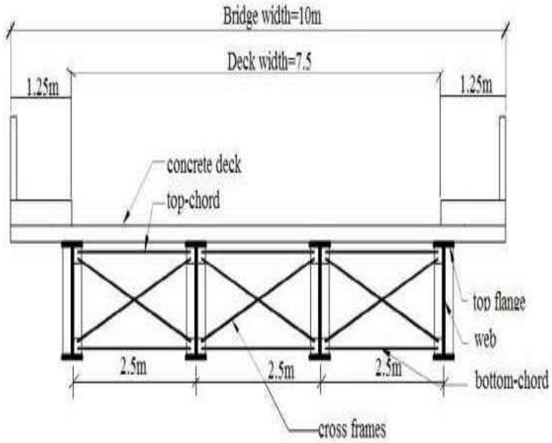


Fig.1: Typical cross-section of the composite concrete-steel bridge

A three-dimensional finite element model is used to analyze composite bridge models adopted in this study and to determine their structural behavior. The available commercial finite element program CSI Bridge is used throughout this study. The composite bridge is divided into concrete deck slab, top steel flange, steel web, bottom steel flange, and the cross-bracing.

Based on previous work [7] on finite element modeling, eight node shell elements with six degrees of freedom at each node are used to model the concrete deck slab and girders web, while the top and bottom girder flanges are modeled using frame elements. **Fig. 2** shows the three-dimensional finite element model for the four curved composite girders. **Fig. 3** shows typical central span details and girders designation used throughout this study, whereas plan view of the full numerical bridge model is presented in **Fig 4**.

Abutments are represented by dimensionless elements called "foundation elements" which attach from lower girder nodes to the earth. The piers are modeled by columns and cap beams [2]. Finally, individual truss elements connected to the nodes at the top and bottom of the girders are used to model the cross-bracings with the top and bottom chords.

The following assumptions are adopted in the investigated bridge models:

1. The reinforced concrete slab deck has composite action behaves as full interaction with steel members. Shoring is assumed to be used during construction.
2. Three continuous equal spans for this bridge.
3. Homogeneous linear elastic for all materials used.
4. Effect of curbs and road super elevation were neglected.
5. Constant radius of curvature between support lines for curved bridges assuming span to radius ratio (L/R) = 0.5, 0.7, 0.9, 1.2, 1.6, 2 and 2.4.

Other curved bridge properties include [7]:

- The thickness of deck slab is 230mm.
- The total width of the bridge is 10m.
- The girder spacing equal 2.5m and the length of the over-hanged slab is equal half to the girder spacing.
- The depth of girder webs is 0.8x(L/25) of the center line span.
- The thickness of girder web is 14mm.
- The width of top steel flange is 425mm and thickness is 25mm while the width of bottom steel flange is 525mm and thickness is 37.5mm

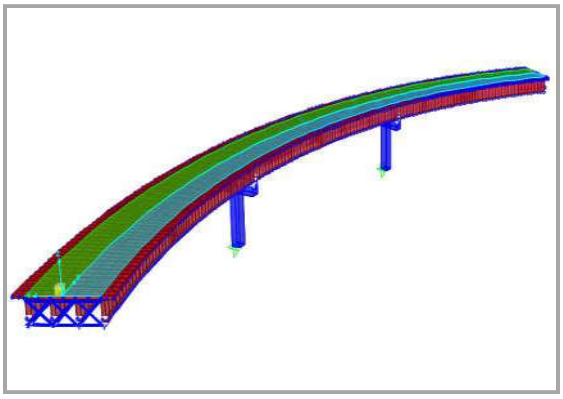


Fig. 2: 3D finite element model for the 4-girders curved bridge, cross frames and lanes width are shown.

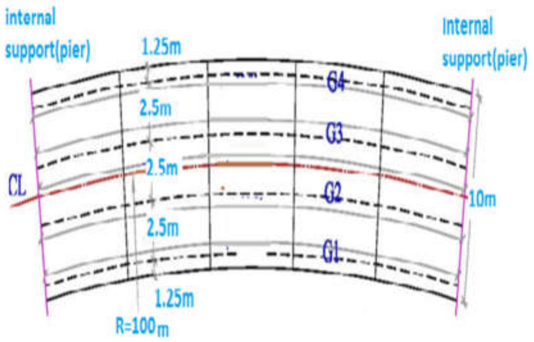


Fig. 3: Plan of the central span of typical curved bridge with 10m deck width.

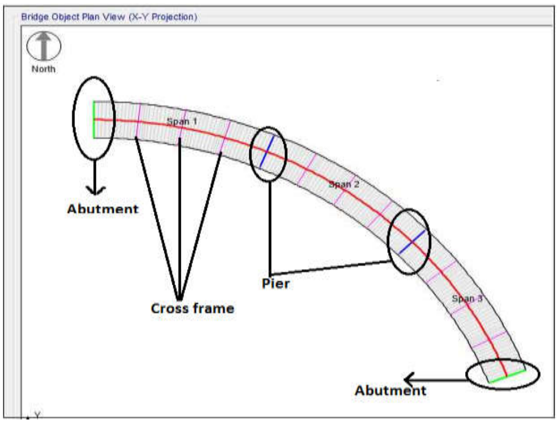


Fig. 4: CSI-bridge Plan view of the bridge object

3. AASHTO LRFD Loading

3.1. Loading

According to AASHTO LRFD [1], vehicular live loading on the roadway of bridges shown in **Fig. 5**, designated HL-93, shall consists of a combination of the following:

1. Design lane load combined with a design truck.
2. Design lane load combined with a design tandem.

The extreme force effect shall be taken as the largest of the above combinations. To find out which type of loading must be considered in the numerical analyses, the same bridge model is analyzed under the two type of AASHTO vehicular live loading. Results for the positive and negative girder distribution factors (GDF) for both types of loading are compared in **Figs. 6 and 7**, respectively. Comparison presented in these figures shows that almost the same effect for the type of vehicular live loading on girder distribution factors at both critical sections for maximum moments in the bridge deck. Accordingly, AASHTO truck loading will be adopted with lane load to simulate bridge live load according to AASHTO standard requirements in this study

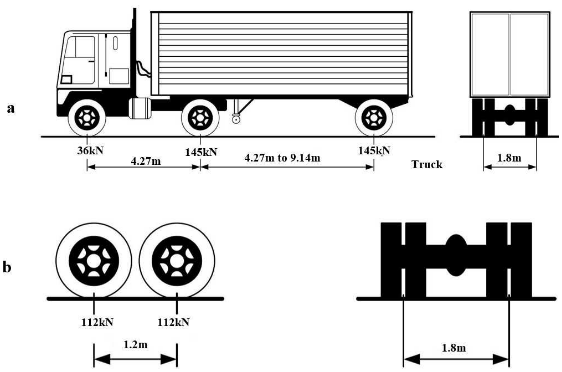


Fig.5: AASHTO LRFD vehicular loading, (a) truck, (b) tandem [5]

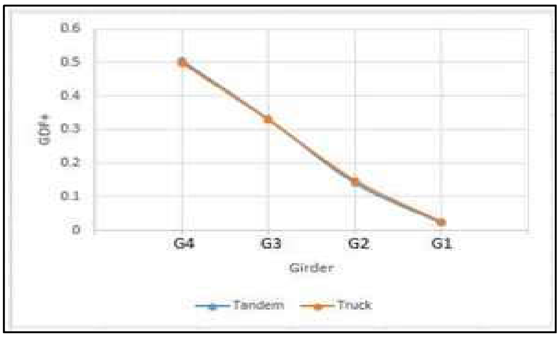


Fig.6: Positive GDF values due to different types of ASSHTO vehicular live load.

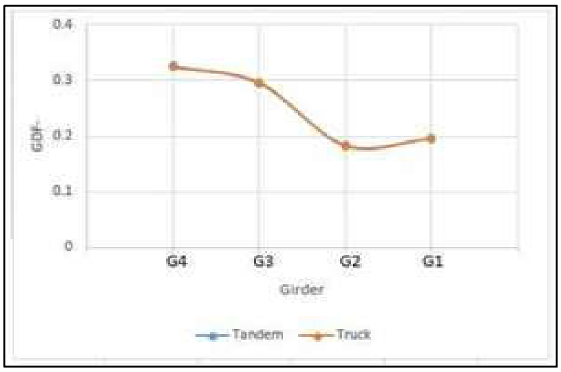


Fig.7: Negative GDF values due to different types of ASSHTO vehicular live load.

The design lane load shall consist of a load of 9.3kN/m uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over a 3.05m width. In considering the design lane load in the design of continuous spans, as many spans shall be loaded with 9.3kN/m uniform load simultaneously as is necessary to produce the maximum effect. The design lane load is placed longitudinally only on these portions of the spans to give maximum effect. The design truck load consists of three axles, the lead axles of 36 kN and the two following axles of 145 kN. The distance between each axle is 4.27m (14 ft) and the transverse spacing of the wheels is 1.8m (6 ft). For a maximum positive moment, the design truck is placed with its central axle at the mid-span, see **Fig. 8**.

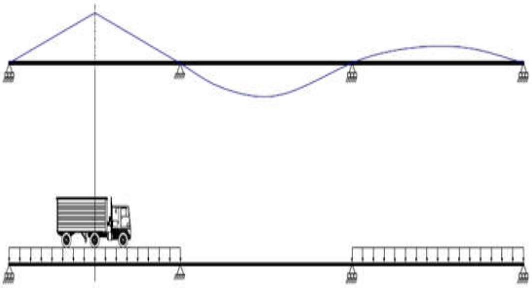


Fig. 8: Location of the lane load and truck for maximum positive moment in span [5].

On the other hand, for the determination of maximum negative moment in a continuous deck AASHTO [4] specifies that two design trucks may be located in each lane spaced a minimum of 15m (50 ft.) between the lead axle of one truck and the rear axle of the other truck. The two design trucks are placed in adjacent spans to produce the maximum effect as shown in **Fig. 9**. The total combined moment (lane + truck) is multiplied by a reduction factor of 90 percent to give design moment.

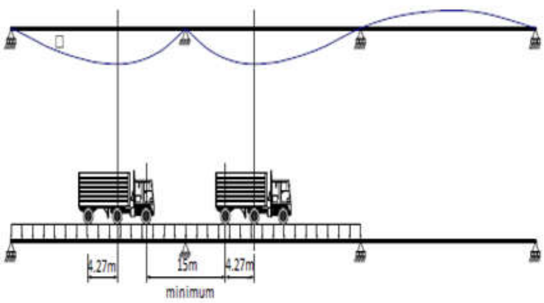


Fig. 9: location of the lane load and truck for maximum negative moment in span [5].

3.2. Load Factors and Load Combinations

The load Combination used is the strength I: Basic load combination relating to the normal vehicular use of the bridge without wind [1].

Strength I: 1.25DC+ 1.50DW + 1.75(LL+IM)

Where

DC = is the dead load of the structure and components present at construction. These have a lower load factor because they are known with more certainty.

DW = are future dead loads, such as future wearing surfaces. These have a higher load factor because they are known with less certainty.

LL = vehicular live load

IM = vehicular dynamic load allowance

I =1+IM/100; IM=33%

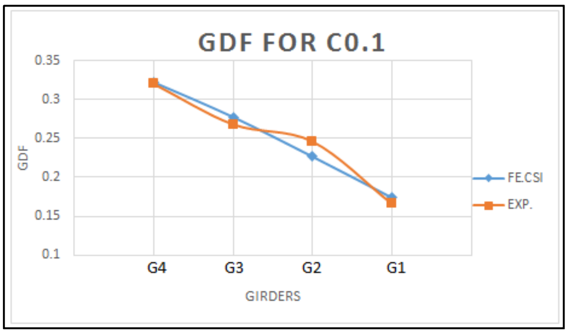
4. Girder Distribution Factor (GDF)

Girder distribution factors (GDF) are used throughout this study to explore stresses distribution among bridge girders and eventually analyze continuous bridges behavior due to AASHTO loading to assess the critical factors affecting their performance. The GDF, given by Equation (1), is the ratio of the maximum generated longitudinal bottom flange stress of the specified girder at the critical section to the summation of the maximum longitudinal stresses response for the whole girders at the same bridge section [3]. The Girder Distribution Factors are calculated for stresses in the positive moment region (GDF+), i.e. stresses at mid-span of the first span, and in the negative moment region (GDF-), i.e. stresses at the first interior support or pier.

$$GDF=\frac{\sigma i}{\sum_{i=0}^{n}\sigma i} \qquad \text{..................(1)}$$

Commonly, the bottom flange of girders at mid-span exhibit the highest tensile stresses, whereas the bottom flange of girders at the first internal support stand for the location where the highest negative moment compressive stresses are expected. Trends in the bottom flange which is the best measure available of the global response [6]. Hence, longitudinal stresses are measured at the tip of the bottom flange of each girder where flexural and warping stresses are maximum, i.e. flange tip with the greatest curvature ratio.

In order to validate the numerical model comparison with experimental work [8] is carried out and results for this comparison are presented in **Fig. 10**



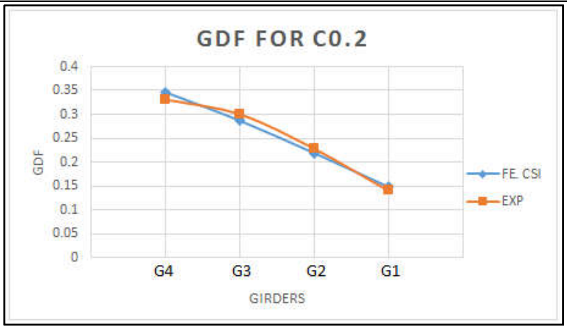


Fig. 10: Comparison with experimental work for different bridge curvature.

This figure shows that the general trend for GDF variation for the different bridge curvature and for both experimental and numerical models are in good correlation.

5. Effect of Bridge Curvature

To fully address the effect of bridge curvature on the lateral stresses distribution among girders, the following tables and figures present results for GDF values due to gravity loads and different cases of AASHTO loading on the bridge deck. Live load cases include; loading the whole deck width, i.e. 2 lane loading, loading the exterior lane only, and loading the interior lane only.

Figs. 11 to **15** show the variation of the positive and negative GDF values for the bridge girders versus the degree of curvature for different live load cases and curvature ratio values (L/R) = 0.5, 0.7, 0.9 1.2, 1.6, 2 and 2.4, where L is the length of the central curved three continuous spans equal 120m and R is the radius of curvature with values equal 240, 170, 132, 100, 75, 60 and 50m, respectively. The 50m radius of curvature satisfies the minimum permitted radius of curvature according to AASHTO LRFD limitation [4]. Girders spacing of 2.5 meters is employed for the different bridge models adopted in this study.

Results presented in these figures reveal that curvature of the bridge has significant impact on GDF values variation in the positive and negative moment regions. It can be observed that GDF value for the exterior girder (G4) increases and for the inner girder (G1) decreases with the increase in the span-to-radius of curvature ratio for all bridge models and live load cases. Whereas, for the intermediate girders, (G2) and (G3), minor effect for the curvature ratios on GDF values is observed. This result is due to fact that increase in the bridge curvature causes increase in the deck warping or bi-moment which result in the increase in the bending stresses carried by the bridge outermost girders and a decrease in the bending stresses carried by the bridge innermost girders.

The same trend for the variation for positive and negative GDF values is observed in the aforementioned figures indicating that the positive GDF values are more susceptible to curvature variation as compared to negative GDF values and that interior girder (G1) distribution factors are highly related to curvature variations as compared to exterior girder (G4).

Moreover, results presented in **Fig.11** and **Fig. 13** show that negative values for G1 distribution factors in the mid-span positive moment region occurred for bridges with high curvature ratios indicating that bottom flange bending stresses for the interior girder in the mid-span changed from tension to compression with the increase in the curvature. This behavior for the interior girder G1 bottom flange bending stresses is due to the combined effect of the live load location, i.e. loaded lane(s), and the degree of bridge curvature which result in live load share on G1 greater than its load share due to primary vertical loads, i.e. dead loads. This result indicates that for continuous bridges with high curvature the innermost girders are subjected to hogging moment.

This situation is confirmed by the results for GDF values presented in **Fig.15** which indicate that the interior girder G1 is under sagging moment when only the interior lane of the bridge is loaded with AASHTO live loading, in which case the live lane and truck loading effects counteract bridge curvature effects.

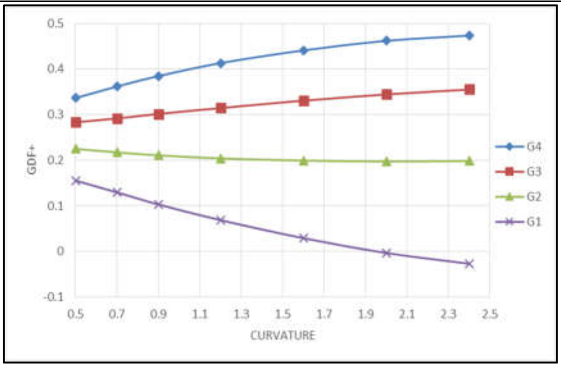


Fig.11: Effect of bridge curvature on (GDF+) values due to 2 lane loading.

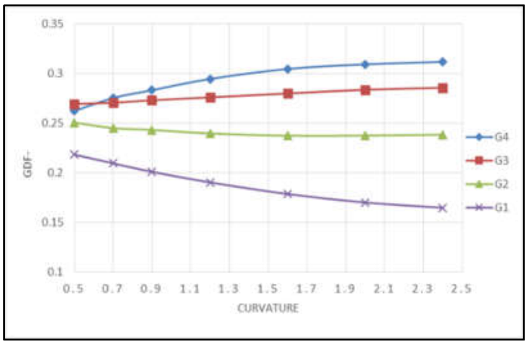


Fig.12: Effect of bridge curvature on (GDF-) values due to 2 lane loading.

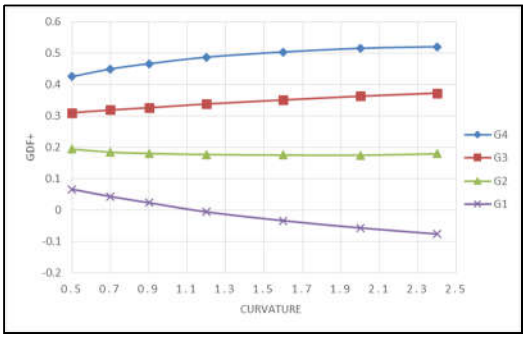


Fig.13: Effect of bridge curvature on (GDF+) values due to exterior lane load.

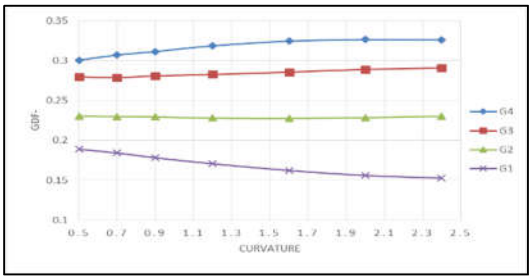


Fig.14: Effect of bridge curvature on (GDF-) values due to exterior lane load.

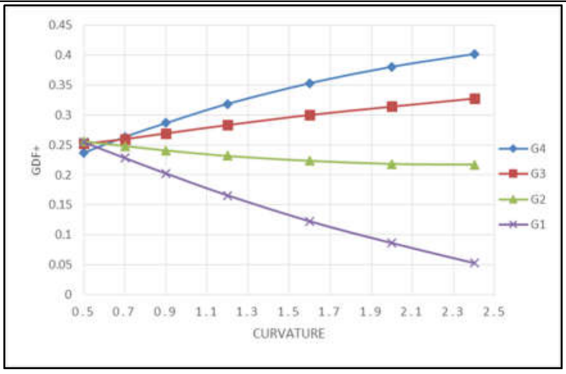


Fig.15: Effect of bridge curvature on (GDF+) values due to interior lane load.

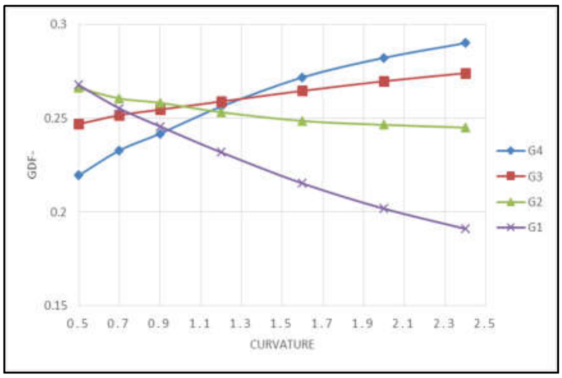


Fig.16: Effect of bridge curvature on (GDF-) values due to interior lane load.

Finally, **Tables. 1** and **2** below show summary for the percentage difference in the positive and negative GDF values for different girders due to increase in the bridge curvature ratio from the lowest value (L/R=0.5) to the maximum allowed (L/R=2.4). Results presented reveal that the intermediate girders are moderately affected by the bridge degree of curvature, while significant effects are observed for edge the girders. Hence, a maximum difference in GDF values of about +67%, +32% for G4 and -214%, -30% for G1 at mid-span and support sections, respectively, is observed.

The same comparison is also presented schematically in **Figs. 17** to **19**.

Table 1: Difference in GDF values for girders at mid-span for different load cases and curvature ratios

load case	girders	GDF for different L/R		diff%
		L/R=0.5	L/R= 2.4	
2lane load	G4	0.34	0.47	38.2
	G3	0.28	0.35	26.8
	G2	0.22	0.20	-9.8
	G1	0.16	-0.03	-118.8
(G4/G1)%		212.50	-1566.67	
1lane EXT. Load	G4	0.43	0.52	20.9
	G3	0.31	0.37	20.0
	G2	0.19	0.18	-7.6
	G1	0.07	-0.08	-214.3
(G4/G1)%		614.29	650.00	
1lane INT. Load	G4	0.24	0.40	66.7
	G3	0.25	0.33	32.0
	G2	0.26	0.22	-15.4
	G1	0.25	0.05	-80.0
(G4/G1)%		96.00	800.00	

Table 2: Difference in GDF values for girders at internal support for different load cases and curvature ratios

load case	girders	GDF for different L/R		diff%
		L/R=0.5	L/R= 2.4	
2lane load	G4	0.26	0.31	19.2
	G3	0.27	0.29	5.7
	G2	0.26	0.24	-8.4
	G1	0.22	0.16	-27.3
(G4/G1)%		118.18	193.75	
1lane EXT. Load	G4	0.30	0.33	10.0
	G3	0.28	0.29	3.9
	G2	0.23	0.23	-0.1
	G1	0.19	0.15	-21.1
(G4/G1)%		157.89	220.00	
1lane INT. Load	G4	0.22	0.29	31.8
	G3	0.25	0.27	11.0
	G2	0.27	0.24	-7.9
	G1	0.27	0.19	-29.6
(G4/G1)%		81.48	152.63	

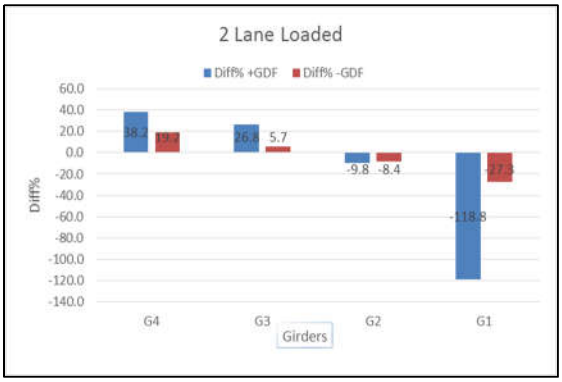


Fig.17: Percentage difference in GDF values due to curvature change (L/R=0.5) to (L/R=2.4) for 2 lane loading

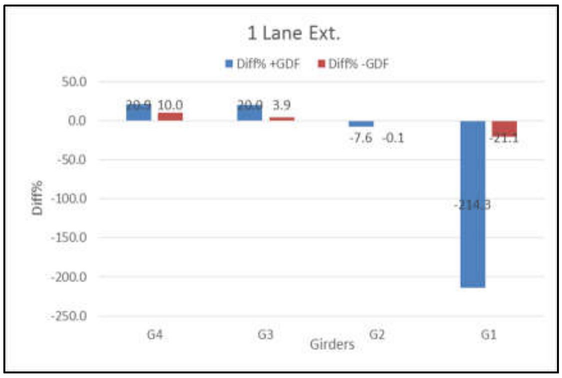


Fig.18: Percentage difference in GDF values due to curvature change (L/R=0.5) to (L/R=2.4) for exterior lane loading.

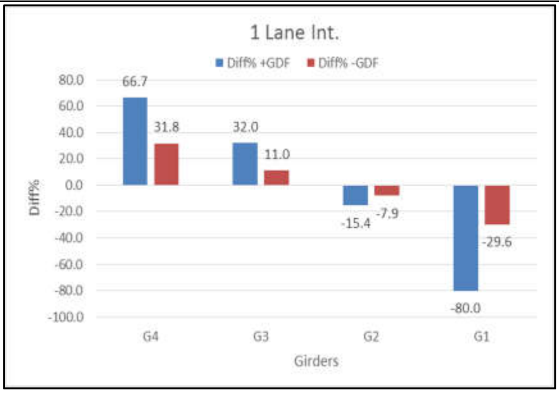


Fig.19: Percentage difference in GDF values due to curvature change (L/R=0.5) to (L/R=2.4) for interior lane loading

6. Conclusions

In this paper, an attempt is carried out to investigate the effects of bridge curvature and spans continuity on the stresses distribution among girders, and thus conclusions are drawn down below.

1. Bridge curvature is the most critical factor which plays an important role in the lateral distribution of stresses among girders.

2. Girder Distribution Factor (GDF) for the outermost girders increases with the increase in the span-to-radius of curvature (L/R) ratio, whereas GDF values for the innermost girders decreases with increasing (L/R) ratio.

3. Results for mid-span girder distribution factors showed that for continuous bridge loaded with AASHO live on the exterior lane only and when bridge curvature ratio (L/R) is greater than 1.2 the interior

girder was subjected to hogging moment. This behavior necessitate to limit continuous curved bridge curvature ratio not exceed certain limit to prevent hogging of bridge girders due to live loads.

4. The study revealed that continuous curved bridges loaded on the interior lane only and with small (L/R) ratio not exceeding 0.50 will perform like straight bridges. This behavior indicated that warping or bi-moment due to curvature for such cases is almost balanced by the effect of live load resultant eccentricity.

5. The results showed that the longitudinal stress share for the outermost girders in the mid-span region to be larger than those for the same girders in the support region. This behavior is in contrast with that for the innermost girders, in which case the longitudinal stresses share at the support region was found larger than for the same girders in the mid-span region.

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سلوك الجسور المركبة ذات الفضاءات المتعددة المستمرة والمنحنية افقيا تحت تاثير احمال المواصفة الامريكية AASHTO LRFD

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وزارة الاسكان والاعمار

الخلاصة

ان الهدف من هذا البحث هو دراسة تاثير درجة التقوس للجسر واستمرارية الفضاءات على التوزيع العرضي للاجهادات الطولية بسبب الانثناء واللي بين الروافد الفولاذية للجسور المركبة المستمرة والمنحنية افقيا. لتحقيق هذا الهدف, تم اجراء تحليل عددي باستخدام طريقة العناصر المحددة بواسطة برنامج CSI Bridge. نماذج الجسور التي تم اعتمادها تمثل جسر مستمر من ثلاث فضاءات متساوية بطول كلي يبلغ 120 متر. تم تحليل الجسور لعدة حالات من توزيع الاحمال الحية ولقيم مختلفة من نسبة تقوس الجسر (L/R). بينت الدراسة ان التقوس للجسر هو من اكثر العوامل المؤثرة على التوزيع العرضي للاجهادات بين الوافد. عموما, كلما ازدادت درجة التقوس, ازدادت اجهادات اللي, وهكذا فان توزيع الاجهادات بين الروافد سيكون غير منتظم لدرجة كبيرة ومنحرف بشدة عن سلوك الجسور المستقيمة. بالاضافة لذلك, كلما ازدادت النسبة (L/R) فان حصة الاجهادات للروافد الخارجية ستزداد وللروافد الداخلية ستتناقص ولنسبة تقوس عالية فانه يحدث انقلاب في العزم والاجهاد في وسط الفضاء للروافد الداخلية. بينت الدراسة ان قيم الاجهاد الاقصى تحدث لحالة التحميل بالاحمال الحية للمواصفة AASHTO على الممر الخارجي فقط.

الكلمات المفتاحية: احمال AASHTO LRFD, الجسور المركبة, الجسور المنحنية, طريقة العناصر المحددة, معاملات توزيع الروافد