

Live Load Distribution Factor in Precast I-Girder Bridge

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Abstract. Live load distribution factors among the girders in a precast I-girder bridge have been studied. For this purpose, we consider a three-dimensional single-span simply supported bridge model in Finite Element Method. The live loads assigned for the bridge model is class 70R and class A from IRC code and HL-93M truck plus lane loading from AASHTO code for a three lane bridge. The design of the bridge and structural analysis has been done by using the computer software CSI Bridge v16.0. The aim of the present work is to find live load distribution factor (LLDF) for the displacement, bending moment and vertical shear force in different girders using FEA. LLDFs are also calculated by analytical methods using expansion in harmonics and by spring analogy. Analytical results have been compared with those calculated from FEM analysis. The results revealed that variation of LLDF in different girders is more in case of AASHTO loading compared to IRC loading. Comparison of FEM values and analytical values for LLDF shows good agreement between analytical and numerical techniques. The effect of cross girder spacing on LLDF has been examined.

1. Introduction

Girder bridges, generally consists of several parallel longitudinal girders, connected through deck slab and if necessary, through cross beams or diaphragms. Computing the response of a bridge to live loads is a complex task. Live-load distribution is a procedure to compute each girder's carrying proportion for the live load, such as the weight of trucks or cars. The distribution is necessary as often all the lanes of bridges are not loaded as well as there is unpredictable eccentricity of live load application. The design live-load moment caused by a truck (or lane of traffic) is first estimated by obtaining the maximum truck (or lane of traffic) moment on a single girder. A designer then obtains the design moments for each girder by multiplying the maximum single girder moment by a factor, which is usually referred to as the live-load distribution factor (LLDF). The AASHTO Standard Specifications for Highway Bridges contained live load distribution factors since 1931 as cited by Reece [1]. For a bridge constructed with a concrete deck on pre-stressed concrete girders and carrying two or more lanes of traffic, the current distribution factor given by AASHTO [2] simply relates girder spacing to the LLFD. This factor, multiplied by the moment on a single girder, caused by one line of wheels, gives the girder design moment. The applicability of the procedures in the Standard Specifications is limited by the fact that they were developed considering only non-skewed, simply supported bridges.



Two of the traditional simplified methods to calculate wheel load distribution are the lever-rule and the rigid diaphragm method. In lever-rule method, wheel load is distributed only between two girders adjacent to the wheel load. In rigid diaphragm method, wheel load is linearly distributed into each girder according to the rigid rotation of transverse diaphragm. The lever-rule and rigid diaphragm method represent the extreme condition of relative stiffness between the deck system and the girders. Results from these two methods may produce conservative or unsafe distribution factors because they fail to represent the actual behaviour of the bridge structure. More details of these methods have been provided in the form of discussion in a paper published by Goodrich and Packet [3]. Grillage idealization of girder slab system has been used to analyse live load response which resulted in matrix formulation [4]. Bakht and Jaeger [5] used a continuum idealization using orthotropic plate theory where flexural and torsional rigidities are uniformly distributed in two orthogonal directions. Hu [6] proposed a spring analogy method to calculate live load distribution factors in multi-girder bridge. He assumed that longitudinal girders of T or I section have spring supports at cross girder location. Verification studies by conducting field test on steel girder bridges to find live load sharing in each girder have been reported by Nowak et. al [7]. Live load distribution factor in integral abutment bridge has been studied by Dicleil and Erhan [8].

Although, several studies have been reported on live load distribution process in literatures, there is still need of complete three dimensional analyses to know how the load sharing varies when both flexural and torsional rigidities of girders contribute to the load distribution. This is significant when precast girders are transported in site and erected in position for speedy construction. In what follows, the present paper outlines a finite element method to estimate live load load distribution factors in a precast girder bridges using multilane traffic loading. The finite element results have been compared with the results using analytical methods.

2. Finite Element Modelling

We have developed a bridge model of single span simply supported pre-cast I-girder bridge which has been analyzed for LLDF under the loading condition given by IRC and AASHTO code. The modelling is done using finite element soft ware CSI Bridge v16.0. The specifications of the superstructure of the bridge are as follows:

Span length=30m; Total width of the slab=12.2 m; concrete slab thickness=220mm; wearing coat 75 mm thick; Number of longitudinal girders=5; spacing of longitudinal girders=2.35m; depth of longitudinal girder=1.4m; thickness of cross girder=0.3m; number of cross girder=6; number of lane=3; kerb (on both sides)=0.475 m wide; cables=6 numbers 12-8 ϕ , characteristic strength of concrete=45 MPa; ultimate tensile strength prestressing wire=1500 MPa; The representative sections of girder at the mid span and end span are shown in Fig.1 Finite element model consists of shell element for the deck slab and frame element for the girders. The cable has parabolic profile defined as

$$y = \frac{4h}{L^2} x(L-x) \quad (1)$$

where h is the central dip, L is the span and y is dip at distance x .

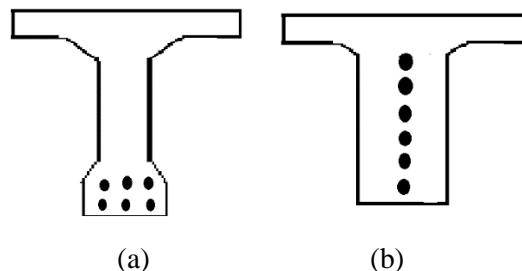


Figure 1. Representative cross section of girder at (a) mid span (b) end span

The 3D view of finite element model with the tendons in parabolic profile is shown in Fig.2. Bridge superstructure is supported by two abutments with the connection of neoprene bearings of 30 cm depth at bottom of the ends of each longitudinal girder. Mesh size for the analysis is taken as 0.5 m in both transverse and longitudinal direction.

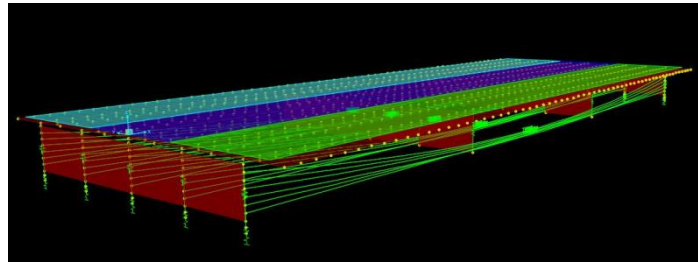


Figure 2. 3D view of FE model

In SAP 2000, the tendon object is a special type of object used to model post-tensioned cables. The object is drawn as a line and the programme automatically connects the tendon object to the nodes in the elements through which it passes. The tendon object is embedded in the girder. Two load combinations have been considered. These are (i) IRC Class A+ IRC 70R [9] and HL-93 Truck+ Lane loading [2].

3. Analytical Methods for LLDF

In order to compute the bending moment, shear and deflection in a girder of slab-girder bridges, the distribution of live loads among the longitudinal girders has to be determined. When there are only two longitudinal girders, the reactions on the longitudinal can be found by assuming the supports of the deck slab as unyielding. With three or more longitudinal girders, the load distribution is estimated using various rational approaches such as Courbons method, Hendry-Jaeger method and Morice and Little method [10]. However, these methods, although popular analytical methods in design practice, sometimes they cannot be used for restrictions in span/width ratio and cross-girder/ longitudinal girder depth ratio. In the present paper, a more generalized approach given by Jaeger and Bakht [11] has been used. The basic philosophy of this method is that the bridge is idealized as semi continuum and applied loading is decomposed in to Fourier series. However, in the instant study only first harmonic analysis has been carried out. Cross girder supports are assumed as linear springs and the problem is reduced to solving for the deflection of discrete springs. In the case of bridge with ' n ' girders there are ' n ' unknown deflections and ' n ' unknown rotations. The necessary ' $2n$ ' equations are obtained from vertical force equilibrium, moment equilibrium and slope compatibility conditions of the girder at cross girder locations.

4. Results and Discussions

Live load distribution factor in precast I-girder bridge for longitudinal girders are evaluated by both FEM analysis and analytical methods. LLDF are found for both IRC loading and AASHTO loading and results are compared. The specifications of bridges are given in section 2. The live load distribution factor in any girder is obtained as the ratio of maximum effect in that girder to the corresponding average effect of all ' n ' girders.

4.1. Comparison of FEM results with analytical results

A comparison of Distribution factor for live bending moment is presented in this section. FEM results are compared with the two analytical methods as Courbon [10] and Harmonic method [11] mentioned in Section 3. Fig.3 shows the comparative values for IRC loading while Fig.4 represents the same for AASHTO loading case considered in the paper. FEM results are very close to the analytical results and therefore, it may be stated that present FEM model gives more generalized approach for evaluation of LLDF when restriction in analytical method needs to be overcome.

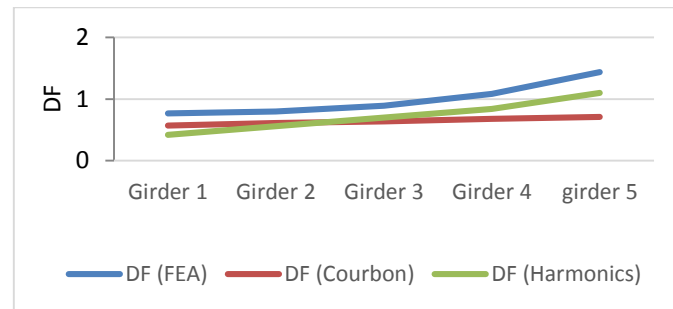


Figure 3. Comparison of Distribution Factor for live load bending moment in Girders with Different methods for IRC loading

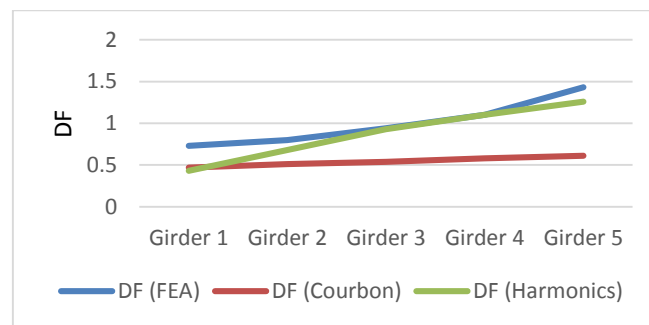


Figure 4. Comparison of Distribution Factor in Girders with Different methods for AASHTO loading

4.2 LLDF for deflection

Table 1 shows the live load distribution factor for displacement due to IRC loading standard on the five girder bridge considered in the study. The result reveals that one of the outermost girders suffers largest deflection due to twisting effect, which increases the deflection caused by the eccentricity of loading. The deflection in the severely affected girder is 60% more than the average deflection of all girders. For AASHTO loading, the maximum deflection is 26 mm in the same girder as earlier whereas outermost girder has 50% increase in deflection compared to central girder. The LLDF for deflection for AASHTO loading in order of girder number are 0.65, 0.80, 0.98, 1.17, 1.37. IRC loading showed higher value for LLDF of girders which actually does not reflect the absolute deflections in girder and changes in value for loading adopted from two codes are due to different axle load magnitude and spacing.

Table 1. LLDF for displacement due to IRC A+70R lane loading

| Girder | Maximum displacement (mm) | LLDF |
|--------|---------------------------|------|
| 1 | 2.74 | 0.43 |
| 2 | 4.27 | 0.67 |
| 3 | 5.95 | 0.94 |
| 4 | 8.15 | 1.29 |
| 5 | 10.5 | 1.66 |

4.3 LLDF for bending moment

The share of bending moment in each girder produced by live load of IRC standard considered in the study is presented in Table-2. In the case of the outermost girder near to the resultant of combined wheel and distributed load, the maximum bending moment is generated. The extracted values from the finite element output are taken. The absolute bending moment, however, was not seen to occur exactly at the mid span. In fact it occurs very near to mid-span offset by a small distance 0.46 m from the centre towards left. This, in fact is governed by the location of heavier concentrated load from the centre and the line of action of resultant load. The present case shows a mean value of 1870 kN.m bending moment and standard deviation 640 kNm. A multiplying factor of 1.55 to the standard deviation added to the average value equalizes the maximum bending moment, which in the present example conforming to the maximum LLDF obtained. For AASHTO loading case, LLDF in the present case are 0.55, 0.63, 0.78, 1.17 and 1.62 in Girder no 1,2,3,4 and 5 respectively. A multiplier factor with standard deviation for this loading is 1.45. The design bending moment for the girder design then can be found as the bending moment in the girder plus standard deviation times the multiplying factor. However, location of the maximum bending moment in any girder due to AASHTO loading differs from that of IRC loading as the axle spacing and loads are different.

4.4 LLDF for shearing force

Table 3 shows the LLDF for shear force in the girders. The maximum value of LLDF is seen in one of the outermost girder. The average value of shearing force is 335 kN and standard deviation is 180 kN. The standard deviation value is more in case of shear force compared to that in case of bending moment. For design purpose, the mean value is added to standard deviation times the multiplication factor. This multiplier in case of shear force works out to be 1.35 which is significantly deviating from maximum LLDF seen in Table-3. In case of AASHTO loading, LLDF for shear force varies in manner as 0.52, 0.35, 0.60, 1.48 and 2.05 showing an unusual trend in 1st and 2nd girder as compared to IRC loading. When more conservative approach is demanded, the highest LLDF may be used in the design.

Table 2. LLDF for bending moment due to IRC A+70R lane loading

| Girder | Maximum B.M. (kN.m) | LLDF |
|--------|---------------------|-------|
| 1 | 1308 | 0.693 |
| 2 | 1445 | 0.77 |
| 3 | 1590 | 0.86 |
| 4 | 2147 | 1.15 |
| 5 | 2864 | 1.53 |

Table 3. LLDF for shear force due to IRC A+70R lane loading

| Girder | Maximum SF (KN) | LLDF |
|--------|--------------------|------|
| 1 | 142 | 0.43 |
| 2 | 210 | 0.63 |
| 3 | 278 | 0.83 |
| 4 | 460 | 1.38 |
| 5 | 576 | 1.73 |

Table 4. LLDF for deflection in longitudinal girders for different cross girder spacing for IRC A+70 R lane loading

| Girder | Maximum Deflection (mm) and LLDF | | |
|--------|----------------------------------|----------------------------------|--------------------------------|
| | Cross girder spacing @ 6m c/c | Cross girder spacing @ 7.5 m c/c | Cross girder spacing @ 15m c/c |
| 1 | 2.74 (0.43) | 4.65 (0.48) | 8.7 (0.56) |
| 2 | 4.27 (0.67) | 7.32 (0.75) | 10.8 (0.70) |
| 3 | 5.95 (0.94) | 8.96 (0.92) | 12.5 (0.81) |
| 4 | 8.15 (1.29) | 12.3 (1.25) | 19.8 (1.28) |
| 5 | 10.5 (1.66) | 15.6 (1.60) | 25.6 (1.65) |

4.5 Effect of cross girder spacing on LLDF

Table 4 represents the maximum deflection and LLDF for the girder deflection in 5 girder precast bridge model when IRC Class A +70 R load is applied. LLDF is shown inside the parenthesis against the deflection value. To examine the effect of cross girder spacing, results are obtained for the bridge model with cross girder spacing @ 6 m c/c; @7.5 m c/c and @ 15 m c/c. It is seen that as the number of cross girder increases, deflection in longitudinal girder decreases due to increased torsional restraint provided by the more number of cross girders. However, no significant change in LLDF for the longitudinal girder deflection has been noted.

5. Conclusions

In the present study, a three dimensional finite element method has been used to find the live load distribution factor in multi-girder precast I- Girder bridge. The results are presented for two different standard of loading adopted from IRC and AASHTO codes. The comparison of finite element results has been given with analytical method. The study reveals that one of the outermost girders is severely affected in all response parameters due to its proximity of resultant axle loads, which needs to be followed to keep minimum clearance from road kerb as specified in design codes. The cross girder spacing has no significant effect on LLDF for the deflection although maximum girder deflection increases with reduction of the number of cross girders. For the present case, it has been found that AASHTO yields slightly higher values of LLDF. The analytical results shows excellent agreement with finite element results and therefore, the study recommends the use of traditional methods of live load distribution if the requirement of bridge model justifies their inherent assumptions.

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