# Wheel Load Distribution in Continuous Steel Girder Bridges Stiffened with Sidewalks and Railings

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**Abstract** This paper presents the results of a parametric study investigating the influence of railings and sidewalks on wheel load distribution in simply-supported two-span steel girder bridges using the finite element method. A total of 300 bridge cases were modeled using three-dimensional finite element analysis (FEA) subject to static wheel loading. Bridge parameters investigated in this study were the span length and girder spacing, for two-lane bridges subjected to AASHTO HS20 truck loadings positioned to produce the maximum positive/negative bending moments. Typical concrete railings and/or sidewalks were placed on either edge or both edges and assumed to be built integrally with the concrete deck slab placed on steel girders. The maximum bending moments were calculated using the FEA results for cases with railings/sidewalks which were compared to the reference bridges without sidewalks or railings and to the AASHTO procedures which do not account for the presence of sidewalks or railings in a bridge deck in their empirical equations. One- and two-span bridges are also compared. The AASHTO load and resistance factor design (LRFD) wheel load distribution formula correlated conservatively with the finite-element results and all were less than the typical AASHTO Standard formula (S/5.5). The presence of sidewalks and railings were shown to increase the load-carrying capacity by as much as 30 % if they were included in the strength evaluation of highway bridges. The results of this investigation will assist structural and bridge engineers in better designing new steel girder bridges as well as evaluating more precisely the load-carrying capacity of existing bridges with sidewalks and railings built as an integral part of the superstructure. Such presence of integral sidewalks and/or railings can also be considered as an adequate and practical method for strengthening and rehabilitating steel girder bridges.

# 1 Introduction

A common type of highway bridge deck is a reinforced concrete slab placed on steel I-beams that is referred to as a steel girder bridge. The analysis and design or load rating of such bridges could be complicated due to general geometric layout, boundary conditions. Typically, the analysis and design of highway bridges in the United States must conform to either the AASHTO Standard Specifications (Specs) for Highway Bridges (2002) prior to 2007 or the current AASHTO Load and Resistance Factor Design (LRFD) Specifications (2012). The analysis of a bridge superstructure is reduced to the analysis of single girder by using wheel load distribution factors. The distribution factor is multiplied by the longitudinal response of a single girder to a truck wheel live load resulting in the total girder response to the design truck loads applied to the bridge deck. This lateral distribution of wheel loads is a critical factor in the analysis and design of highway bridges. The current AASHTO distribution factors (Specs or LRFD) do not consider the influence of raised sidewalks and/or railings that are built integrally with the bridge deck, nor their effect on the increase of the bridge's stiffness and its load carrying capacity.

Mabsout et al. (1997a) reported a comparative study of four finite-element modeling techniques employed by various researchers. These finite-element analysis (FEA) models were used to analyze a typical one-span, two-lane, composite steel girder bridge. The maximum girder moments at critical sections and their corresponding wheel load distribution factors of the four FEA models were compared and found to be very close to each other. In addition, Mabsout et al. (1998) reported the results of a study of the effect of continuity on wheel load distribution factors for 78 bridges. Typical two-equal-span, two-lane, straight, composite steel girder bridges were selected for this study, and bridge parameters such as span length and girder spacing were varied within practical ranges. The wheel load distribution factors calculated using FEA were similar to the results obtained using the AASHTO LRFD formula and were less than the values obtained using AASHTO Standard formula (S/5.5).

Several studies were conducted on the influence of sidewalks and railings on wheel load distribution in steel and prestressed girder bridges which was shown to increase the stiffness of the superstructure and improve the load-carrying capacity of these bridges (Mabsout et al. 1997b; Eamon and Nowak 2002; Chung et al. 2006; Conner and Huo 2006; Akinci et al. 2008). In particular, Mabsout et al. (1997b) conducted a parametric study on typical one-span, two-lane, simply-supported, composite steel girder bridges to investigate the influence of various parameters such as: span length, girder spacing, raised sidewalks, and the addition of railings on live load distribution. The presence of sidewalks and railings was shown to increase the stiffness of the superstructure and improve the load-carrying capacity of steel bridges by as much as 30%.

This paper builds on previously published research by Mabsout et al. (1997a, 1997b, 1998) and other studies, by performing a parametric study investigating the influence of railings and sidewalks on wheel load distribution in simply-supported, two-span, two-lane steel girder bridges. The FEA results of 300 bridge cases that were modeled using FEA and subject to static wheel loading are presented. The maximum bending moments were calculated using the finite element analysis results and were used to compare with both AASHTO Specs and LRFD wheel load distribution factors, as well as with reference bridges without sidewalks and/r railings.

## 2 AASHTO Specifications

The wheel load distribution factors are only a function of the girder spacing according to the AASHTO Standard Specifications (2002). Typically, AASHTO design wheel loads are positioned on a girder using influence lines to produce the maximum design live load moment, which is then multiplied by an empirical load distribution factor such as S/1676, where S is the girder spacing in millimeters (or S/5.5 for steel girder bridges, where S is the girder spacing in feet). If the girder spacing is greater than 4.27 m (14 ft), AASHTO recommends the use of simple beam distribution for the estimation of the wheel load distribution factor. This AASHTO procedure has been criticized for being conservative due to its approach for using simplistic load distribution factors.

The AASHTO LRFD (2012) introduced new wheel load distribution factors based on published research in the last few decades. AASHTO LRFD wheel load distribution formulae presented in AASHTO Table 4.6.2.2.2b-1 account for parameters such as span length, girder spacing and cross-sectional properties of the bridge deck. The wheel distribution factor for bending moment in steel girder bridges is:

$$g = 0.075 + (S/2900)^{0.6}\ (S/L)^{0.2}\ [K_g / Lt_s^3]^{0.1} \qquad (1)$$

Where: $K_g = n\,(I + Ae_g^2)$

$S$ = girder spacing (mm, $1100 < S < 4900$)

$L$ = span length of beam (mm, $6000 < L < 73000$)

$K_g$ = longitudinal stiffness parameter ($mm^4$, $4x10^9 < K_g < 3x10^{12}$)

$n$ = modular ratio between beam and deck material

$I$ = moment of inertia of beam ($mm^4$)

$A$ = girder gross area ($mm^2$)

$e_g$ = distance between the centers of gravity of the basic beam and deck (mm)

$t_s$ = depth of concrete slab (mm, $110 < t_s < 300$)

## 3 Bridge Cases Analyzed

Typical two-equal-span, simply supported, two-lane steel girder bridges were selected for this investigation. The longitudinal axis of the bridges was assumed to be at right angles to the supports. The bridge deck consisted of a 190 mm (7.5 in) reinforced concrete slab supported by structural steel girders (W36X160). The various span lengths considered in this study were 12, 18, 24, 30, and 36 m (40, 60, 80, 100, and 120 ft). The girder spacing was set at 1.8, 2.4, and 3.6 m (6, 8, and 12 ft). Given that the typical lane width is 3.6 m (12 ft), and allowing for shoulder width or raised sidewalk of 1.2 m (4 ft) on each of the slab edges, the overall bridge slab widths were taken to be 9.6 m (32 ft) for two-lanes; these dimensions account for the existence of sidewalks and/or railings in the cases they are present.

The typical reinforced concrete sidewalk selected for this study was 1.2 m (48 in) wide and 190 mm (7.5 in) thick. The sidewalk was first placed on the left side of the bridge deck [1S(L)], right side [1S(R)], and then on both sides [2S] for all combinations of span lengths, width, and girder spacing's considered. Similarly, a typical reinforced concrete railing or parapet [200 mm (8 in) thick by 750 mm (30 in) high] was placed on the left, right, and on both sides of the deck [1R(L), 1R(R) and 2R] for all bridge combinations considered. Finally, the combination of sidewalk and railing were similarly added to the bridge deck on the left, right, and both sides [1SR(L), 1SR(R) and 2SR] for all bridge combinations considered. Reference bridge deck cross sections with no sidewalks and railings (thereafter referred to as the "NoSR" case) were also investigated for comparative analysis. A typical cross-section considered for a two-lane bridge with a combination of both railing and sidewalk on the right is shown in Figure 1.

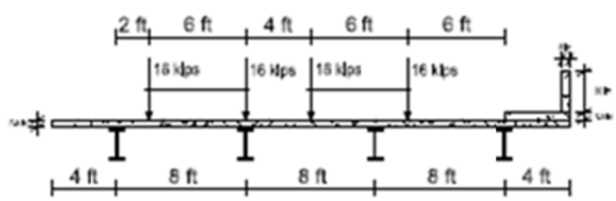


**Fig. 1.** Typical cross-section and layout for a two-lane bridge, girder spacing S=2.4 m, sidewalk/railing right (1SR(R))

## 4 Bridge Loading

Longitudinally, a train of AASHTO HS20 trucks (spaced at 9 m from each other) was positioned in each lane using influence lines in order to achieve the most severe loading conditions. Hence, only one span is loaded when looking for the maximum positive bending moment in the two-span bridges, while both spans are loaded simultaneously in order to maximize the negative bending moment at the interior support. In all cases, the trucks were assumed to be traveling in the same direction.

Transversely, the AASHTO HS20 trucks were positioned at a distance of 1.2 m (4 ft) from each other and in such a position to produce the most critical loading conditions on the bridge. The number of trucks on each bridge deck was limited to the number of lanes. The maximum girder moment was then calculated and used in determining the FEA load distribution factors.

## 5 Finite Element Analysis

The general FEA program SAP2000 (version 17) was used to generate the finite-element models. SAP2000 was used to generate nodes, elements, and meshes (element size of 0.6x0.6 m) for the steel bridges investigated. The concrete slabs and sidewalks were modeled using quadrilateral shell elements and the steel girders were idealized as space-frame elements (Mabsout et al. 1997a). The centroid of all steel girders coincided with the centroid of concrete slab elements. The railings were modeled as concentric frame elements with a moment of inertia and stiffness equivalent to an eccentric element applied on top of the slab. Hinges were assigned at one bearing location and rollers at both the interior support and opposing end to simulate simple support conditions. Figure 2 presents a sample FEA model plan-view of HS20 truck live loads on a two-span bridge.

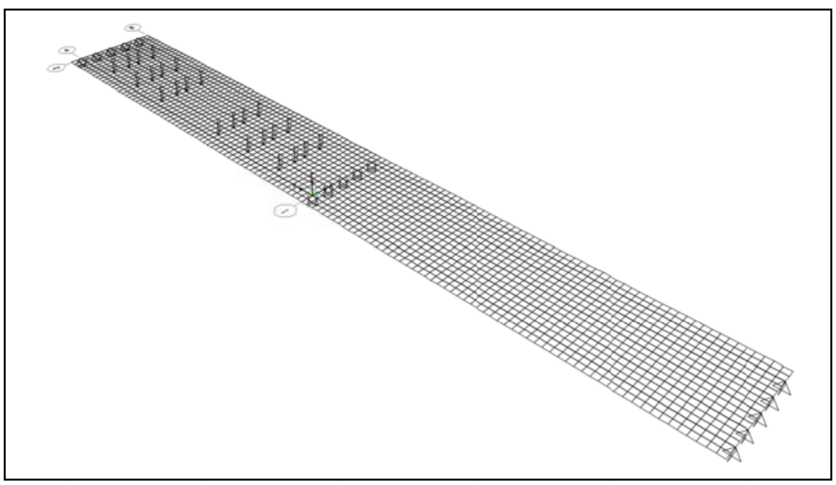

**Fig. 2.** AASHTO HS20 Trucks Loading on bridge deck (NoSR) (positive moment investigated, two-span, two-lane, span length L = 36 m, girder spacing S = 1.8 m)

## 6 Finite Element Analysis Results

Mabsout et al. (1997b) reported the results of one-span, two-lane composite steel girder bridges in the presence of integral sidewalks and railings, and compared the FEA distribution factors with LRFD equation (1) and the AASHTO Specs formulae. To complement this work, the previous study was extended to continuous two-span brides where FEA results for maximum positive and negative bending moments calculated at critical section in the concrete slab and in steel girders for all of the 300 two-span two-lane bridges were considered. The sidewalks or railings were considered to be a part of the concrete section that assisted the exterior girders in resisting wheel loads. The FEA results for bridges with sidewalks and/or railings were compared with reference bridge cases (without sidewalks or railings) as well as with AASHTO Specs and LRFD procedures.

### *6.1 FEA Results vs. AASHTO*

The maximum positive and negative bending moments were determined from FEA analysis, and corresponding distribution factors were then compared with the AASHTO Specs (DF=S/5.5) formula and LRFD equation (l).

Figure 3 shows a sample variation of all the distribution factors as a function of span length for the two-span, two-lane bridges (positive moment scenario) for a girder spacing of 2.4 m.

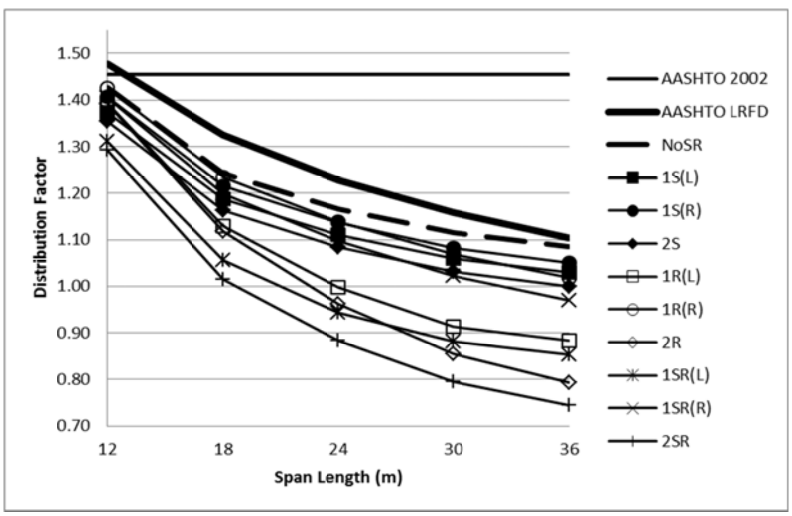


**Fig. 3.** Sensitivity of Distribution Factor to span length for interior girders (slab + steel, two-span, two lanes, positive moment scenario, girder spacing = 2.4 m)

With reference to Figure 3, AASHTO Specs (DF=S/5.5) factors are shown to be the most conservative. To a lesser extent, AASHTO LRFD equation (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings.

Table 1 presents a sample summary of the percent decrease in FEA wheel load distribution factors for the entire two-span, two-lane bridges (positive moment scenario) when compared to AASHTO LRFD Equation (1).

**Table 1.** Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab, Two Lanes) with AASHTO LRFD Equation (1)
Percent decrease in DF = [(FEA-LRFD)/LRFD] x 100

| L (m) | S (m) | LRFD | No SR | 1S (L) | 1S (R) | 2S | 1R (L) | 1R (R) | 2R | 1SR (L) | 1SR (R) | 2SR |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 | 1.8 | 1.20 | -9 | -10 | -11 | -12 | -9 | -9 | -9 | -11 | -11 | -13 |
| | 2.4 | 1.48 | -3 | -7 | -5 | -8 | -6 | -3 | -6 | -11 | -5 | -12 |
| | 3.6 | 1.98 | 2 | -1 | -1 | -4 | 1 | 1 | 0 | -4 | -3 | -8 |
| 18 | 1.8 | 1.08 | -11 | -15 | -12 | -16 | -16 | -10 | -17 | -19 | -12 | -23 |
| | 2.4 | 1.32 | -6 | -10 | -8 | -12 | -15 | -7 | -16 | -20 | -10 | -23 |
| | 3.6 | 1.77 | -6 | -10 | -10 | -13 | -12 | -10 | -16 | -17 | -15 | -25 |
| 24 | 1.8 | 1.01 | -7 | -12 | -9 | -14 | -19 | -8 | -23 | -22 | -10 | -28 |
| | 2.4 | 1.23 | -5 | -10 | -7 | -12 | -19 | -7 | -22 | -23 | -11 | -28 |
| | 3.6 | 1.64 | -7 | -11 | -11 | -15 | -17 | -14 | -24 | -22 | -19 | -32 |
| 30 | 1.8 | 0.95 | -5 | -10 | -7 | -12 | -20 | -6 | -26 | -23 | -10 | -31 |
| | 2.4 | 1.16 | -4 | -9 | -6 | -11 | -21 | -8 | -26 | -24 | -12 | -31 |
| | 3.6 | 1.54 | -6 | -10 | -11 | -14 | -20 | -16 | -31 | -24 | -21 | -37 |
| 36 | 1.8 | 0.91 | -3 | -8 | -5 | -10 | -20 | -5 | -27 | -22 | -10 | -31 |
| | 2.4 | 1.10 | -2 | -7 | -5 | -9 | -20 | -8 | -28 | -23 | -12 | -33 |
| | 3.6 | 1.47 | -4 | -9 | -9 | -13 | -21 | -17 | -34 | -25 | -22 | -38 |

The following general conclusions were reported for interior girders of typical two-lane composite steel girder bridges when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: for spans up to 24 m, the FEA distribution factors were smaller than LRFD Eq. (1) by about 10% for one-span bridges, 6% for two-span bridges (positive moment), and 2% for two-span bridges (negative moment). For spans between 24 and 36 m, FEA were smaller than LRFD Eq. (1) by about 5% for one-span bridges, 3% for two-span bridges (positive moment), and 0% for two-span bridges (negative moment).
2. Sidewalk on one side (left or right): for spans up to 24 m, FEA were smaller than LRFD Eq. (1) by about 12% for one-span bridges, 8% for two-span bridges (positive moment), and 3% for two-span bridges (negative moment). For spans between 24 and 36 m, FEA were smaller than LRFD Eq. (1) by about 9% for

one-span bridges, 6% for two-span bridges (positive moment), and 1% for two-span bridges (negative moment).

3. Sidewalk on both sides: for spans up to 24 m, FEA were smaller than LRFD Eq. (1) by about 15% for one-span bridges, 12% for two-span bridges (positive moment), and 7% for two-span bridges (negative moment). For spans between 24 and 36 m, FEA were smaller than LRFD Eq. (1) by about 13% for one-span bridges, 10% for two-span bridges (positive moment), and 5% for two-span bridges (negative moment).
4. Railing on one side (left or right): LRFD Eq. (1) was found to be about 10% higher than FEA for one-span bridges, 7% higher than FEA for two-span bridges (positive moment), and 2% higher than FEA for two-span (negative moment).
5. Railing on both sides: LRFD Eq. (1) was found to be about 20% higher than FEA for one-span bridges, 16% higher than FEA for two-span bridges (positive moment), and 10% higher than FEA for two-span (negative moment).
6. Sidewalk and railing on one side (left or right): LRFD Eq. (1) was found to be about 15% higher than FEA for one-span bridges, 12% higher than FEA for two-span bridges (positive moment), and 7% higher than FEA for two-span (negative moment).
7. Sidewalk and railing on both sides: LRFD Eq. (1) was found to be about 30% higher than FEA for one-span bridges, 25% higher than FEA for two-span bridges (positive moment), and 15% higher than FEA for two-span (negative moment).

## *6.2 FEA Results Bridges with Sidewalks and Railings vs. NoSR Reference Bridges*

Comparison of FEA distribution factors with and without sidewalks and railings for interior girders was also performed for each of the two-span cases (positive moment and negative moment). Similar results for the one-span bridge cases were taken directly from the previous work done by Mabsout et al. (1997b). The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The average reductions in FEA distribution factors when compared to the base case are summarized in Table 2.

The labels in the leftmost column of the table represent the different combinations of sidewalks and/or railings on the bridge, such as the presence of sidewalk on one side only (1S) or on both sides (2S), the presence of railing on one side (1R) or on both sides (2R), and the combination of sidewalk and railing on one side (1SR) or on both sides (2SR).

In reality, the percentage reduction in distribution factor implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a two-lane steel girder bridge superstructure. Hence, looking at the values in the last column of Table 2, one can notice the impact of adding sidewalks and railings on two-lane bridges, where a 27 % increase in load-carrying capacity can be reached.

**Table 2.** Percentage Reduction in the Distribution Factor Comparing the Different S/R Combinations to the Reference Case NoSR

| | Girder Spacing | Span Length | One-Span Positive Moment (%) | Two-Span Positive Moment (%) | Two-Span Negative Moment (%) | Average Percentage Reduction in DF (%) |
|---|---|---|---|---|---|---|
| 1S | 0 to 3.6 m | 12 to 36 m | 3 | 3 | 2 | 2 |
| 2S | 0 to 3.6 m | 12 to 36 m | 7 | 7 | 6 | 6 |
| 1R | 0 to 2.4 m | 12 to 36 m | 2 | 1 | 1 | 1 |
| | 2.4 to 3.6 m | 12 to 24 m | 5 | 4 | 3 | 3 |
| | | 24 to 36 m | 12 | 10 | 8 | 9 |
| 2R | 0 to 3.6 m | 12 to 18 m | 12 | 10 | 7 | 9 |
| | | 18 to 24 m | 20 | 17 | 12 | 15 |
| | | 24 to 36 m | 25 | 23 | 22 | 23 |
| 1SR | 0 to 2.4 m | 12 to 24 m | 3 | 3 | 2 | 2 |
| | | 24 to 36 m | 8 | 6 | 4 | 5 |
| | 2.4 to 3.6 m | 12 to 24 m | 11 | 9 | 6 | 8 |
| | | 24 to 36 m | 17 | 16 | 10 | 13 |
| 2SR | 0 to 3.6 m | 12 to 18 m | 19 | 14 | 10 | 13 |
| | | 18 to 24 m | 25 | 23 | 17 | 20 |
| | | 24 to 36 m | 30 | 29 | 25 | 27 |

## 7 Summary and Conclusions

The current AASHTO procedures (Standard Specifications or LRFD) do not consider the influence of raised sidewalks and/or railings that are built integrally with the bridge deck, nor their effect on the increase of the bridge's stiffness and load carrying capacity. The use of FEA analysis in this paper proved that the presence of sidewalks and railings has been shown to increase the load-carrying capacity of interior girders by a factor somewhere between 10% and 30%. As a matter of fact, it needs to be mentioned that the case of adding a sidewalk only at one side of a bridge (case 1S) is inefficient and is not worth to be considered; especially when comparing it to the most efficient case obtained upon the addition of both sidewalks and railings at both sides (case 2SR). Furthermore, adding a railing is shown to be more effective than adding a sidewalk; which sounds logical as the railing extends above the slab and acts like an inverted beam, thus contributing much to the stiffness of the bridge deck. Moreover, results show that more reduction in the distribution factor is observed for one-span bridges than for two-equal-span bridges (positive moment), which in turn showed bigger reduction values than two-equal-span bridges (negative moment). In addition, more reduction is observed for larger girder spacing and longer span.

The research therefore assists structural engineers in evaluating more precisely the load-carrying capacity of existing bridges, in the presence of sidewalks and/or railings. Such can also be considered as an adequate and practical method for strengthening and rehabilitating steel girder bridges.

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