AMERICAN UNIVERSITY OF BEIRUT

INFLUENCE OF SIDEWALKS AND RAILINGS ON MULTI-SPAN MULTI-LANE STEEL GIRDER BRIDGES

by
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AN ABSTRACT OF THE THESIS OF

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Title: Influence of Sidewalks and Railings on Multi-Span Multi-Lane Steel Girder Bridges

The conventional analysis and design of highway bridges ignore the contribution of sidewalks and/or railings in a bridge deck when calculating the flexural strength of superstructures. In fact, the presence of sidewalks and railings acting integrally with the bridge deck has the effect of stiffening and therefore altering the lateral wheel load distribution on highway bridges. The current research presents a parametric study to investigate the influence of typical sidewalks and railings on load distribution and load-carrying capacity of multi-span multi-lane steel girder bridges. The finite-element method is used to investigate the effect of span length, slab width, girder spacing on one-span and two-equal-spans simply supported, two-lane, three-lane, and four-lane steel girder bridges. The finite-element program SAP2000 is selected for the analysis. American Association of State Highway and Transportation Officials (AASHTO) HS20 design trucks were positioned on the bridges to produce the maximum moments. Various configurations of sidewalks and/or railings on either or both edges of the slab are considered. Bridges without sidewalks and railings served as reference cases. The wheel load distribution factor for the reference cases and for cases with sidewalks and/or railings are calculated and compared. The finite-element analysis results were also compared with AASHTO procedures. 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 40 % if they were included in the strength evaluation of highway bridges. The research will therefore assist structural engineers in better designing new steel girder bridges, or 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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CHAPTER 1

INTRODUCTION

1.1. Background

Since the early 1900s, steel bridges have been undergoing a steady evolution in design and construction. Bridge engineers have continuously attempted to improve and expand their methods of analysis, design, and construction, as new types of bridges were conceived. Often this was the result of new analysis or construction techniques. Many types of bridges are in use today, ranging from long-span suspension structures to short-span slab bridges.

A common type of bridge deck is a reinforced concrete slab placed on steel beams (I-girders) generally referred to as steel girder bridges. The analysis of these bridges is complicated by the general geometric boundaries and loading conditions. A thorough understanding of the lateral load distribution from the slab to the beams is crucial for the development of realistic designs for these highway bridges.

Typically, the design of highway bridges in the United States must conform to the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for highway bridges (2002) or AASHTO Load and Resistance Factor Design (LRFD) design specifications (2010). Each method gives different results due to the live-loading conditions.

1.2. AASHTO Design Procedures

The AASHTO Standard Specifications design procedures were originally developed and updated over the years based on research work by Westergaard (1926, 1930), Jensen (1938, 1939), and Newmark (1948). This method suggests the use of simplified procedures for the analysis and design of steel girder bridges. The analysis of a bridge superstructure is reduced to the analysis of one single girder with the introduction of wheel load distribution factors. The distribution factor is multiplied by the longitudinal response of a single girder to a truck wheel live load (i.e., half the weight of truck axle loads) resulting in the total girder response to the design truck loads on the bridge deck. This lateral distribution of wheel loads is a critical factor in the analysis and design of highway bridges. In the last two decades, however, the AASHTO LRFD bridge design specifications were developed as a comprehensive specification to incorporate the latest research, and achieve a more uniform margin of safety for all bridge structures. The new formulas are generally more complex than those previously recommended by AASHTO Standard Specifications, but they present a greater degree of accuracy.

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.

1.3. Literature Review and Background Research

Straight steel girder bridges have been investigated by many researchers in the past. Burdette and Goodpasture (1988) reported the results of a study performed to identify and evaluate aspects of bridge behavior that are not normally considered during bridge evaluation and rating. The investigators identified several potential sources of load capacity enhancement. These sources include the effects of composite action, continuity, and skew. However, quantifying the effects of these variables was thought to be difficult without the benefit of some sort of load testing. Zokaie et al. (1991) performed sensitivity studies of the wheel load distribution in steel girder bridges by varying bridge parameters. It was found that the girder spacing is the most significant parameter, followed by the span length. Tarhini and Frederick (1992) reported the results of a parametric study that demonstrated that the type of bridge deck construction (composite versus non-composite), presence of cross-bracing, variation in girder size, and variation in the concrete thickness had negligible effects on wheel load distribution factors. Mabsout et al. (1997) 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. Further studies by Mabsout et al. (1998 and 1999) were conducted on straight multi-span multi-lane steel girder bridges using a simple shell and frame models for the slab and girders, respectively. These FEA-determined wheel load distribution factors compared favourably with AASHTO LRFD Design Specifications and all were generally less than the conservative AASHTO Standard Specifications equation (S/5.5).

A previous and limited preliminary study by Mabsout et al. (1997) was conducted to investigate the influence of sidewalks and railings on wheel load distribution in steel girder highway bridges. The study was limited to one bridge case with one single span and two lanes and a limited number of combinations of sidewalks and railings. 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 and evaluation of highway bridges.

A recent and extensive study by Waked et al. (2010) was conducted to investigate the influence of sidewalks and railings on wheel load distribution in one-span concrete slab highway bridges. Typical one-span, simply supported, multi-lane (one to four lanes), reinforced concrete slab bridges were modeled and analyzed using the finite-element method and various configurations of sidewalks and/or railings on either or both edges of the slab were considered. The case of one-span bridges with no sidewalks and railings served as reference bridges. AASHTO design trucks (HS20) are assumed, longitudinally and transversally, in order to produce maximum bending moments. The wheel load distribution on the bridge slab at the critical section for the reference and continuous sidewalk/railing cases were calculated and compared. The results were also assessed with the AASHTO Standard Specifications and AASHTO LRFD Design Specifications procedures.

Furthermore, a study by Nuwayhid et al. (2014) was conducted to investigate the influence of sidewalks and railings on wheel load distribution in one-span steel girder bridges. Typical one-span, simply supported, multi-lane (two to four lanes), steel girder bridges were modelled and analyzed using the finite-element method with various configurations of sidewalks and/or railings on either or both edges of the slab considered. Similarly, the wheel load distribution on the bridge slab at the critical section for the reference and continuous sidewalk/railing cases were calculated and compared. Recommendations related to the interpretation of the effect of sidewalks and railings were proposed to bridge engineers.

The studies above by Mabsout et al. (1997 to 1999), Waked et al. (2010) and most importantly Nuwayhid et al. (2014) form the basis of the current research which addresses the influence of sidewalks and railings on multi-span multi-lane steel girder highway bridges.

1.4. Research Objectives

Sidewalks and railings or parapets acting integrally with the bridge deck have the effect of stiffening and attracting load to the slab edge and therefore altering the lateral wheel load distribution on highway bridges.

In this research, the finite-element method is used to investigate the influence of integral sidewalks and railings on the wheel load distribution and the load-carrying capacity of steel girder bridges. Typical one-span and two-equal-spans, simply supported, multi-lane (two to four lanes), steel girder bridges were considered. A parametric study was conducted where a variation of span length, slab width, and girder spacing is considered. Various configurations of sidewalks and/or railings on either or both edges of the slab were considered. The case of one-span and two-equal-spans bridges with no sidewalks and railings served as reference bridges. AASHTO design trucks (HS20) were assumed, longitudinally and transversally, positioned using influence lines in order to produce the maximum positive and/or negative bending moments on the critical girders.

The study focused on determining an accurate wheel load distribution on the girders to provide a safe and economical design of the bridge. The wheel load distribution factors at the critical section for the reference and sidewalk/railing cases were calculated and compared. The results were also assessed with the AASHTO Standard Specifications (2002) and AASHTO LRFD design specifications (2010) procedures. Recommendations related to the interpretation of the effect of sidewalks and railings were proposed to bridge engineers.

1.5. Scope and Methodology of Proposed Research

The current research presents the finite-element results of a parametric study to accurately evaluate the effect of sidewalks and railings on wheel load distribution in multi-span multi-lane steel girder highway bridges. The research dwells on previous work by the author which addressed wheel load distribution of one-span bridges with/without sidewalks and railings. It culminates the series of work done on steel girder bridges and presents a comprehensive understanding of these types of bridges.

In the present research, the finite-element modeling consisted of shells and frames for concrete slab and steel girders, respectively; and composite action between slab and girders was assumed. The finite-element program SAP2000 (version 15.2.0) was selected for the analysis. The finite-element method was used to investigate the effect of span length, girder spacing, on simply supported, one-span and two-equal-spans, two-lane, three-lane, and four-lane steel girder bridges. Five typical span lengths were investigated: 40, 60, 80, 100, and 120 ft (12, 18, 24, 30, and 36 m). Three girder spacing (6, 8, and 12 ft, or 1.8, 2.4, and 3.6 m) were examined in combination with the span lengths considered. The lane width considered in this study is 12 ft (3.6 m). The corresponding bridge width was taken to be 32 ft (9.6 m) for the two-lane bridges, 44 ft (13.2 m) for the three-lane bridges, and 56 ft (16.8 m) for the four-lane bridges, which accommodates for shoulders on each side; the total width accommodates for either shoulders only or for cases with combinations of sidewalks and/or railings on either or both sides.

The bridge live loading was assumed to produce the maximum design moments in the critical girders. Longitudinally, HS20 trucks were assumed to be traveling in the same direction. Tarhini and Frederick (1992) reported the use of a train of HS20 trucks spaced at 30 ft (9 m) to simulate the lane loading condition which governs for longer span bridges. This train of trucks was not reduced by 25% as suggested in the development of AASHTO lane loading conditions. The train of trucks was positioned in each lane using influence lines and creating the most severe loading conditions on the longer span bridge cases. Transversely, AASHTO HS20 design trucks were placed side-by-side on the bridge superstructures, with a distance of 4 ft (1.2 m) between the loading points for the two, three, and four lanes. The number of trucks positioned transversely on each bridge deck was the same as the number of lanes. The transverse position of all the trucks shown was selected in order to produce the worst loading conditions on the bridge. These positions led to calculating the maximum FEA longitudinal bending moments in one of the interior girders, which are used to compute the maximum wheel load distribution factors.

The cases of one-span and two-equal-spans bridges without sidewalks and railings were considered as the reference cases. Sidewalks and/or railings were then placed integrally at either or both of the slab edges. The maximum longitudinal bending moments (positive moments for the one-span case, and positive and negative moments for the two-equal-spans "positive" and "negative" cases) were computed and corresponding wheel load distribution factors on the girders were reported and compared for the reference bridges and the bridges with sidewalks and railings. The finite-element analysis results were also assessed with the AASHTO Standard Specifications (2002) and LRFD procedures (2010).

1.6. Thesis Organization

The Thesis is divided into five main chapters including the introduction. Chapter 2 addresses the objective of the research and presents a clear description of AASHTO design methods. Chapter 3 includes a description of the bridge cases and parameters to be studied as well as the finite-element models used in the analysis. Chapter 4 presents the collected results of the finite-element method (FEM) and assesses them in comparison to the AASHTO procedures. Finally, Chapter 5 presents a summary of the research as well as the conclusions and recommendations to be drawn.

CHAPTER 2

PROBLEM DESCRIPTION

2.1. Introduction

This chapter provides a thorough background describing steel girder bridges in general as well as a summary of existing design theories and practices (AASHTO procedures) and previous work done on steel girder bridges. It also underlines the main objective of the current research, which lies behind the use of sidewalks and railings to influence the lateral load distribution in steel girder bridges.

2.2. Effect of Sidewalks and Railings on Steel Girder Bridges

As mentioned earlier, both AASHTO procedures (Standard Specifications and LRFD) do not consider the influence of raised sidewalks and/or railings that are built integrally with the bridge deck. In this context, previous research has shown sidewalks and railings (or parapets) acting integrally with the bridge deck to produce a significant increase in the bridge-deck’s stiffness and load-carrying capacity. In fact, 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% for single-span two-lane bridges, as shown in Figure 2.1 (Mabsout et al., 2008).

Hence, building-on and combining the work done in previous research, this thesis presents the results of a parametric study that investigates the influence of sidewalks and railings on wheel load distribution in simply supported, one-span and two-equal-spans, multi-lane steel girder bridges. Bridge cases were modeled using three-dimensional (3D) finite-element analysis subject to static wheel loading. The

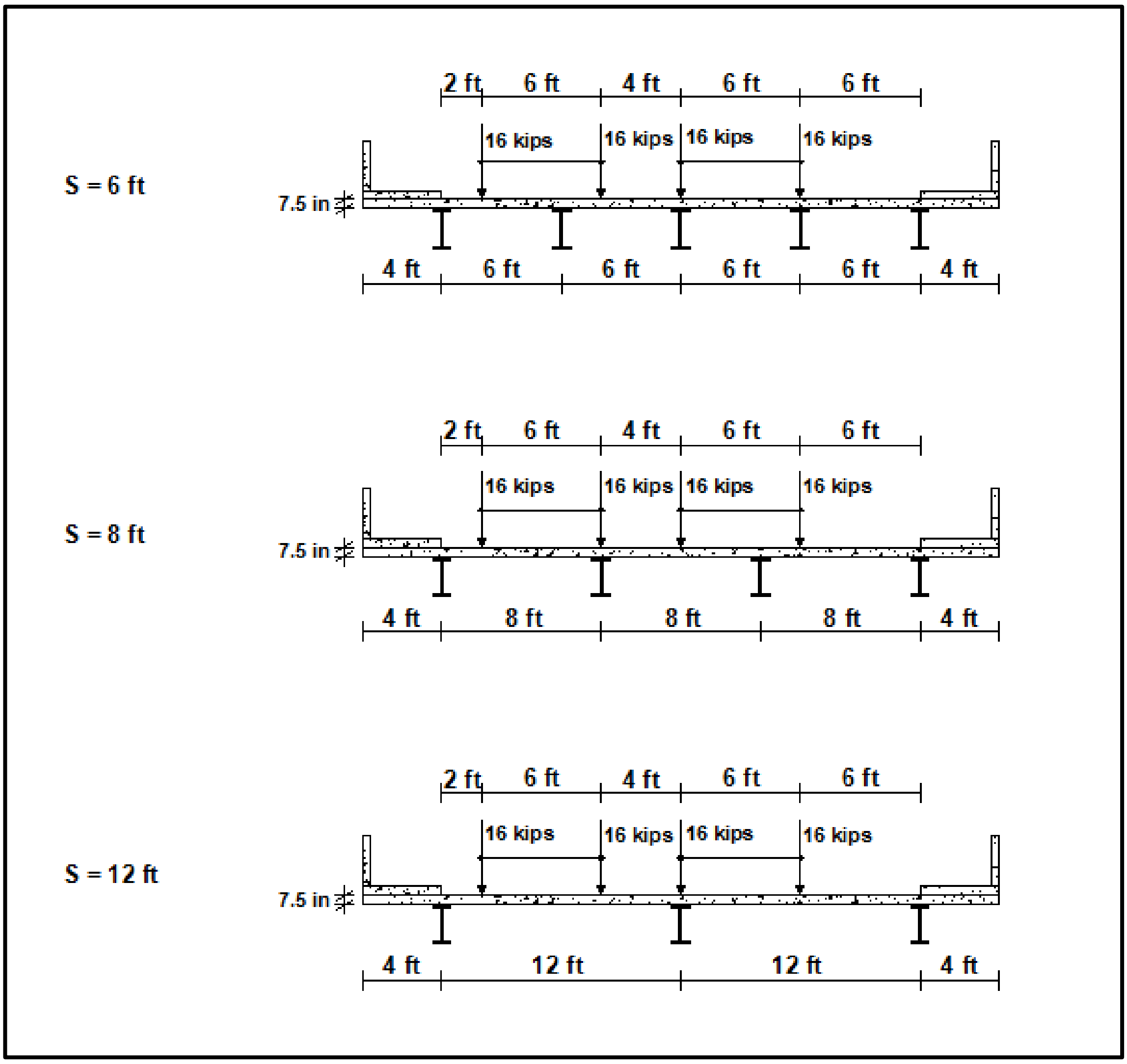


Figure 2.1. Typical Bridge Cross-section - Case of Two Lanes with Sidewalks and Railings on Both Sides

various bridge parameters investigated in this study were the span length, number of lanes (two to four), with AASHTO HS20 truck loadings positioned transversally and longitudinally to produce the maximum longitudinal live load bending moments. Raised sidewalks and/or railings were placed on either edge or both edges of the bridge deck and assumed to be built integrally with the concrete slabs. The bridge parameters were varied within practical ranges in order to investigate their effect on live load bending moments and deflections. The maximum bending moments and deflections were calculated using the finite-element analysis, and hence the distribution factor (DF) is obtained by dividing the maximum finite-element analysis (FEA) moment in the critical girder by the maximum moment computed in a simply-supported beam subject to a single line wheel load of a design truck. Results are also assessed with both AASHTO Standard Specifications and LRFD procedures.

2.3. AASHTO Design Recommendations

The procedure adopted by AASHTO for the design of steel girder bridges is to reduce the analysis of a bridge superstructure to that of a single girder with the introduction of wheel load distribution factors. Hence, the distribution factor is multiplied by the longitudinal response of a single girder to a truck wheel live load (i.e., half the weight of truck axle loads) resulting in the total girder response to the design truck loads on the bridge deck.

2.3.1. AASHTO Standard Specifications for Highway Bridges

According to the AASHTO Standard Specifications (2002), the wheel load distribution factor is only a function of the girder spacing. Typically, AASHTO design loads are positioned on the 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/5.5 for steel girder bridges, where S is the girder spacing in feet (or S/1676, where S is the girder spacing in millimeters). If the girder spacing is 14 ft (4.27 m), AASHTO recommends the use of simple beam distribution for the estimation of the wheel load distribution factor. These investigations were limited in scope to two-lane bridges.

AASHTO also specify modification factors for live loads to account for multi-lane loading. The AASHTO Standard Specifications (2002) specify that results obtained from three- and four-lane bridge decks where all lanes are loaded simultaneously are to be multiplied by 0.90 and 0.75, respectively. These reduction factors in live loads are imposed to account for the probability of having all lanes loaded at the same time and at locations along the bridge deck producing the maximum bending moment in a bridge superstructure. However, occasionally all lanes could be loaded simultaneously, and the AASHTO allows the bridge superstructure to support this overload temporarily. The AASHTO analysis and design procedures for steel girder bridges have been criticized for being conservative. This conservatism is attributed to its simplistic load distribution factors.

2.3.2. AASHTO LRFD Bridge Design Specifications

The AASHTO LRFD Bridge Design Specifications (2010) introduced comprehensive wheel load distribution factors based on considerable analytical and experimental research performed and published in the last three decades. AASHTO LRFD wheel load distribution formulae were based on NCHRP Project 12-26, which was introduced by Zokaie et al. (1991). These formulae account for parameters such as span length, girder spacing and cross-sectional properties of the bridge deck. The final report of the NCHRP Project 12-26 presented a new wheel distribution factor for bending moment in steel girder bridges as:

$$g = 0.075 + (S/9.5)^{0.6} (S/L)^{0.2} [K_g / (12.0Lt_s^3)]^{0.1} \quad (1)$$

Equivalent SI equation:

$$g = 0.075 + (S/2900)^{0.6} (S/L)^{0.2} [K_g / Lt_s^3]^{0.1} \quad (2)$$

where:

$Kg = n (I+Ae_g^2)$

S = girder spacing (ft., $3.5 \leq S \leq 16.0$) or (mm, $1100 \leq S \leq 4900$)

L =span length of beam (ft., $20 \leq L \leq 240$) or (m, $6000 \leq L \leq 73000$)

K_g = longitudinal stiffness parameter (in^4, $10{,}000 \leq K_g \leq 7{,}000{,}000$) or ($mm^4$, $4x109 \leq K_g \leq 3x1012$)

n = modular ratio between beam and deck material

I = moment of inertia of beam (in^4) or (mm^4)

A = girder gross area (in^2) or (mm^2)

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

t_s = depth of concrete slab (in, $4.5 \leq t_s \leq 12.0$) or (mm, $110 \leq t_s \leq 300$)

The above equation is recommended for highway bridges with at least two lanes, composite or non-composite, single- and multi-span steel girder bridges. The multiple lane reduction factors were built into the newly developed wheel load distribution formula. Even though this equation was recommended for bridge decks with at least four girders, the presence of three girders in a bridge deck was also investigated in this paper and the finite-element results were evaluated and compared with Equation (1).

AASHTO LRFD (2010) contain a similar expression that results in a 50% value of Equation 1. This is due to the fact that AASHTO LRFD considers the entire design truck instead of the half truck (wheel loads) as the case in the development of Equation 1 and the procedures used in the AASHTO Standard Specifications (2002).

CHAPTER 3

BRIDGES ANALYZED

3.1. Introduction

In this chapter, the different parameters influencing the distribution of load on steel girder bridges are addressed in detail, in addition to the properties of the bridges considered. Further, bridge loading is discussed extensively and the chapter concludes with the finite-element analysis discussed in brief including the properties of all elements chosen.

3.2. Bridge Description

3.2.1. Bridge Geometry and Properties

Typical one-span and two-equal-spans, simply supported, two-, three-, and four-lane steel girder bridges were selected for this study. The longitudinal axis of the bridges was assumed to be at right angles to the supports. The bridge deck consists of a 7.5 in (19.1 cm) reinforced-concrete slab supported by W36X160 structural steel (A36) girders. The span lengths considered in this study are 40, 60, 80, 100, and 120 ft (12, 18, 24, 30, 36 m). The girder spacings were set at 6, 8, and 12 ft (1.8, 2.4, 3.6 m). Given that the typical lane width is 12 ft (3.6 m), and allowing for shoulder width of 4 ft on each of the slab edges, the overall bridge slab width was taken to be 32 ft (9.6 m) for two-lane bridges, 44 ft (13.2 m) for three-lane bridges, and 56 ft (16.8 m) for four-lane bridges; these dimensions also account for the existence of sidewalks and/or railings in the cases where they are present.

The variables listed above consist of parameters already investigated in existing research and their effect on wheel load distribution in steel girder bridges (mainly the girder spacing and the span length) was reported and analyzed. Using these same basic parameters, the main additions to be investigated and that form the basis of this research are sidewalks and railings which can be present on either or both sides of the bridge deck. Hence, different combinations of sidewalks and/or railings were considered. The sidewalk [4 ft (1.2 m) wide by 7.5 in (19.1 cm) high] was first placed on the left side of the bridge deck [1S(L)], then on the right side [1S(R)], and then on both sides [2S] for all combinations of span lengths and girder spacings considered. Similarly, a typical reinforced concrete railing or parapet [8 in (20.3 cm) thick by 30 in (76.2 cm) high] was placed on the left, right, and on both sides of the deck [1R(L), 1R(R) and 2R respectively] for all bridge combinations considered. Finally, the sidewalk and railing were placed simultaneously on the left side [1SR(L)], then both were placed to the right [1SR(R)] and last sidewalks and railings were placed on both sides of the bridge deck [2SR] for all combinations of girder spacings and span lengths considered. It was assumed that the sidewalks and/or railings were properly reinforced and connected integrally to the bridge deck in order to transmit the shear forces and to act integrally with the superstructure. No expansion joints were assumed to be present in the bridge deck. Base reference bridge deck cross-sections with no sidewalks and railings (thereafter referred to as the "NoSR" case) were also investigated for comparative studies. Sample cross-sections considered for two-, three-, and four-lane bridge cases with and without sidewalks and/or railings are shown in Figures 3.1(a) to 3.1(f). Furthermore, Table 3.1 summarizes the variation of parameters among the bridges studied which sum up to a total of 450 bridges studied for a given span. Hence,

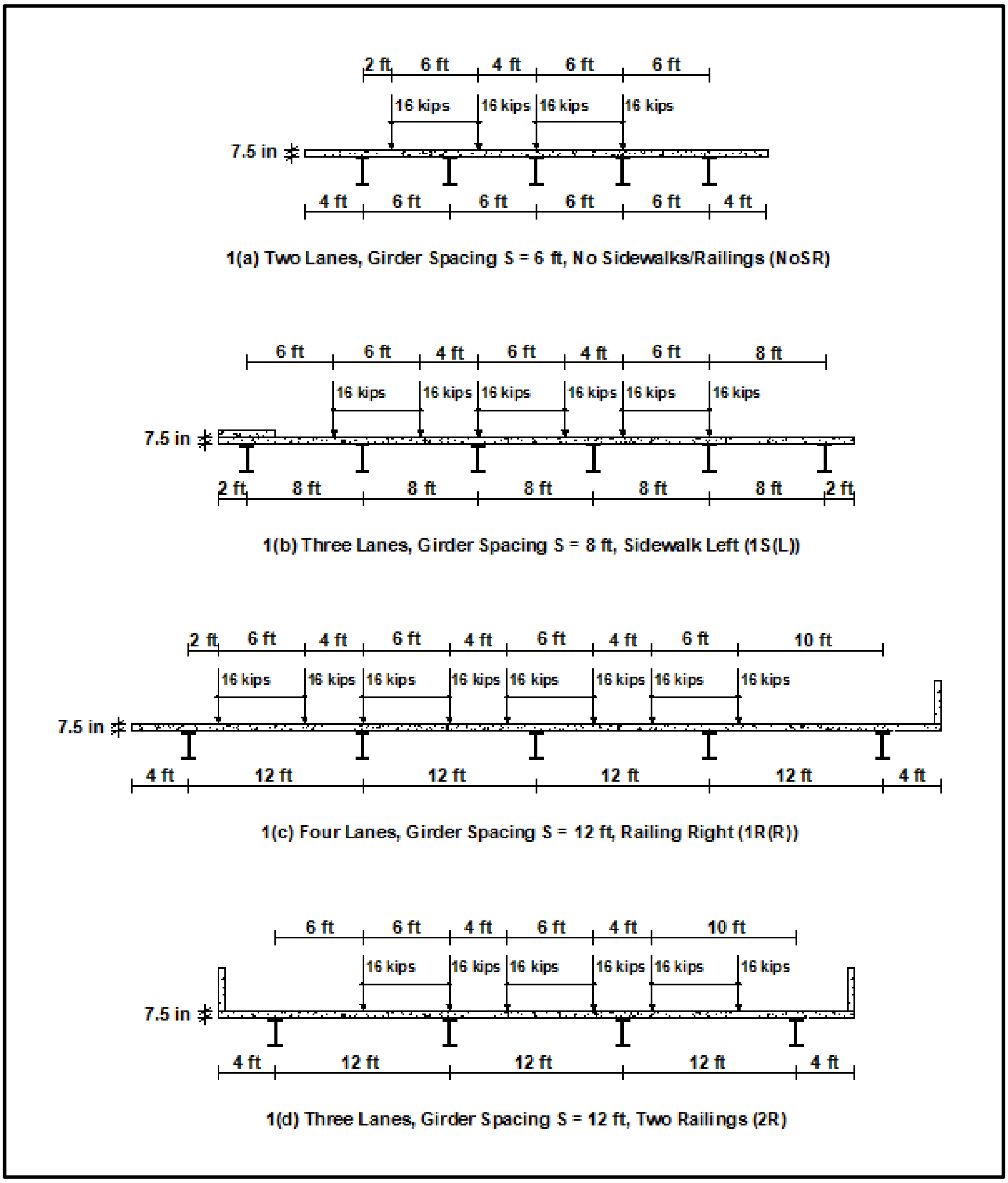
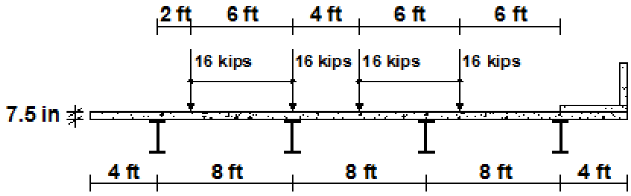
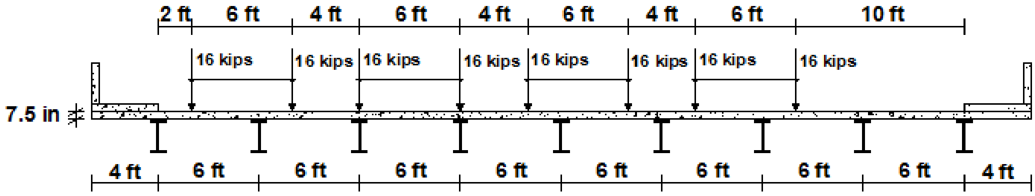


Figure 3.1. Typical Bridge Cross-Sections, with and without Sidewalks and/or Railings



1(e) Two Lanes, Girder Spacing S = 8 ft, Sidewalk/Railing Right (1SR(R))



1(f) Four Lanes, Girder Spacing S = 6 ft, Two Sidewalks/Railings (2SR)

Figure 3.1 (Continued)

Table 3.1. Variable Parameters Investigated and Geometric Characteristics of the Modeled Bridges

Number of Lanes	Span Length L (ft)	Girder Spacing S (ft)	Different Combinations of Sidewalks and Railings	Total Number of Bridges
2	40	6,8,12	NoSR, 1R(L), 1R(R), 1S(L), 1S(R), 1SR(L), 1SR(R), 2R, 2S, 2SR	150
	60			
	80			
	100			
	120			
3	40	6,8,12	NoSR, 1R(L), 1R(R), 1S(L), 1S(R), 1SR(L), 1SR(R), 2R, 2S, 2SR	150
	60			
	80			
	100			
	120			
4	40	6,8,12	NoSR, 1R(L), 1R(R), 1S(L), 1S(R), 1SR(L), 1SR(R), 2R, 2S, 2SR	150
	60			
	80			
	100			
	120			
Total				450

450 models were generated to investigate the positive moment in the one-span bridge case, while 900 models were used to study the two-equal-spans bridge cases, which split into 450 cases required to check for the two-span maximum positive moment and another 450 needed to check for the two-span maximum negative moment. In sum, a total of 1350 bridges were investigated in this research.

3.2.2. Physical Properties of Materials

Concrete

Typical normal strength concrete was assumed in the modeling of the bridge superstructure with the following properties:

- Compressive Strength: f'_c (28 days) = 4,000 psi (27.5 MPa)
- Modulus of Elasticity: $E_c = 3.6 \times 10^6$ psi (24.8 GPa)
- Poisson's ratio: $v = 0.2$

Steel

- Steel beams were modeled as W36x160 with a Modulus of Elasticity (E_s) equal to 29×10^6 psi (200 GPa).

3.2.3. Bridge Loading

According to AASHTO Standard Specifications for Highway Bridges (2002), the structural analysis of highway bridges must take into consideration either truck or lane live loading. Generally, the analysis of a highway bridge must therefore consider these two load cases separately and adopt the governing one. For the purpose of this research however, the bridge loadings considered herein were restricted to AASHTO truck loading conditions only, based on the assumption that the two-way slab bending problem can be reduced to a one-way (beam) bending with truck loading being the governing load case for the bridges studied. Therefore, AASHTO HS20-44 (see Figure 3.2) design trucks were used, having a total weight of 72 Kips (324 kN), distributed over two rear axles of 32 Kips (144 kN) each and one front axle of 8 Kips (36 kN). All three axles are equidistant with a 14ft (4.2 m) distance separating adjacent axles.

Longitudinally, trucks were assumed to be travelling in the same direction. Based on previous research done by Tarhini and Frederick (1992), a train of AASHTO HS20 trucks was placed on each lane of a given bridge to simulate the lane loading condition which prevails for long-span bridges; with a spacing of 30 ft (9 m) separating adjacent trucks. This train of trucks was not reduced by 25% as suggested in the development of AASHTO lane loading conditions. For every bridge investigated, the train of trucks was positioned longitudinally in each lane using influence lines in order to achieve the most severe loading conditions. In this context, it should be noted that for the two-span cases; when looking for the maximum positive moment in the two-equal-spans bridges, only one of the two equal spans was loaded with trucks while the other span was left unloaded (free of trucks), since loading the adjacent span would cause reduction of the positive moment in the second span; as reflected in the influence line diagram of positive moment in any two-equal-spans bridge (see Figure 3.3). However, when looking for the maximum negative moment in a two-equal-spans bridge, both spans were simultaneously loaded with a train of trucks to maximize the negative moment at the interior support as reflected by the influence line diagram for negative moment in a two-span bridge (See Figure 3.3). Finally, concerning the one-span bridges, the maximum moment was located according to Barre’s theorem, which states

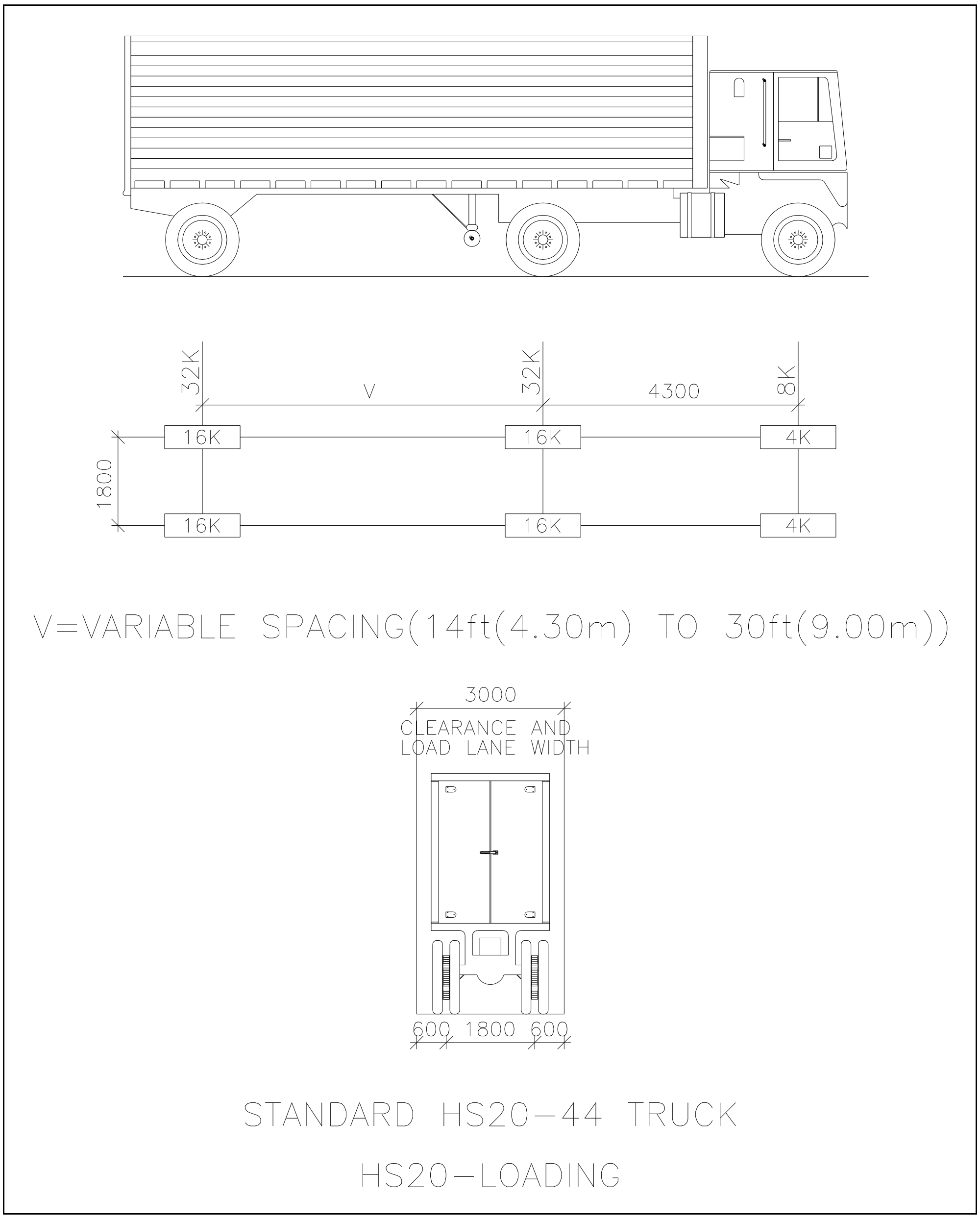


Figure 3.2. Longitudinal and Transversal Sections of AASHTO HS20 Design Truck

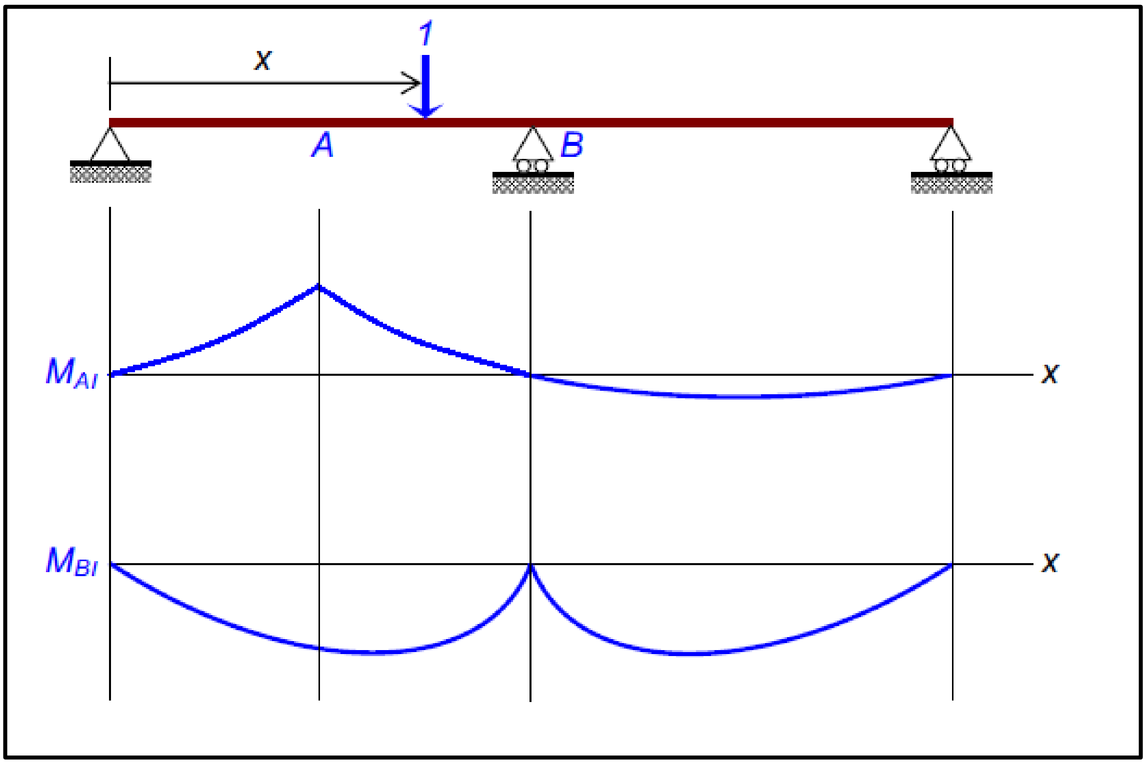


Figure 3.3. Influence Line Diagrams for Maximum Positive Moment at Point A and Maximum Negative Moment at Point B in a Two-equal-spans Beam

that for a series of point loads (truck loads in our case) moving on a single-span, simply supported bridge; the maximum moment occurs when the span's midpoint lies midway between the resultant and the nearest load. Hence, the maximum moment in any one-span bridge was located according to Barre's theorem and maximum positive moment was calculated accordingly. Figures 3.4, 3.5 and 3.6 present the longitudinal positioning of trucks on one-span and two-equal-spans bridges which was determined based on influence lines in order to produce the maximum positive moment in a one-span bridge, the maximum positive moment in a two-equal-spans bridge and the maximum negative moment in a two-equal-spans bridge respectively.

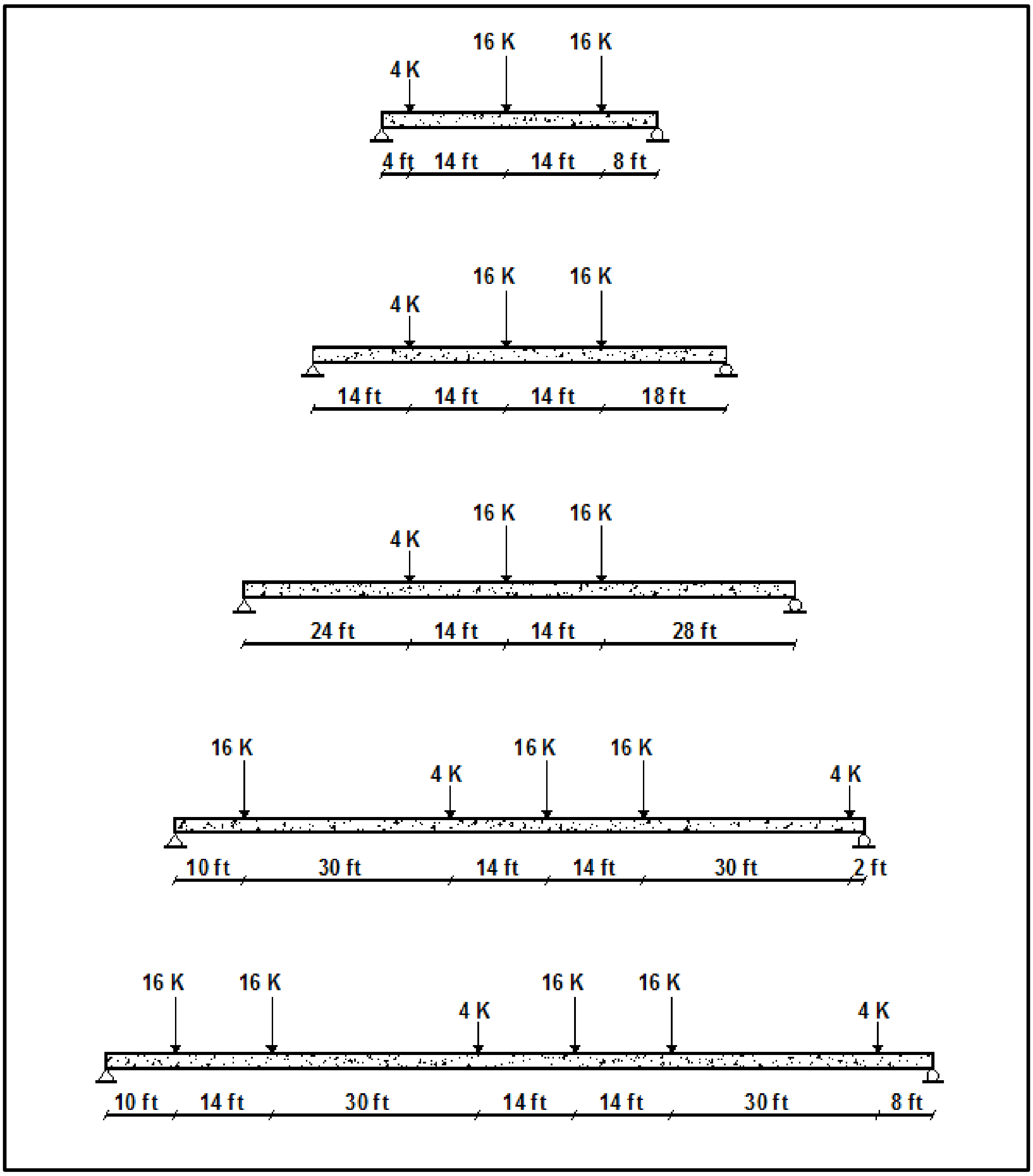


Figure 3.4. Longitudinal Beam Section of a Single-Span Bridge and Critical Position of HS20 Trucks for the Five Different Span Lengths

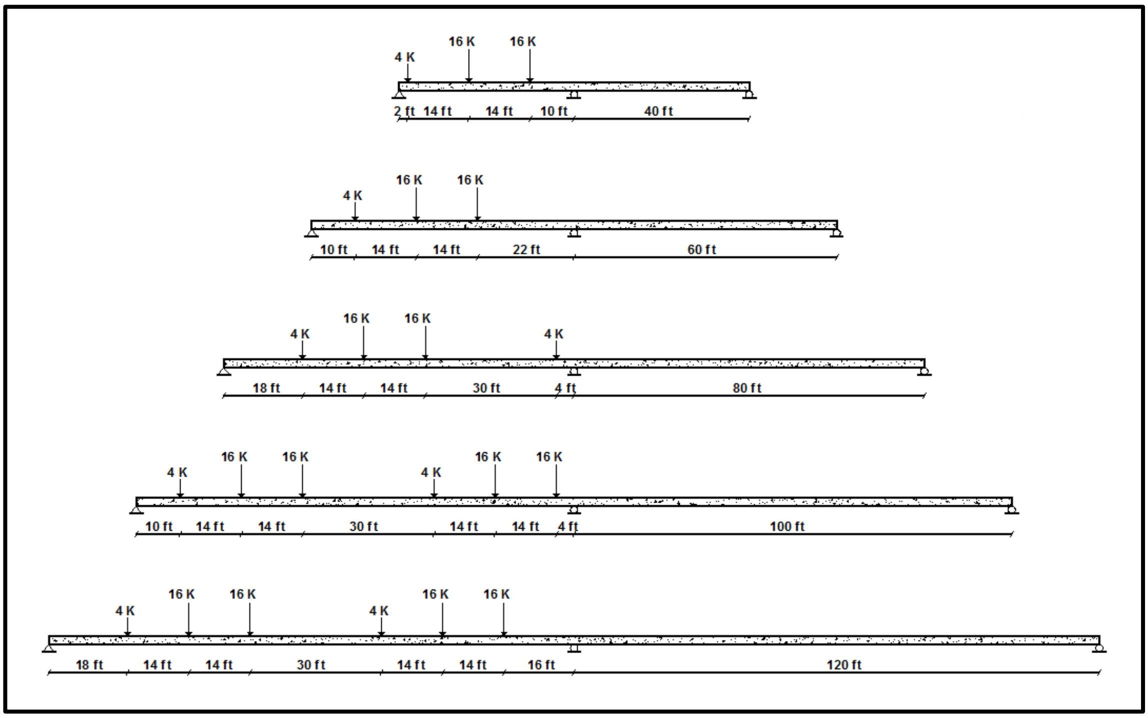


Figure 3.5. Longitudinal Beam Section of a Two-Equal-Spans Bridge and Critical Position of HS20 Trucks That Produce the Maximum Positive Moment for the Five Different Span Lengths

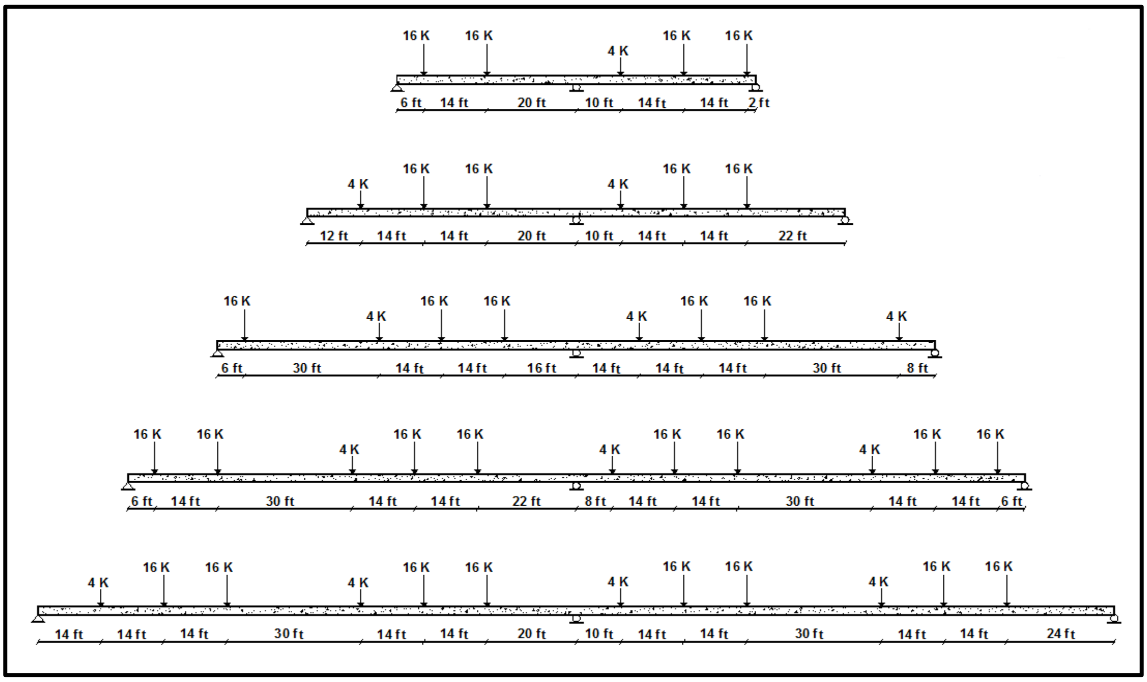


Figure 3.6. Longitudinal Beam Section of a Two-Equal-Spans Bridge and Critical Position of HS20 Trucks That Produce the Maximum Negative Moment at Interior Support for the Five Different Span Lengths

Transversally, the AASHTO design trucks were positioned side-by-side on the bridge superstructures, with a distance of 4 ft (1.2 m) between the loading points. The number of trucks on each bridge deck was limited to the number of lanes. Based on previous research related to the subject, the transverse position of the trucks was selected in order to produce the most critical loading conditions on the bridge. The maximum girder moment was then calculated and used in determining the FEA load distribution factors. Typical truck lateral load cases adopted for two-, three-, and four-lane bridges are shown in Figures 3.7 through 3.9.

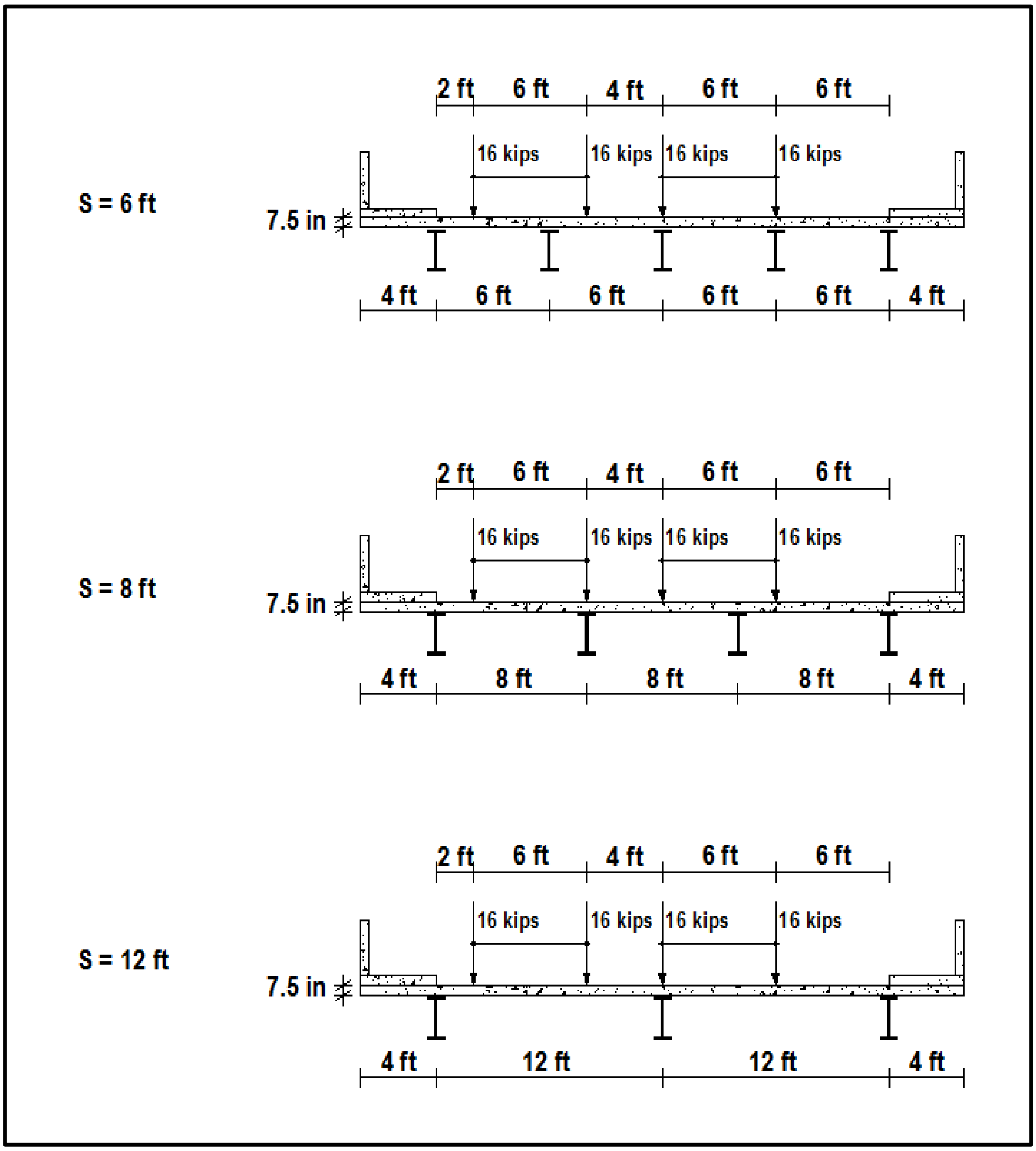


Figure 3.7. Typical Cross-Section of Two-Lane Bridges (Case 2SR) and the Critical Transverse Position of HS20 Trucks for the Different Girder Spacing Considered

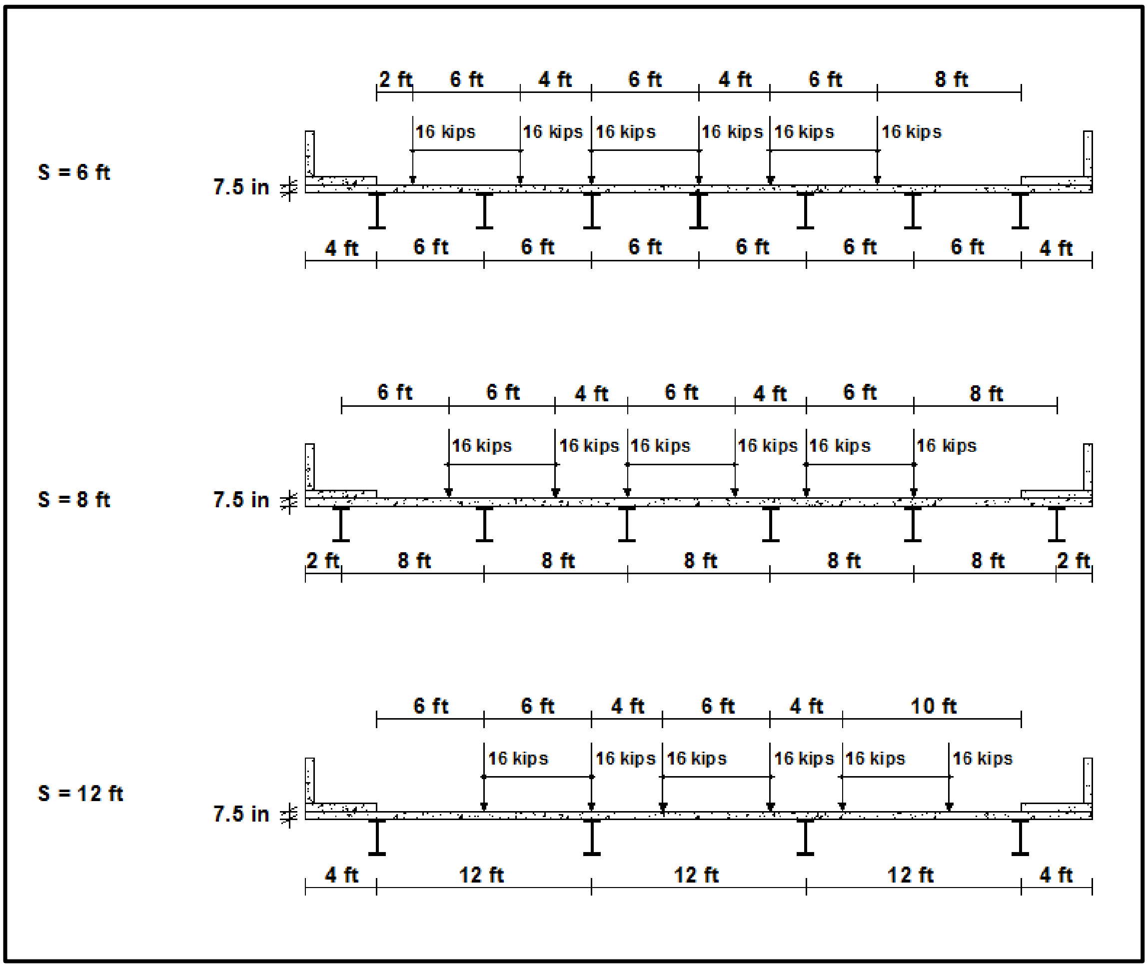


Figure 3.8. Typical Cross-Section of Three-Lane Bridges (Case 2SR) and the Critical Transverse Position of HS20 Trucks for the Different Girder Spacing Considered

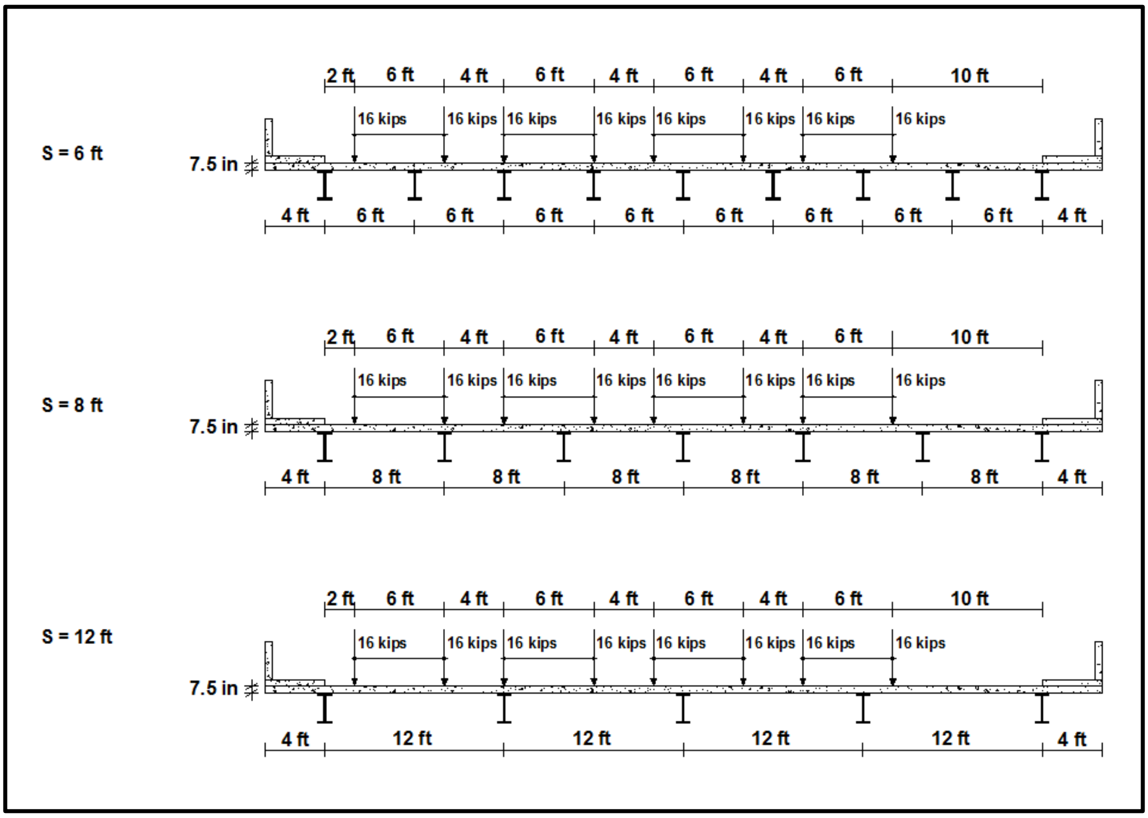


Figure 3.9. Typical Cross-Section of Four-Lane Bridges (Case 2SR) and the Critical Transverse Position of HS20 Trucks for the Different Girder Spacing Considered

3.3. Finite Element Analysis

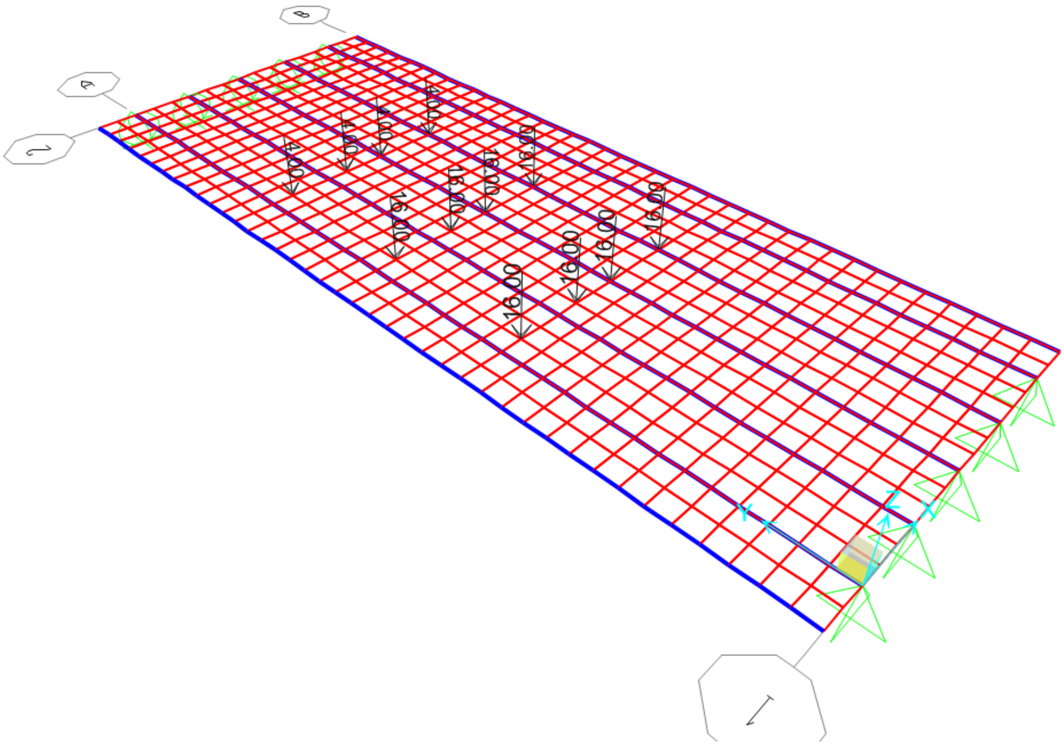
The geometry of a bridge superstructure can be idealized for theoretical analysis in many different ways. The various assumptions and simplifications used in formulating and idealizing the bridge superstructure can have a significant effect on how closely the calculated results match the actual behavior. The finite-element method can be used to predict the actual behavior of complex structures. Bridge superstructures can be modeled using FEA in many different ways. It is in the idealization phase of the analysis – the selection of the finite-element models – that the greatest differences in approaches are encountered. Mabsout et al. (1997) reported a comparative study of four finite-element modeling techniques employed by various researchers. It was shown that the FEA model idealizing the concrete slab as quadrilateral shell elements and the steel girders as space-frame members, with the centroid of the girders in the same plane as the concrete slab, can be used to accurately predict wheel load distribution.

The general FEA program SAP2000 (version 15.2.0) was used to generate the three-dimensional (3D) finite-element models. This study considered all elements to be linearly elastic and the analysis assumed small deformations and deflections. SAP2000 was used to generate nodes, elements, and 3D meshes for the slab bridges investigated. The concrete slabs and sidewalks were modeled using quadrilateral shell elements (SHELL, with 6 degrees of freedom at each node), choosing a membrane and plate bending behavior and neglecting shear deformations. On the other hand, steel girders were idealized as space-frame members (FRAME, with six degrees of freedom at each node). The centroid of all steel girders coincided with the centroid of concrete slab elements. The railings were modeled as concentric frame elements (FRAME, with six degrees of freedom at each node) with a moment of inertia and stiffness equivalent to an eccentric element applied on top of the slab. The external supports were assumed to be located along the centroidal axes of the beam elements. For the one-span bridges, hinges were assigned at one bearing location and rollers at the other to simulate simple support conditions, while for two-span bridges, hinges were assigned at one end and rollers at both the interior support and the opposing end to simulate simple support conditions. AASHTO HS20 wheel loads were applied at isolated nodes in order to produce maximum longitudinal bending moments. A typical square element size of 2x2 ft (0.6x0.6 m) was tested and adopted for the slab discretization.

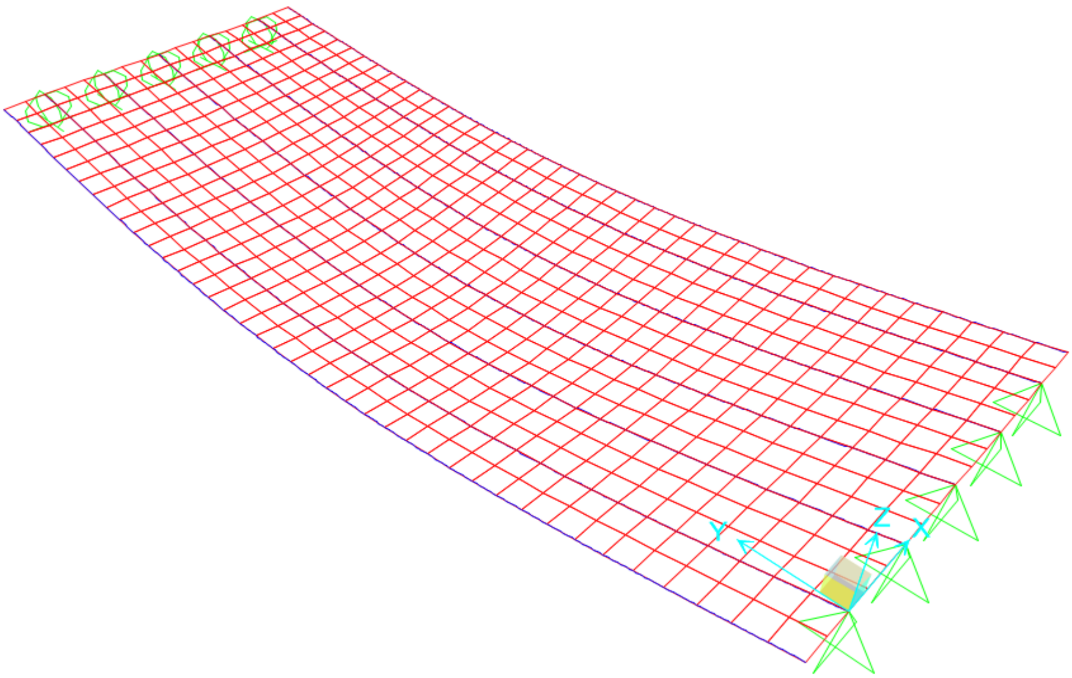
The relevant output to be extracted from the finite-element analysis (SAP 2000) includes the deflection at the nodes and the maximum longitudinal bending moment (positive and negative moments) in the girders as well as the transverse bending moments in the slab shell elements. SAP2000 generates the required longitudinal bending moment diagrams in the girders, and contour plots of the transverse bending moments in the slab. Figures 3.10, 3.11, and 3.12 present sample finite-element plans of truck live loads, deflections, and moment contours for typical one-span and two-equal-spans bridges.

The next step involved the extraction of the maximum longitudinal bending moments of all interior girders, and then adding to them the contribution of slab shell moments in order to calculate the total moment carried by any interior girder. At this stage, it should be noted that this step was performed only for interior girders, as adding the corresponding shell (slab sidewalk) moment to the longitudinal bending moment of exterior girders would be considered an overestimate of the maximum design moment of the girders. In fact, the presence of sidewalk at either end adds to the slab thickness, and hence induces additional stiffness in the slab and increases its capacity. In such a case, it would be wrong to add the moment from the slab edges to the longitudinal bending moment of exterior girders, as the stiff slab at the edge should be considered a fixed support due to its relatively high stiffness compared to the interior part of slab which is not covered by sidewalks. As a result, the longitudinal moment of every interior girder was calculated, and the contribution of the slab is added to calculate the total moments carried by all interior girders. Next, the maximum moment in the critical girder was divided by the maximum moment calculated in a single girder subject to truck wheel loads in order to calculate the distribution factor. One should keep in mind that exterior girders have been neglected so far since slab-moment contribution shouldn't be added to the longitudinal bending moment of exterior girders. However, doing so would disregard the contribution of exterior girders to the analysis, and one can argue that in some cases, one of the two exterior girders could carry the highest longitudinal bending moment had longitudinal bending moment been considered separately. Therefore, a similar analysis was conducted in parallel in which only the longitudinal bending moment was considered for all girders (interior and exterior ones). Similarly, the maximum moment in the critical girder and the distribution factor were calculated for all bridge cases. In sum, for any bridge case analyzed, two types of distribution factor were extracted; the first one includes slab+girder moments but considers only interior girders, while the other one scans all steel girders but takes into consideration only the longitudinal bending moment in the girders.

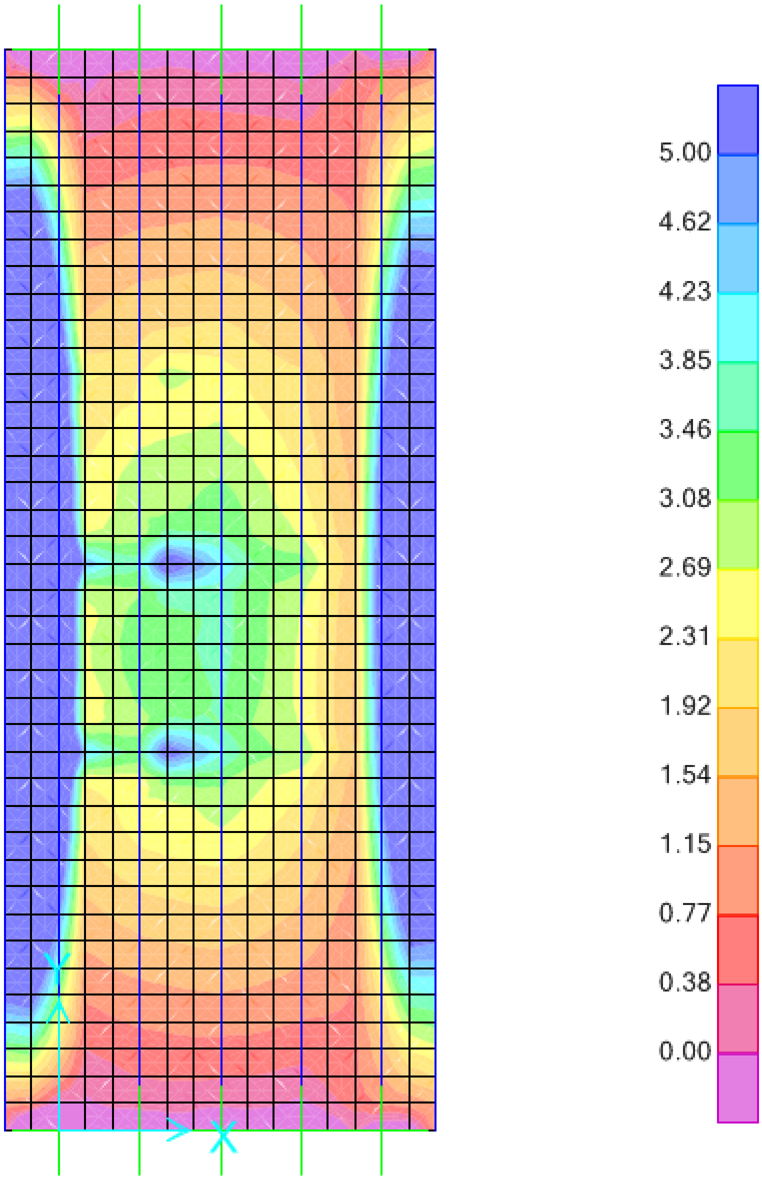
Finally, the finite-element results for all bridges with different combinations of sidewalks and railings are summarized and compared to both the reference case (case without sidewalks and railings) as well as the AASHTO Standard and LRFD procedures. Recommendations are given to assist bridge engineers in evaluating the capacity of existing bridges and in the design of future bridges.



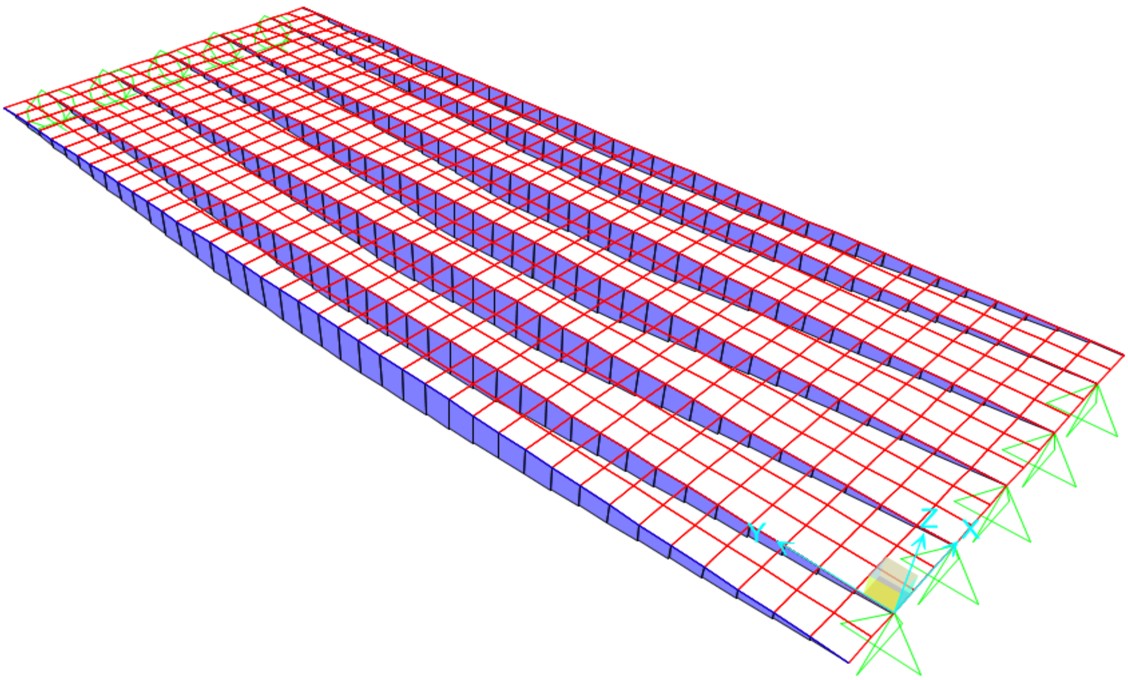
3.10(a) Truck Wheel Live Load



3.10(b) Deformed Shape

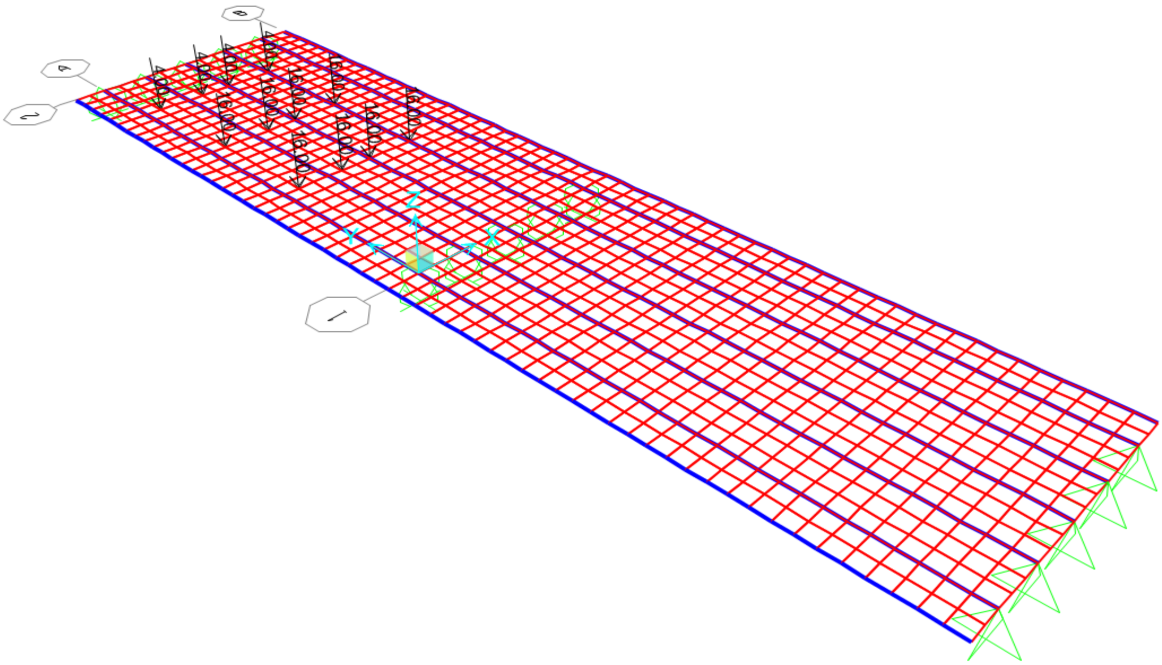


3.10(c) Transverse Bending Moment Mesh Plot in Slab Shell Elements

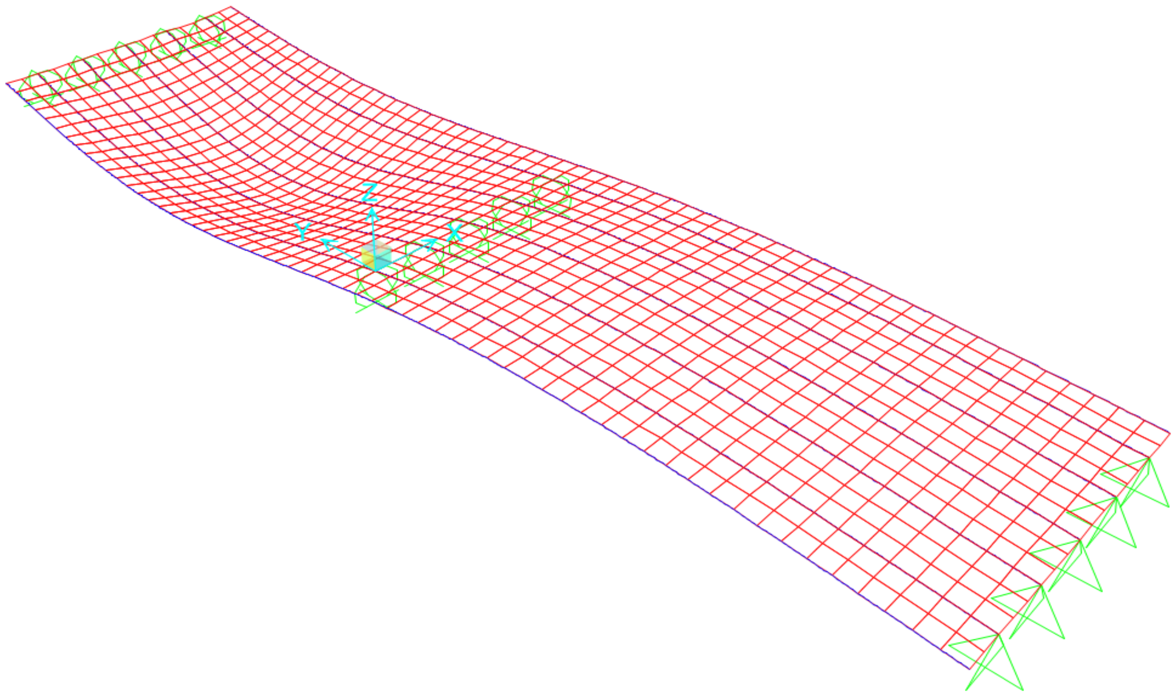


3.10(d) Longitudinal Bending Moment in Steel Girders

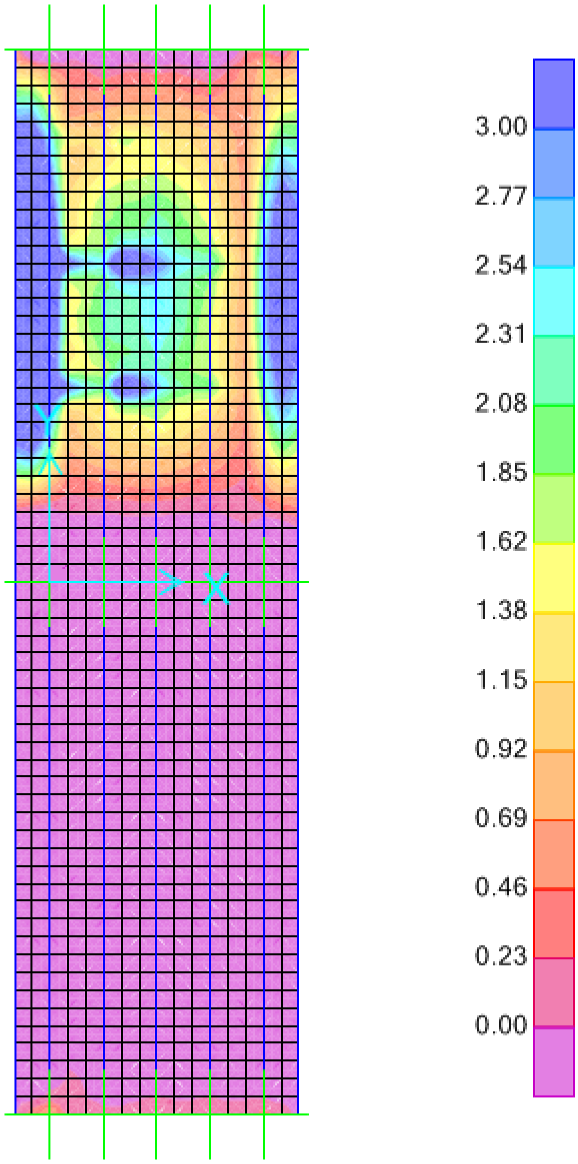
Figure 3.10. Finite-Element Model for an 80 ft Span, Two-Lane One-Span Bridge, with 6 ft Girder Spacing (Case 2SR)



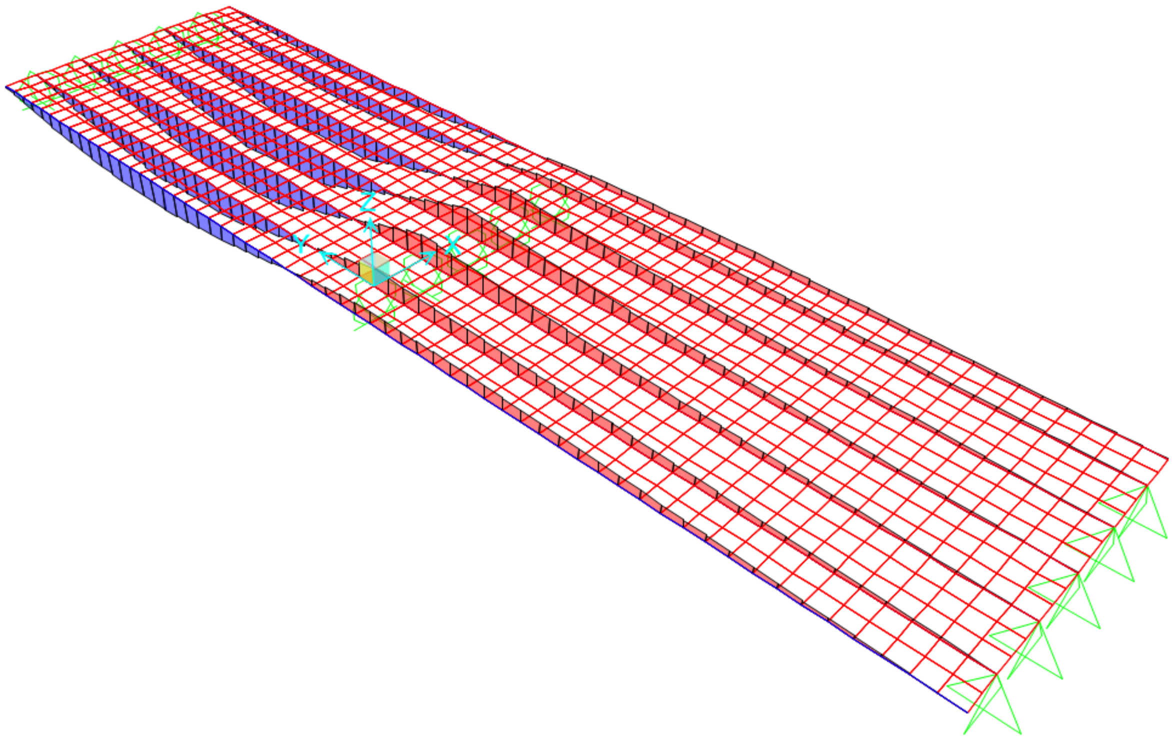
3.11(a) Truck Wheel Live Load



3.11(b) Deformed Shape

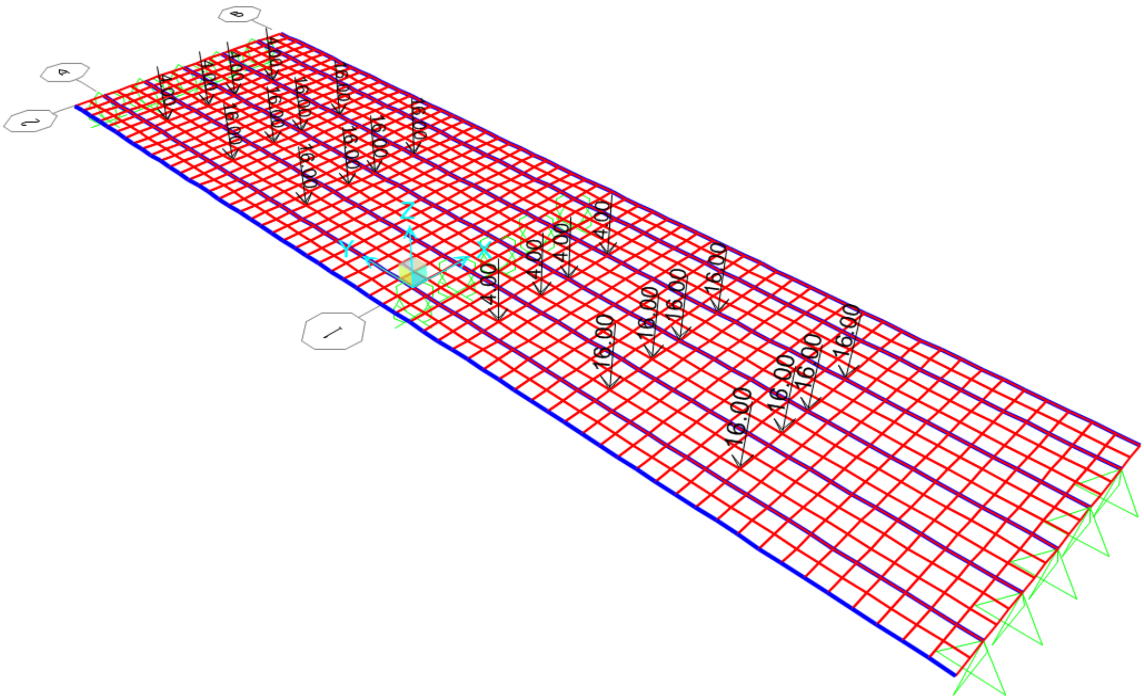


3.11(c) Transverse Bending Moment Mesh Plot in Slab Shell Elements

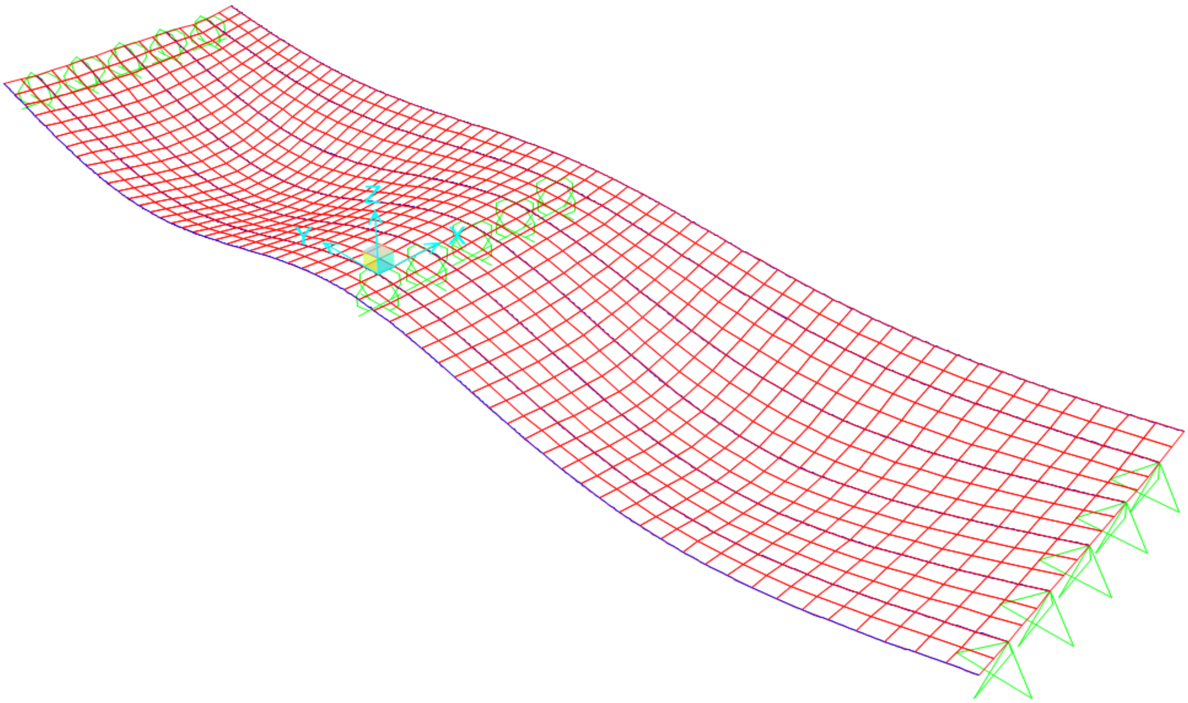


3.11(d) Longitudinal Bending Moment in Steel Girders

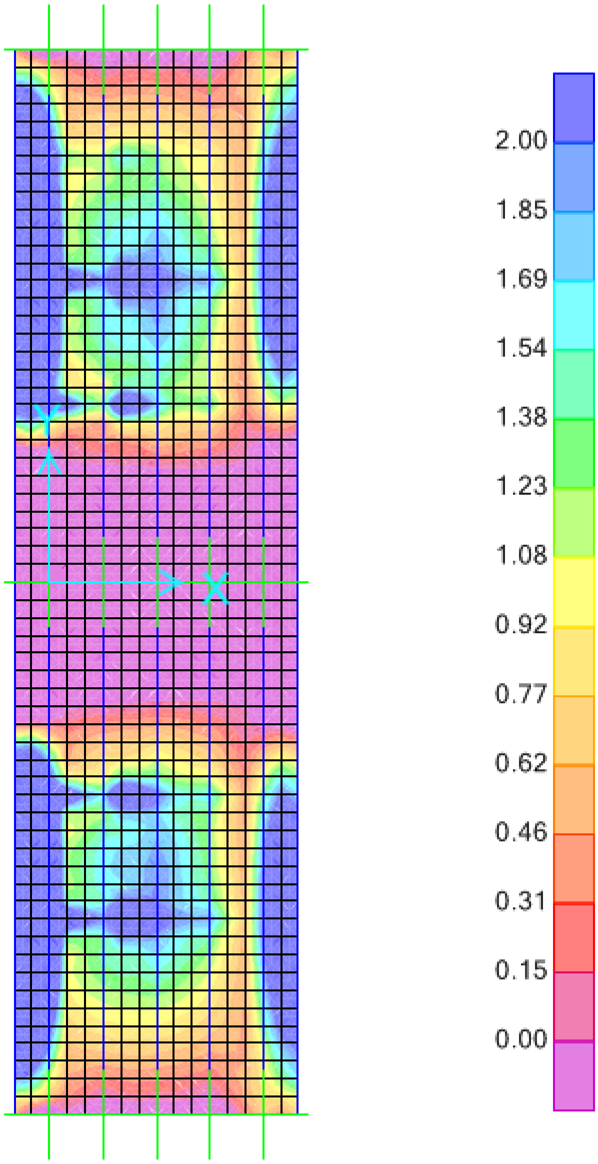
Figure 3.11. Finite-Element Model for a 60 ft Span, Two-Lane Two-Equal-Spans Bridge, with 6 ft Girder Spacing (Case 2SR) - Only One Span Loaded



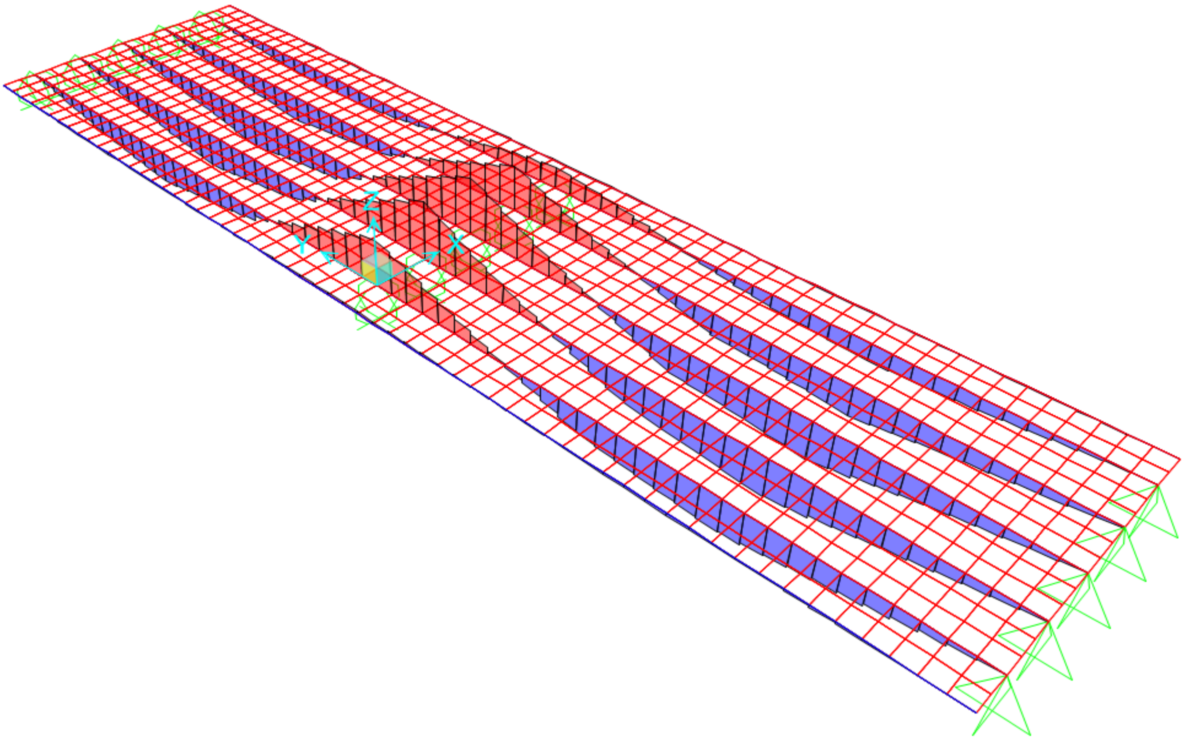
3.12(a) Truck Wheel Live Load



3.12(b) Deformed Shape



3.12(c) Transverse Bending Moment Mesh Plot in Slab Shell Elements



3.12(d) Longitudinal Bending Moment in Steel Girders

Figure 3.12. Finite-Element Model for a 60 ft Span, Two-Lane Two-Equal-Spans Bridge, with 6 ft Girder Spacing (Case 2SR) - Both Spans Loaded Simultaneously

CHAPTER 4

FINITE ELEMENT ANALYSIS AND DISCUSSION

4.1. Introduction

This chapter is divided into 3 main sections, each of which contains a total of 450 bridge cases studied and analyzed using the finite-element software SAP2000 (version 15.2.0). As mentioned in Chapter 3, several output features can be extracted from SAP2000, such as the deformed shape of the bridge under applied truck loads as well as the longitudinal bending moments in both the girders and the slab. Hence, the first section consists mainly of extracting the positive moments and their corresponding distribution factors from single-span bridge cases, while the two other sections consist of extracting the maximum positive and negative moments (along with their corresponding distribution factors) from two-span bridge cases. Results are presented in both table and chart (graph) formats. Finally, results are assessed with both AASHTO Standard Specifications (2002) and AASHTO LRFD (2010) procedures and the effect of sidewalks and/or railings on resisting truck loads applied on steel girder bridges is observed. To note that AASHTO LRFD (2010) procedures are represented by AASHTO (NCHRP) which in turn is defined by equation (1) in chapter 2. In fact, AASHTO (LRFD) contains a similar expression that results in a 50% value of Equation (1). This is due to the fact that AASHTO (LRFD) considers the entire design truck instead of the half truck (wheel loads) considered in the development of Equation (1) and the procedures used in the AASHTO Standard Specifications (2002).

The FEA reports stresses in the shell elements and forces or moments in the frame elements. Typically, stresses are proportional to the bending moments under linear elastic conditions.

The girder moments were calculated in two parts: the first was the bending moment contribution of the effective concrete slab and the second was the bending moment in the steel frame element. The finite-element stresses in the concrete slab were identified over the contributing area (effective flange width) that was used in calculating the bending moment from the slab.

The sidewalks and railings assisted in resisting the wheel loads and the bending stresses were redistributed in the bridge deck. The sidewalks or railings were considered to be a part of the concrete section that assisted the exterior girders in resisting wheel loads. This assumption modified the bridge behavior in resisting highway loadings.

The use of FEA results in calculating the maximum bending moments in the bridge deck at critical sections, usually in exterior girders, will overestimate the applied loading on a highway bridge due to the presence of sidewalks and/or railings. Typically, the bending moment in an exterior girder is higher than the bending moment in an interior girder due to the contribution of sidewalks and/or railings. However, if the sidewalks and/or railings are ignored, the bending moments in the exterior steel girders are typically smaller than the moments in the interior steel girders. Therefore, the maximum wheel load distribution for interior girders (due to the combination of moments from concrete slab and steel beam) will be compared with AASHTO formulas in order to determine the effect of sidewalks and railings on the bridge superstructure. Furthermore, the maximum wheel load distribution due to bending in the steel beams will also be compared with AASHTO formulas.

4.2. One-Span Bridges - Positive Moment

4.2.1. Two-Lane Bridges

The results for the one-span bridges have been reported by Nuwayhid (2014) and are included here in Section 4.2 and in details for the purpose of preventive study in this thesis which include, in addition to the one-span bridge cases, the two-equal-spans bridge cases for maximum positive and negative moments, reported in Sections 4.3 and 4.4, respectively.

Tables A.1-A.3 in the appendix show a summary of the bending moments calculated in the concrete slab and in steel girders for a one-span 2-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table 4.1 (same as Table A.1 in the appendix) shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 18 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 47 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 52 %.

Since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur in either one of the two left side girders, except for the shortest span, where in the case of its 6 ft girder spacing, the maximum was always at the center girder. When the sidewalks and/or railings were placed on the

Table 4.1. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (2 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Girder 5 (7)**	**Total (8)**
No SR	Girder	514.8	503.3	472.9	405.4	311.5	2208.0
	Slab	31.9	32.6	23.8	16.4	17.0	121.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	546.7	535.9	496.7	421.9	328.5	2329.6
1S(L)	Girder	473.1	477.1	460.9	403.2	317.6	2132.0
	Slab	108.5	32.5	23.9	16.7	16.1	197.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	581.6	509.6	484.8	420.0	333.7	2329.6
1S(R)	Girder	506.0	492.6	459.4	392.1	313.7	2163.9
	Slab	31.4	31.9	22.3	13.9	66.3	165.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	537.4	524.5	481.7	406.0	380.0	2329.6
2S	Girder	465.4	467.6	448.1	389.7	316.9	2087.7
	Slab	106.9	31.9	22.6	14.3	66.2	241.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	572.3	499.5	470.8	404.0	383.1	2329.6
1R(L)	Girder	348.3	407.9	429.0	398.8	336.2	1920.1
	Slab	26.6	32.6	24.6	17.6	16.7	118.0
	Railing	291.5	0.0	0.0	0.0	0.0	291.5
	Total	666.3	440.6	453.6	416.3	352.9	2329.6
1R(R)	Girder	526.6	498.2	447.6	353.9	227.9	2054.1
	Slab	32.7	32.9	23.6	14.9	14.7	118.7
	Railing	0.0	0.0	0.0	0.0	156.7	156.7
	Total	559.3	531.1	471.1	368.8	399.4	2329.6
2R	Girder	353.6	398.1	398.6	339.0	238.0	1727.3
	Slab	27.4	33.1	24.5	16.0	14.2	115.2
	Railing	305.5	0.0	0.0	0.0	181.6	487.1
	Total	686.5	431.3	423.1	355.0	433.8	2329.6
1SR(L)	Girder	342.9	390.7	413.3	387.4	328.3	1862.5
	Slab	78.5	31.2	23.9	17.2	15.0	165.8
	Railing	301.4	0.0	0.0	0.0	0.0	301.4
	Total	722.7	421.9	437.2	404.6	343.3	2329.6
1SR(R)	Girder	512.1	482.2	429.1	338.4	234.7	1996.5
	Slab	31.9	31.8	21.5	11.4	49.4	146.0
	Railing	0.0	0.0	0.0	0.0	187.1	187.1
	Total	544.0	514.0	450.6	349.9	471.2	2329.6
2SR	Girder	338.0	371.4	370.9	316.9	235.8	1633.1
	Slab	78.9	31.1	22.3	13.0	48.7	194.1
	Railing	303.4	0.0	0.0	0.0	199.1	502.5
	Total	720.3	402.6	393.3	329.9	483.6	2329.6

left side or on both sides, the maximum bending moment occurred in the left exterior girder. However, using Tables A.1-A.3 to identify the maximum bending moments at critical sections (usually occurring in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge. The wheel load distribution factors, for the AASHTO (2002) formula and (1), are shown in Table 4.2 for the various span lengths and girder spacings considered in this study.

The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 10 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides. These maximum bending moments and FEA distribution factors are summarized in Table 4.3 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 2-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.4 for all the bridges.

Table 4.2. Distribution Factors for AASHTO Standard Specifications and NCHRP 12-26 (equivalent to AASHTO LRFD)

L (ft)	S (ft)	AASHTO	NCHRP 12-26
40	6	1.09	1.20
	8	1.46	1.48
	12	2.18	1.98
60	6	1.09	1.08
	8	1.46	1.32
	12	2.18	1.77
80	6	1.09	1.01
	8	1.46	1.23
	12	2.18	1.64
100	6	1.09	0.95
	8	1.46	1.16
	12	2.18	1.54
120	6	1.09	0.91
	8	1.46	1.10
	12	2.18	1.47

Table 4.3. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,698	2818	2780	2775	2738	2812	2813	2806	2739	2757	2806
	8	2,698	3658	3523	3622	3489	3532	3660	3534	3303	3616	3262
	12	2,698	5163	5022	5032	4892	5071	5113	5019	4832	4928	4599
60	**6**	4,838	4605	4387	4536	4324	4161	4609	4039	3991	4512	3723
	8	4,838	5805	5539	5686	5430	5153	5736	5061	4800	5562	4576
	12	4,838	7655	7339	7340	7042	7043	7275	6626	6583	6843	5837
80	**6**	6,989	6431	6115	6294	5994	5443	6373	5175	5246	6168	4831
	8	6,989	7983	7614	7763	7416	6711	7728	6386	6378	7411	5885
	12	6,989	10303	9866	9816	9421	8996	9386	8003	8481	8785	7190
100	**6**	9,905	8859	8393	8626	8198	7295	8663	6817	7072	8309	6413
	8	9,905	10918	10387	10570	10086	8885	10350	8217	8539	9859	7673
	12	9,905	14076	13470	13403	12863	11822	12397	10088	11240	11630	9273
120	**6**	13,962	12205	11543	11844	11253	9918	11748	9141	9656	11200	8626
	8	13,962	15011	14259	14500	13824	12106	13927	10842	11704	13223	10206
	12	13,962	19425	18563	18510	17737	15857	16624	13139	15164	15661	12273

(b) Distribution factor = DF = Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,698	1.04	1.03	1.03	1.01	1.04	1.04	1.04	1.02	1.02	1.04
	8	2,698	1.36	1.31	1.34	1.29	1.31	1.36	1.31	1.22	1.34	1.21
	12	2,698	1.91	1.86	1.87	1.81	1.88	1.90	1.86	1.79	1.83	1.70
60	**6**	4,838	0.95	0.91	0.94	0.89	0.86	0.95	0.83	0.82	0.93	0.77
	8	4,838	1.20	1.14	1.18	1.12	1.06	1.19	1.05	0.99	1.15	0.95
	12	4,838	1.58	1.52	1.52	1.46	1.46	1.50	1.37	1.36	1.41	1.21
80	**6**	6,989	0.92	0.87	0.90	0.86	0.78	0.91	0.74	0.75	0.88	0.69
	8	6,989	1.14	1.09	1.11	1.06	0.96	1.11	0.91	0.91	1.06	0.84
	12	6,989	1.47	1.41	1.40	1.35	1.29	1.34	1.15	1.21	1.26	1.03
100	**6**	9,905	0.89	0.85	0.87	0.83	0.74	0.87	0.69	0.71	0.84	0.65
	8	9,905	1.10	1.05	1.07	1.02	0.90	1.04	0.83	0.86	1.00	0.77
	12	9,905	1.42	1.36	1.35	1.30	1.19	1.25	1.02	1.13	1.17	0.94
120	**6**	13,962	0.87	0.83	0.85	0.81	0.71	0.84	0.65	0.69	0.80	0.62
	8	13,962	1.08	1.02	1.04	0.99	0.87	1.00	0.78	0.84	0.95	0.73
	12	13,962	1.39	1.33	1.33	1.27	1.14	1.19	0.94	1.09	1.12	0.88

Table 4.4. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-4	-5	-6	-7	-4	-4	-5	-7	-6	-5
	8	**1.46**	-7	-11	-8	-11	-10	-7	-10	-16	-8	-17
	12	**2.18**	-12	-15	-14	-17	-14	-13	-15	-18	-16	-22
60	**6**	**1.09**	-13	-17	-14	-18	-21	-13	-23	-24	-14	-29
	8	**1.46**	-18	-22	-20	-23	-27	-19	-28	-32	-21	-35
	12	**2.18**	-27	-30	-30	-33	-33	-31	-37	-38	-35	-45
80	**6**	**1.09**	-16	-20	-17	-21	-29	-16	-32	-31	-19	-37
	8	**1.46**	-22	-25	-24	-27	-34	-24	-37	-37	-27	-42
	12	**2.18**	-32	-35	-36	-38	-41	-38	-47	-44	-42	-53
100	**6**	**1.09**	-18	-22	-20	-24	-32	-20	-37	-34	-23	-41
	8	**1.46**	-25	-28	-27	-30	-39	-28	-43	-41	-32	-47
	12	**2.18**	-35	-38	-38	-40	-45	-43	-53	-48	-46	-57
120	**6**	**1.09**	-20	-24	-22	-26	-35	-23	-40	-37	-26	-43
	8	**1.46**	-26	-30	-29	-32	-41	-32	-47	-43	-35	-50
	12	**2.18**	-36	-39	-39	-42	-48	-45	-57	-50	-49	-60

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-13	-14	-15	-16	-13	-13	-14	-16	-15	-14
	8	**1.48**	-8	-12	-9	-12	-11	-8	-11	-17	-9	-18
	12	**1.98**	-4	-6	-6	-9	-5	-5	-6	-10	-8	-14
60	**6**	**1.08**	-12	-16	-13	-17	-21	-12	-23	-24	-14	-29
	8	**1.32**	-9	-14	-11	-15	-20	-10	-21	-25	-13	-29
	12	**1.77**	-11	-15	-15	-18	-18	-15	-23	-23	-20	-32
80	**6**	**1.01**	-9	-13	-10	-15	-23	-9	-26	-25	-12	-31
	8	**1.23**	-7	-11	-10	-14	-22	-10	-26	-26	-14	-31
	12	**1.64**	-10	-14	-14	-18	-22	-18	-30	-26	-23	-37
100	**6**	**0.95**	-6	-11	-8	-13	-23	-8	-28	-25	-12	-32
	8	**1.16**	-5	-9	-8	-12	-23	-10	-28	-26	-14	-33
	12	**1.54**	-8	-12	-12	-16	-23	-19	-34	-26	-24	-39
120	**6**	**0.91**	-4	-9	-7	-11	-22	-7	-28	-24	-12	-32
	8	**1.10**	-3	-7	-6	-10	-21	-10	-30	-24	-14	-34
	12	**1.47**	-5	-10	-10	-14	-23	-19	-36	-26	-24	-40

Figures 4.1-4.3 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings.

A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 2-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.5. It should be noted that Table 4.5 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.6 for all the bridges. Figures 4.4-4.6 show a trend similar to Figures 4.1-4.3, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

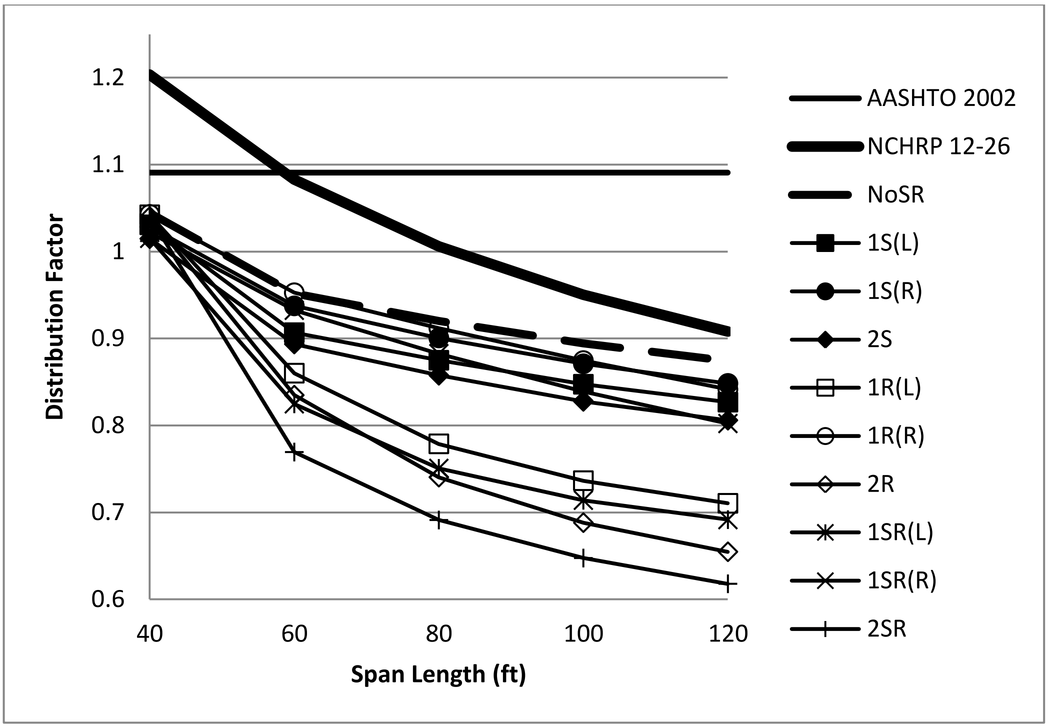


Figure 4.1. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

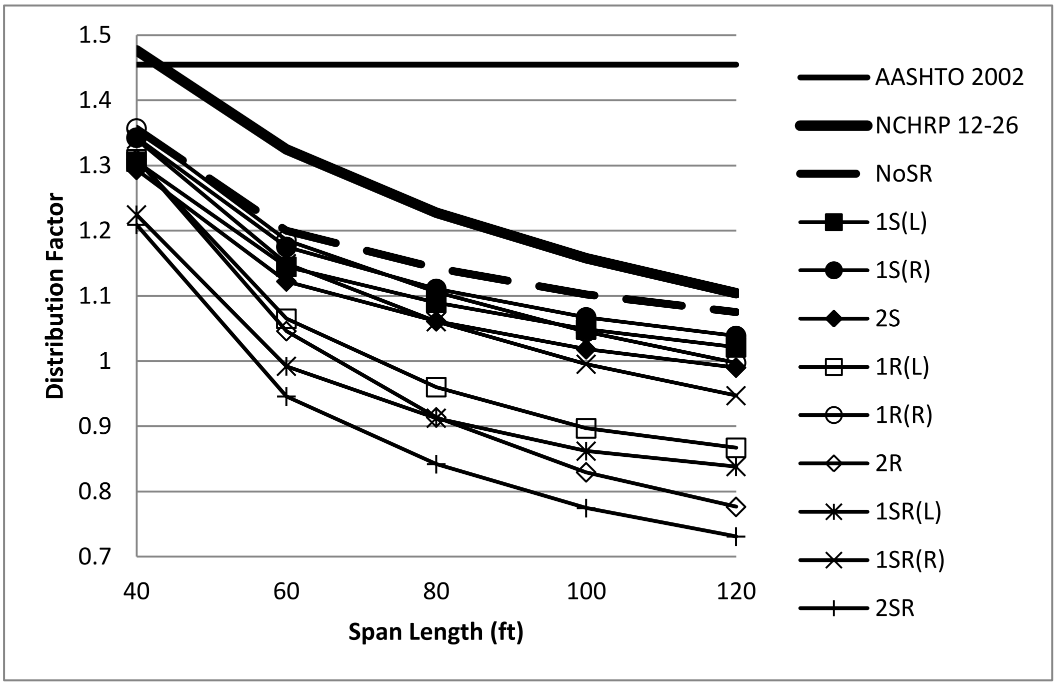


Figure 4.2. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

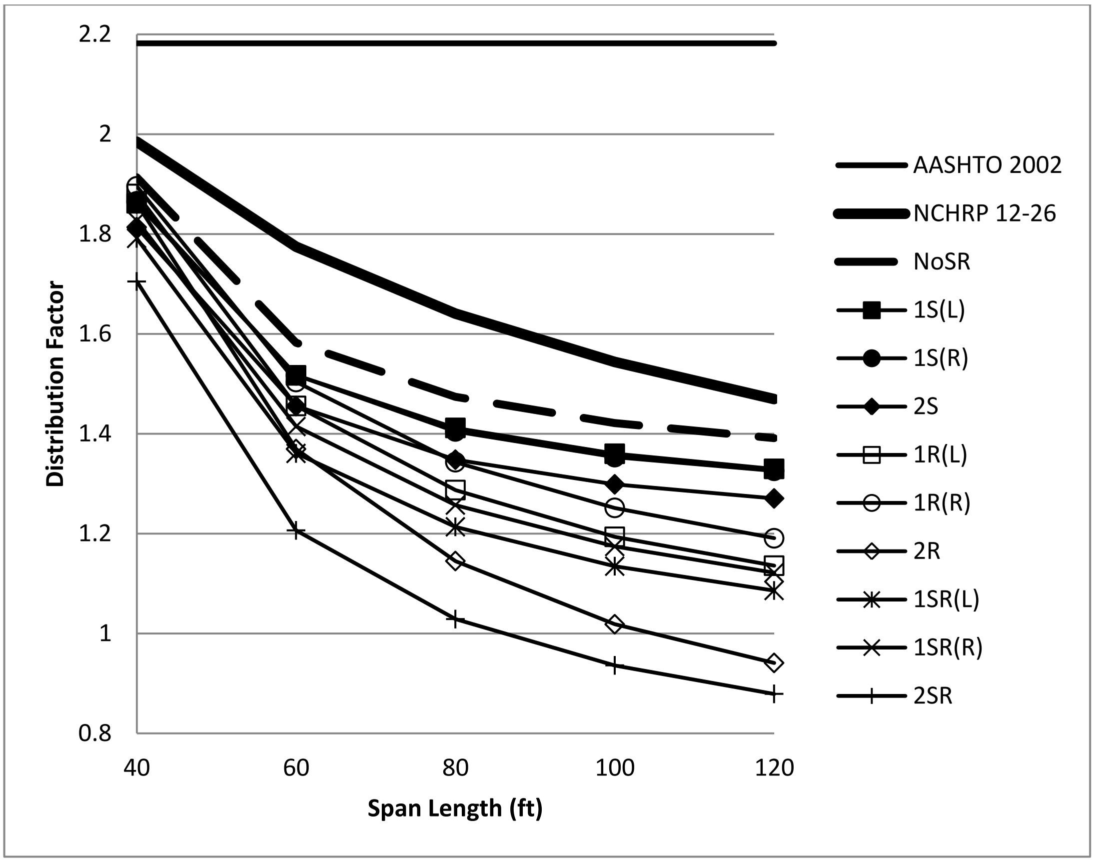


Figure 4.3. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.5. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	2640	2603	2599	2563	2629	2634	2623	2557	2580	2623
	8	2,698	3316	3189	3281	3157	3185	3316	3185	2972	3273	2930
	12	2,698	4450	4322	4333	4206	4354	4400	4302	4135	4231	3917
60	6	4,838	4300	4088	4234	4028	3898	4300	3771	3735	4209	3472
	8	4,838	5335	5083	5223	4980	4683	5260	4585	4363	5097	4140
	12	4,838	6715	6423	6432	6156	6109	6557	5698	5696	6423	4992
80	6	6,989	6178	5725	6072	5612	5148	6319	4783	4960	6146	4457
	8	6,989	7474	7014	7333	6831	6112	7608	5785	5808	7364	5327
	12	6,989	9540	8798	9324	8485	7933	9574	6983	7468	9204	6270
100	6	9,905	8757	8010	8550	7850	6939	8911	6363	6726	8551	5982
	8	9,905	10603	9668	10335	9439	8251	10684	7531	7950	10201	7029
	12	9,905	13553	12270	13157	11937	10573	13360	8922	10048	12672	8205
120	6	13,962	12203	11166	11841	10909	9478	12273	8612	9227	11641	8119
	8	13,962	14800	13493	14336	13142	11385	14665	10042	11005	13840	9445
	12	13,962	18969	17196	18295	16655	14352	18257	11775	13725	17118	11002

(b) Distribution factor = DF = Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	0.98	0.96	0.96	0.95	0.97	0.98	0.97	0.95	0.96	0.97
	8	2,698	1.23	1.18	1.22	1.17	1.18	1.23	1.18	1.10	1.21	1.09
	12	2,698	1.65	1.60	1.61	1.56	1.61	1.63	1.59	1.53	1.57	1.45
60	6	4,838	0.89	0.84	0.88	0.83	0.81	0.89	0.78	0.77	0.87	0.72
	8	4,838	1.10	1.05	1.08	1.03	0.97	1.09	0.95	0.90	1.05	0.86
	12	4,838	1.39	1.33	1.33	1.27	1.26	1.36	1.18	1.18	1.33	1.03
80	6	6,989	0.88	0.82	0.87	0.80	0.74	0.90	0.68	0.71	0.88	0.64
	8	6,989	1.07	1.00	1.05	0.98	0.87	1.09	0.83	0.83	1.05	0.76
	12	6,989	1.37	1.26	1.33	1.21	1.14	1.37	1.00	1.07	1.32	0.90
100	6	9,905	0.88	0.81	0.86	0.79	0.70	0.90	0.64	0.68	0.86	0.60
	8	9,905	1.07	0.98	1.04	0.95	0.83	1.08	0.76	0.80	1.03	0.71
	12	9,905	1.37	1.24	1.33	1.21	1.07	1.35	0.90	1.01	1.28	0.83
120	6	13,962	0.87	0.80	0.85	0.78	0.68	0.88	0.62	0.66	0.83	0.58
	8	13,962	1.06	0.97	1.03	0.94	0.82	1.05	0.72	0.79	0.99	0.68
	12	13,962	1.36	1.23	1.31	1.19	1.03	1.31	0.84	0.98	1.23	0.79

Table 4.6. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-10	-11	-12	-13	-11	-10	-11	-13	-12	-11
	8	**1.46**	-16	-19	-17	-20	-19	-16	-19	-25	-17	-26
	12	**2.18**	-24	-27	-26	-28	-26	-25	-27	-30	-28	-33
60	**6**	**1.09**	-18	-22	-20	-24	-26	-18	-28	-29	-20	-34
	8	**1.46**	-24	-28	-26	-29	-34	-26	-35	-38	-28	-41
	12	**2.18**	-36	-39	-39	-42	-42	-38	-46	-46	-39	-53
80	**6**	**1.09**	-19	-25	-20	-26	-32	-17	-37	-35	-19	-41
	8	**1.46**	-27	-31	-28	-33	-40	-25	-43	-43	-28	-48
	12	**2.18**	-37	-42	-39	-44	-48	-37	-54	-51	-40	-59
100	**6**	**1.09**	-19	-26	-21	-27	-36	-17	-41	-38	-21	-45
	8	**1.46**	-27	-33	-29	-35	-43	-26	-48	-45	-29	-51
	12	**2.18**	-37	-43	-39	-45	-51	-38	-59	-53	-41	-62
120	**6**	**1.09**	-20	-27	-22	-28	-38	-19	-43	-39	-24	-47
	8	**1.46**	-27	-34	-30	-36	-44	-28	-51	-46	-32	-54
	12	**2.18**	-38	-44	-40	-45	-53	-40	-61	-55	-44	-64

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-19	-20	-20	-21	-19	-19	-19	-21	-21	-19
	8	**1.48**	-17	-20	-18	-21	-20	-17	-20	-25	-18	-26
	12	**1.98**	-17	-19	-19	-21	-19	-18	-20	-23	-21	-27
60	**6**	**1.08**	-18	-22	-19	-23	-26	-18	-28	-29	-20	-34
	8	**1.32**	-17	-21	-18	-22	-27	-18	-28	-32	-20	-35
	12	**1.77**	-22	-25	-25	-28	-29	-24	-34	-34	-25	-42
80	**6**	**1.01**	-12	-19	-14	-20	-27	-10	-32	-29	-13	-37
	8	**1.23**	-13	-18	-15	-20	-29	-11	-33	-32	-14	-38
	12	**1.64**	-17	-23	-19	-26	-31	-16	-39	-35	-20	-45
100	**6**	**0.95**	-7	-15	-9	-17	-26	-5	-32	-29	-9	-36
	8	**1.16**	-8	-16	-10	-18	-28	-7	-34	-31	-11	-39
	12	**1.54**	-11	-20	-14	-22	-31	-13	-42	-34	-17	-46
120	**6**	**0.91**	-4	-12	-7	-14	-25	-3	-32	-27	-8	-36
	8	**1.10**	-4	-12	-7	-15	-26	-5	-35	-29	-10	-39
	12	**1.47**	-8	-16	-11	-19	-30	-11	-43	-33	-17	-46

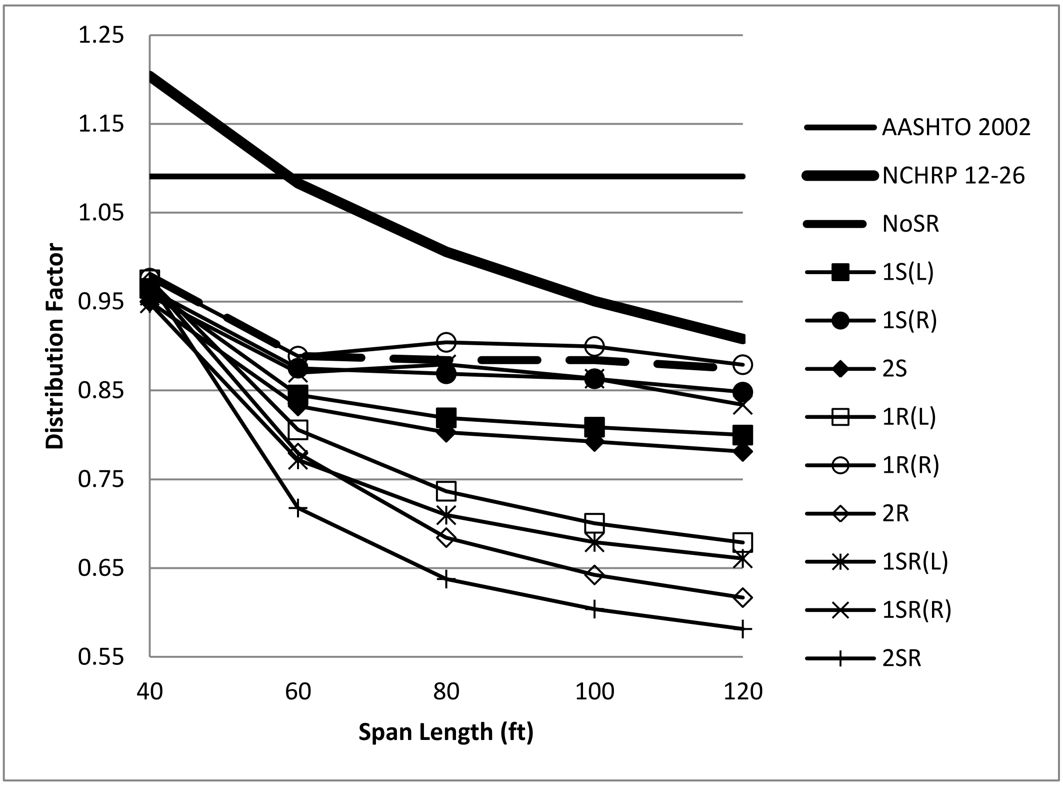


Figure 4.4. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6 ft)

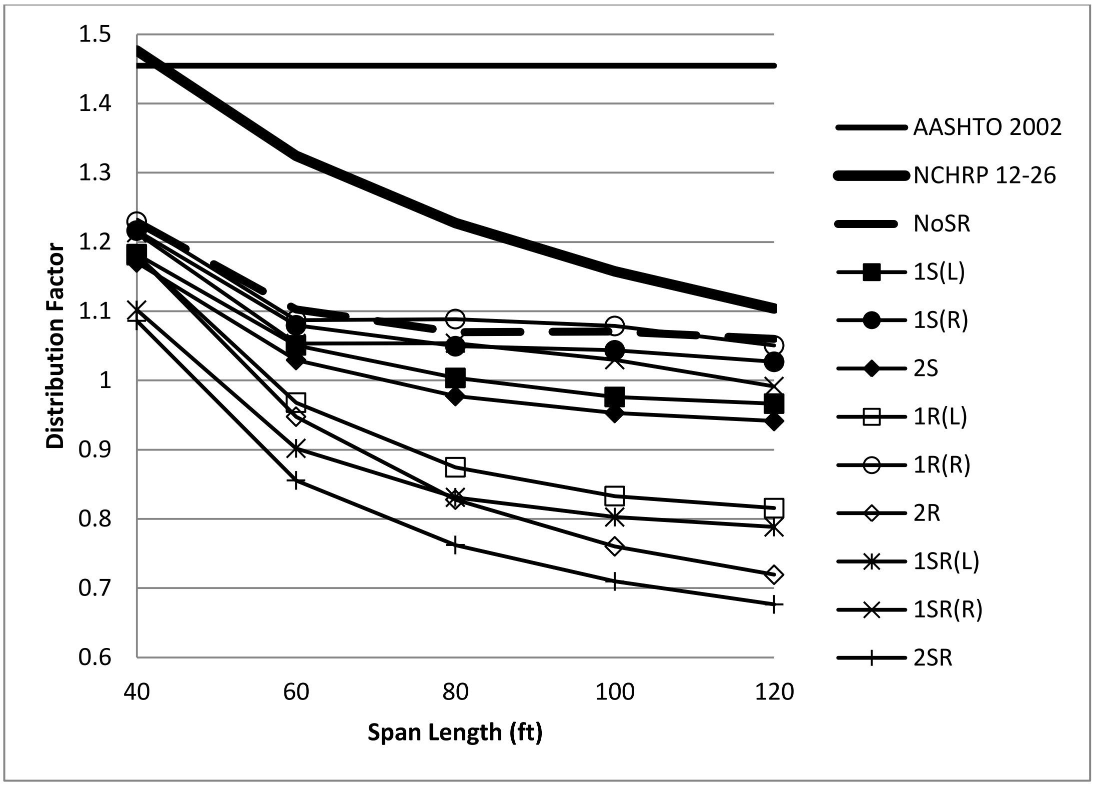


Figure 4.5. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8 ft)

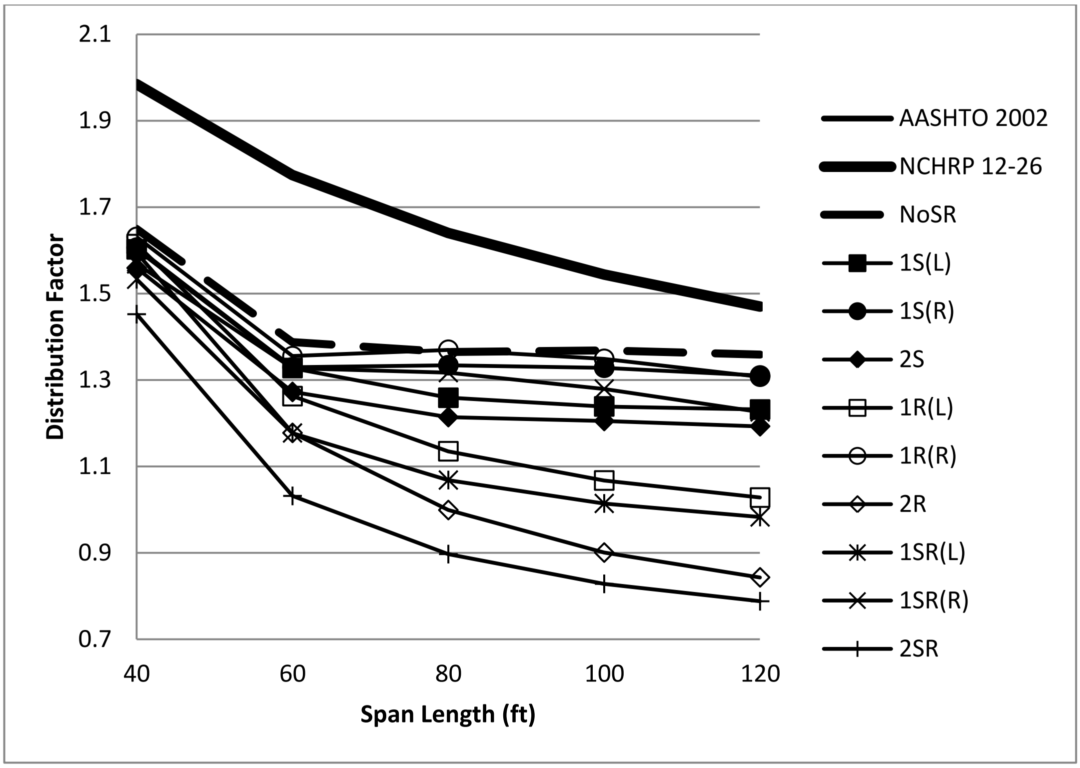


Figure 4.6. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12 ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.6 (considering steel beams only) as compared to Table 4.4 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.4 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: the FEA distribution factors are smaller than (1) by about 10 % for spans up to 80 ft and by about 5 % for spans between 80 and 120 ft.
2. Sidewalk on one side (left or right): (1) is about 12 % higher than FEA distribution factors for spans up to 80 ft and about 9 % higher for spans between 80 and 120 ft.
3. Sidewalk on both sides: (1) is about 15 % higher than FEA distribution factors for spans up to 80 ft and about 13 % higher for spans between 80 and 120 ft.
4. Railing on one side (left or right): (1) is about 10 % higher than FEA distribution factors.
5. Railing on both sides: (1) is about 20 % higher than FEA distribution factors.
6. Sidewalk and railing on one side (left or right): (1) is about 15 % higher than FEA distribution factors.
7. Sidewalk and railing on both sides: (1) is about 30 % higher than FEA distribution factors.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it's important to note that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a 10 % reduction in distribution factor is reached. Otherwise, the average reductions in FEA distribution factors when compared to the base case were 3 % when introducing sidewalks on one side; 7 % when introducing sidewalks on both sides; 2 % for spans between 40 and 120 ft (12 and 36 m) with girder spacing up to 8 ft and 5 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing between 8 and 12 ft and 12 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 8 and 12 ft when introducing railings on one side; 12 % for spans between 40 and 60 ft (12 and 18 m) and 20 % for spans between 60 and 80 ft (18 m and 24 m) and 25 % for spans between 80 and 120 ft (24 and 36 m) when introducing railings on both sides; 3 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing up to 8ft and 11 % for spans between 40 and 80 ft with girder spacing between 8 and 12 ft and 8 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 8 ft and 17 % for spans between 80 and 120 ft with girder spacing between 8 and 12 ft for a combination of sidewalks and railings on one side; and 19 % for spans between 40 and 60 ft (12 and 18 m) and 25 % for spans between 60 and 80 ft (18 and 24 m) and 30 % for spans between 80 and 120 ft (24 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.2.2. Three-Lane Bridges

Tables A.4-A.6 in the appendix show a summary of the bending moments calculated in the concrete slab and in steel girders for a one-span 3-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.4 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 18 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 46 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 52 %.

Since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur in either one of the two left side girders. When the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment occurred in the left exterior girder. However, using Tables A.4-A.6 to identify the maximum bending moments at critical sections (usually occurring in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge. It is worth mentioning that these maximum wheel load distributions have been reduced by 10 %, as permitted by the AASHTO Standard Specifications (2002) for 3-lane bridges, in order to account for the improbable situation of having all lanes loaded at the same time and at locations along the bridge deck producing the maximum bending moment in a bridge superstructure. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 12 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides. These maximum bending moments and FEA distribution factors are summarized in Table 4.7 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 3-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.8 for all the bridges.

Table 4.7. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,698	3226	3216	3207	3197	3250	3225	3248	3229	3209	3212
	8	2,698	4225	4146	4201	4122	4220	4228	4223	4129	4206	4110
	12	2,698	5907	5889	5763	5746	5917	5841	5851	5897	5633	5618
60	**6**	4,838	5270	5149	5222	5103	5109	5278	5093	4998	5226	4887
	8	4,838	6615	6417	6493	6302	6471	6608	6463	6301	6465	6070
	12	4,838	9203	9087	8823	8716	9175	8751	8648	9008	8574	7969
80	**6**	6,989	7203	6951	7137	6861	6572	7252	6510	6409	7168	6166
	8	6,989	8571	8518	8388	8269	8094	8454	7959	8151	8213	7464
	12	6,989	12080	11834	11539	11312	11811	11382	10451	11459	11049	9563
100	**6**	9,905	10057	9581	9927	9462	8571	10122	8411	8374	9937	8010
	8	9,905	11449	11532	11201	11217	10574	11092	10033	10796	10762	9628
	12	9,905	16295	15917	15564	15219	15597	14987	13085	15067	14506	12121
120	**6**	13,962	13933	13226	13699	13025	11402	13970	11078	11165	13622	10575
	8	13,962	15149	15408	14827	15021	13771	14782	12634	14165	14366	12395
	12	13,962	22246	21683	21255	20746	20834	20005	16823	20072	19342	15783

(b) Distribution factor = 0.9*DF = 0.9*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,698	1.08	1.07	1.07	1.07	1.08	1.08	1.08	1.08	1.07	1.07
	8	2,698	1.41	1.38	1.40	1.38	1.41	1.41	1.41	1.38	1.40	1.37
	12	2,698	1.97	1.96	1.92	1.92	1.97	1.95	1.95	1.97	1.88	1.87
60	**6**	4,838	0.98	0.96	0.97	0.95	0.95	0.98	0.95	0.93	0.97	0.91
	8	4,838	1.23	1.19	1.21	1.17	1.20	1.23	1.20	1.17	1.20	1.13
	12	4,838	1.71	1.69	1.64	1.62	1.71	1.63	1.61	1.68	1.59	1.48
80	**6**	6,989	0.93	0.90	0.92	0.88	0.85	0.93	0.84	0.83	0.92	0.79
	8	6,989	1.10	1.10	1.08	1.06	1.04	1.09	1.02	1.05	1.06	0.96
	12	6,989	1.56	1.52	1.49	1.46	1.52	1.47	1.35	1.48	1.42	1.23
100	**6**	9,905	0.91	0.87	0.90	0.86	0.78	0.92	0.76	0.76	0.90	0.73
	8	9,905	1.04	1.05	1.02	1.02	0.96	1.01	0.91	0.98	0.98	0.87
	12	9,905	1.48	1.45	1.41	1.38	1.42	1.36	1.19	1.37	1.32	1.10
120	**6**	13,962	0.90	0.85	0.88	0.84	0.73	0.90	0.71	0.72	0.88	0.68
	8	13,962	0.98	0.99	0.96	0.97	0.89	0.95	0.81	0.91	0.93	0.80
	12	13,962	1.43	1.40	1.37	1.34	1.34	1.29	1.08	1.29	1.25	1.02

Table 4.8. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-1	-2	-2	-2	-1	-1	-1	-1	-2	-2
	8	**1.46**	-3	-5	-4	-6	-4	-3	-4	-6	-4	-6
	12	**2.18**	-10	-10	-12	-12	-9	-11	-10	-10	-14	-14
60	**6**	**1.09**	-10	-12	-11	-13	-13	-10	-13	-15	-11	-17
	8	**1.46**	-16	-18	-17	-20	-18	-16	-18	-20	-18	-23
	12	**2.18**	-21	-22	-25	-26	-22	-25	-26	-23	-27	-32
80	**6**	**1.09**	-15	-18	-16	-19	-22	-14	-23	-24	-15	-27
	8	**1.46**	-24	-25	-26	-27	-29	-25	-30	-28	-28	-34
	12	**2.18**	-29	-30	-32	-33	-30	-33	-38	-32	-35	-44
100	**6**	**1.09**	-16	-20	-17	-21	-29	-16	-30	-30	-17	-33
	8	**1.46**	-29	-28	-30	-30	-34	-31	-38	-33	-33	-40
	12	**2.18**	-32	-34	-35	-37	-35	-38	-45	-37	-40	-49
120	**6**	**1.09**	-18	-22	-19	-23	-33	-17	-34	-34	-19	-37
	8	**1.46**	-33	-32	-35	-34	-39	-35	-44	-37	-37	-45
	12	**2.18**	-34	-36	-37	-39	-38	-41	-50	-41	-43	-53

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-11	-11	-11	-11	-10	-11	-10	-11	-11	-11
	8	**1.48**	-5	-6	-5	-7	-5	-4	-5	-7	-5	-7
	12	**1.98**	-1	-1	-3	-3	-1	-2	-2	-1	-5	-6
60	**6**	**1.08**	-9	-12	-10	-12	-12	-9	-13	-14	-10	-16
	8	**1.32**	-7	-10	-9	-12	-9	-7	-9	-12	-9	-15
	12	**1.77**	-4	-5	-8	-9	-4	-8	-9	-6	-10	-16
80	**6**	**1.01**	-8	-11	-9	-12	-16	-7	-17	-18	-8	-21
	8	**1.23**	-10	-11	-12	-13	-15	-11	-16	-14	-14	-22
	12	**1.64**	-5	-7	-9	-11	-7	-11	-18	-10	-13	-25
100	**6**	**0.95**	-4	-8	-5	-10	-18	-3	-20	-20	-5	-23
	8	**1.16**	-10	-9	-12	-12	-17	-13	-21	-15	-16	-24
	12	**1.54**	-4	-6	-8	-10	-8	-12	-23	-11	-15	-29
120	**6**	**0.91**	-1	-6	-3	-8	-19	-1	-21	-21	-3	-25
	8	**1.10**	-12	-10	-13	-12	-20	-14	-26	-17	-16	-28
	12	**1.47**	-2	-5	-7	-9	-9	-12	-26	-12	-15	-31

Figures 4.7-4.9 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 3-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.9. It should be noted that Table 4.9 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.10 for all the bridges. Figures 4.10-4.12 show a trend similar to Figures 4.7-4.9, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

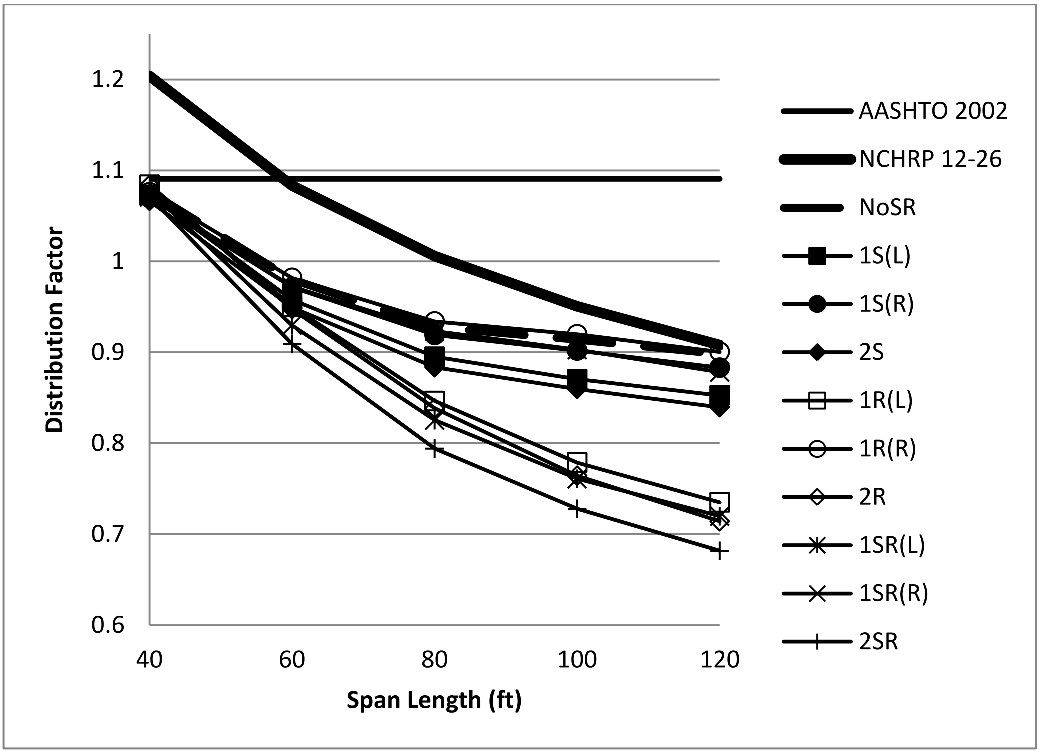


Figure 4.7. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

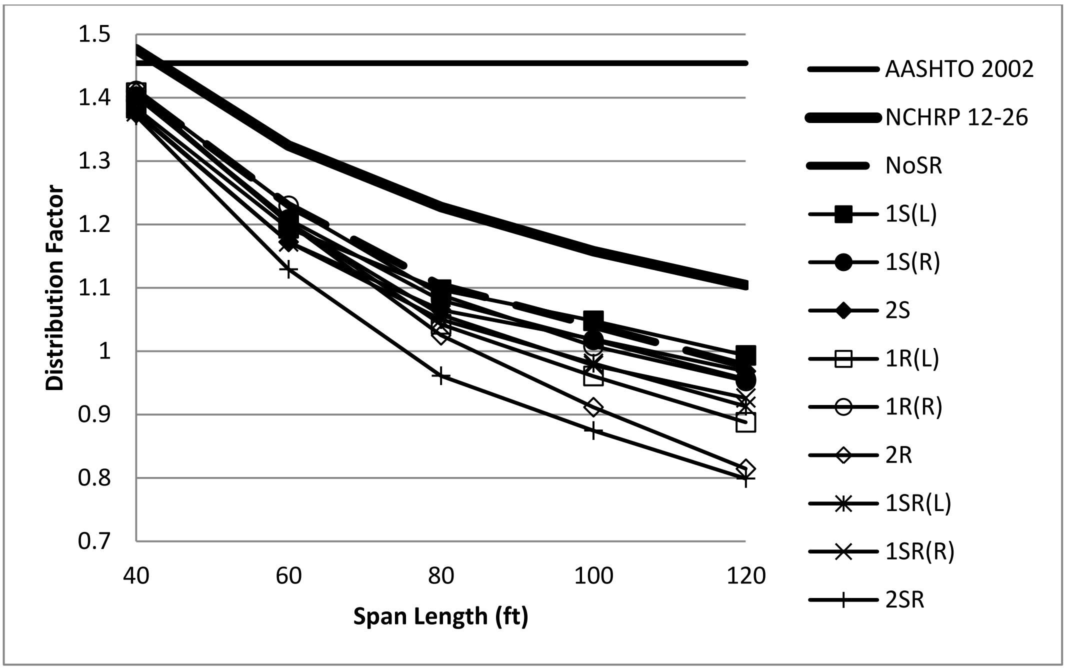


Figure 4.8. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

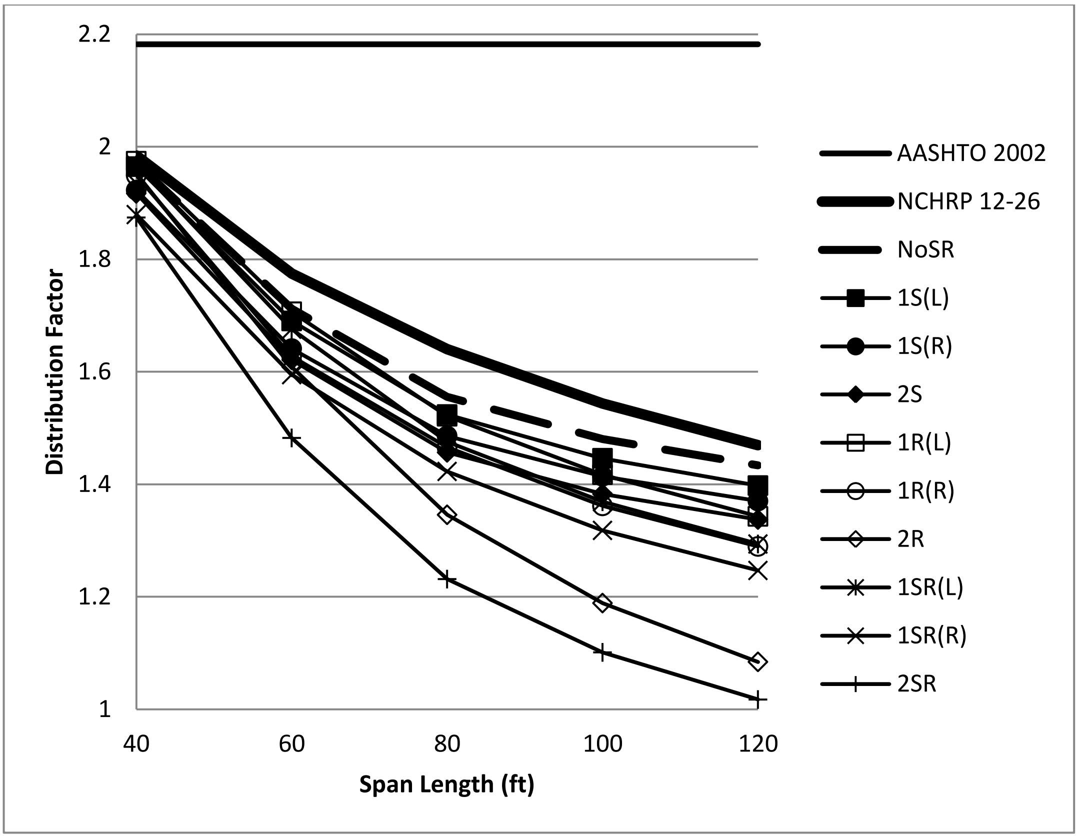


Figure 4.9. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.9. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	3018	2985	3013	2973	3017	3018	3015	2995	3015	2978
	8	2,698	3862	3788	3838	3764	3855	3865	3858	3768	3843	3749
	12	2,698	5155	5137	5026	5009	5164	5087	5096	5143	4902	4880
60	6	4,838	4979	4860	4933	4816	4763	4985	4761	4655	4934	4545
	8	4,838	6072	5889	5956	5779	5920	6060	5907	5721	5923	5540
	12	4,838	8129	8020	7785	7684	8090	7699	7566	7933	7533	6950
80	6	6,989	6775	6589	6709	6505	6160	6816	6125	5977	6736	5800
	8	6,989	7902	7665	7733	7507	7488	7776	7271	7277	7555	6795
	12	6,989	10887	10677	10391	10196	10628	10080	9294	10327	9780	8518
100	6	9,905	9666	9062	9560	8948	8100	9870	7944	7868	9715	7566
	8	9,905	10634	10327	10405	10112	9875	10270	9206	9607	9969	8679
	12	9,905	14825	14489	14140	13830	14158	13431	11710	13693	12997	10866
120	6	13,962	13767	12746	13551	12567	10842	14075	10473	10591	13733	10025
	8	13,962	14172	13772	13871	13491	12924	13806	11669	12591	13416	11116
	12	13,962	20395	19882	19453	18982	20011	18116	15170	19380	17515	14250

(b) Distribution factor = 0.9*DF = 0.9*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	1.01	1.00	1.01	0.99	1.01	1.01	1.01	1.00	1.01	0.99
	8	2,698	1.29	1.26	1.28	1.26	1.29	1.29	1.29	1.26	1.28	1.25
	12	2,698	1.72	1.71	1.68	1.67	1.72	1.70	1.70	1.72	1.64	1.63
60	6	4,838	0.93	0.90	0.92	0.90	0.89	0.93	0.89	0.87	0.92	0.85
	8	4,838	1.13	1.10	1.11	1.08	1.10	1.13	1.10	1.06	1.10	1.03
	12	4,838	1.51	1.49	1.45	1.43	1.50	1.43	1.41	1.48	1.40	1.29
80	6	6,989	0.87	0.85	0.86	0.84	0.79	0.88	0.79	0.77	0.87	0.75
	8	6,989	1.02	0.99	1.00	0.97	0.96	1.00	0.94	0.94	0.97	0.88
	12	6,989	1.40	1.37	1.34	1.31	1.37	1.30	1.20	1.33	1.26	1.10
100	6	9,905	0.88	0.82	0.87	0.81	0.74	0.90	0.72	0.71	0.88	0.69
	8	9,905	0.97	0.94	0.95	0.92	0.90	0.93	0.84	0.87	0.91	0.79
	12	9,905	1.35	1.32	1.28	1.26	1.29	1.22	1.06	1.24	1.18	0.99
120	6	13,962	0.89	0.82	0.87	0.81	0.70	0.91	0.68	0.68	0.89	0.65
	8	13,962	0.91	0.89	0.89	0.87	0.83	0.89	0.75	0.81	0.86	0.72
	12	13,962	1.31	1.28	1.25	1.22	1.29	1.17	0.98	1.25	1.13	0.92

Table 4.10. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-8	-9	-8	-9	-8	-8	-8	-8	-8	-9
	8	**1.46**	-12	-13	-12	-14	-12	-12	-12	-14	-12	-14
	12	**2.18**	-21	-21	-23	-23	-21	-22	-22	-21	-25	-25
60	**6**	**1.09**	-15	-17	-16	-18	-19	-15	-19	-21	-16	-22
	8	**1.46**	-23	-25	-24	-26	-25	-23	-25	-27	-25	-29
	12	**2.18**	-31	-32	-34	-34	-31	-34	-35	-32	-36	-41
80	**6**	**1.09**	-20	-22	-21	-23	-27	-19	-28	-29	-20	-31
	8	**1.46**	-30	-32	-32	-34	-34	-31	-36	-36	-33	-40
	12	**2.18**	-36	-37	-39	-40	-37	-40	-45	-39	-42	-50
100	**6**	**1.09**	-19	-24	-20	-25	-32	-18	-34	-34	-19	-37
	8	**1.46**	-34	-36	-35	-37	-39	-36	-43	-40	-38	-46
	12	**2.18**	-38	-40	-41	-42	-41	-44	-51	-43	-46	-55
120	**6**	**1.09**	-19	-25	-20	-26	-36	-17	-38	-37	-19	-41
	8	**1.46**	-37	-39	-39	-40	-43	-39	-48	-44	-41	-51
	12	**2.18**	-40	-41	-42	-44	-41	-46	-55	-43	-48	-58

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-16	-17	-17	-18	-16	-16	-16	-17	-16	-17
	8	**1.48**	-13	-14	-13	-15	-13	-13	-13	-15	-13	-15
	12	**1.98**	-13	-14	-16	-16	-13	-14	-14	-14	-18	-18
60	**6**	**1.08**	-14	-17	-15	-17	-18	-14	-18	-20	-15	-22
	8	**1.32**	-15	-17	-16	-19	-17	-15	-17	-20	-17	-22
	12	**1.77**	-15	-16	-18	-19	-15	-19	-21	-17	-21	-27
80	**6**	**1.01**	-13	-16	-14	-17	-21	-13	-22	-23	-14	-26
	8	**1.23**	-17	-20	-19	-21	-21	-18	-24	-24	-21	-29
	12	**1.64**	-15	-16	-18	-20	-17	-21	-27	-19	-23	-33
100	**6**	**0.95**	-8	-13	-9	-14	-23	-6	-24	-25	-7	-28
	8	**1.16**	-17	-19	-18	-21	-22	-19	-28	-25	-22	-32
	12	**1.54**	-13	-15	-17	-19	-17	-21	-31	-19	-24	-36
120	**6**	**0.91**	-2	-9	-4	-11	-23	0	-26	-25	-2	-29
	8	**1.10**	-17	-20	-19	-21	-25	-19	-32	-26	-22	-35
	12	**1.47**	-11	-13	-15	-17	-12	-21	-33	-15	-23	-37

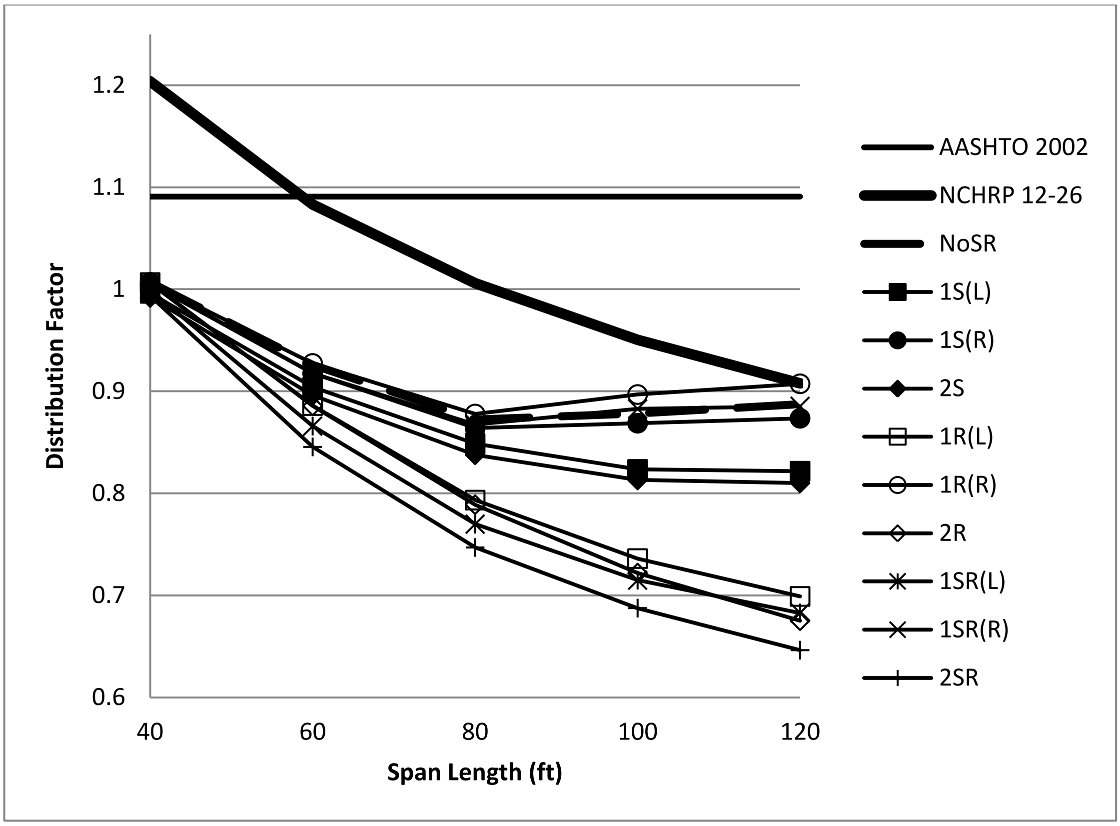


Figure 4.10. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6 ft)

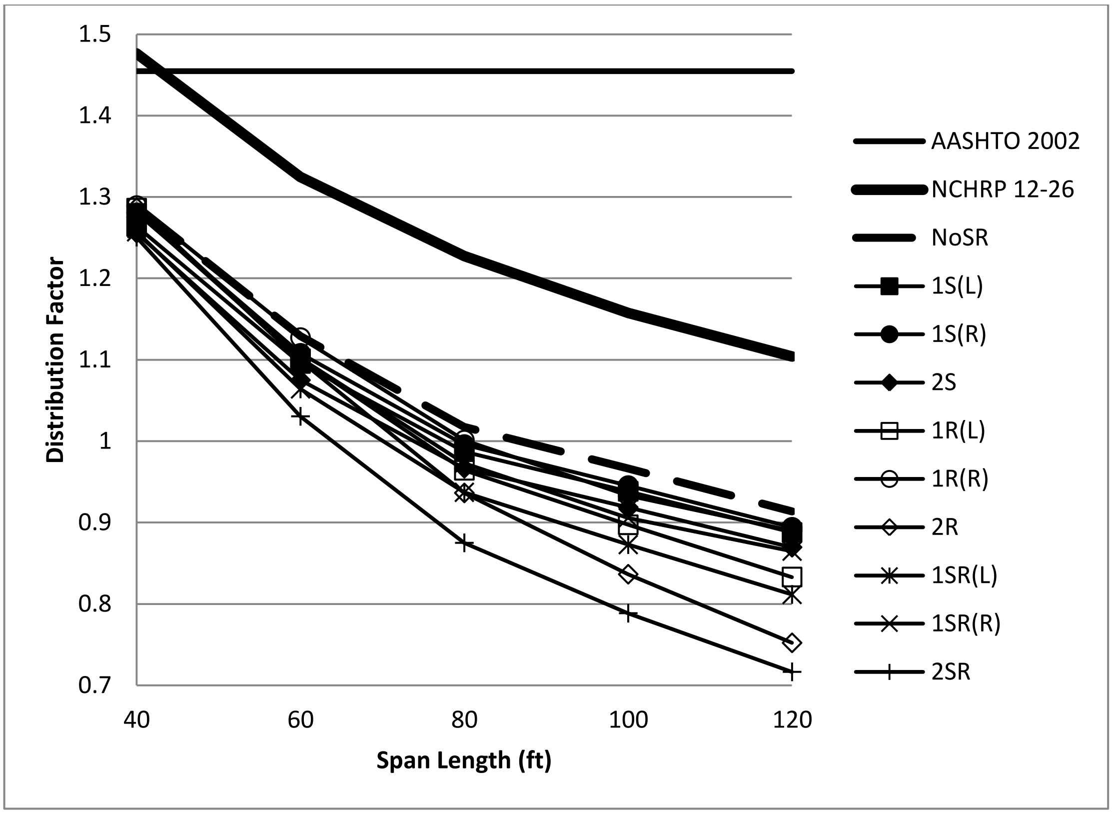


Figure 4.11. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8 ft)

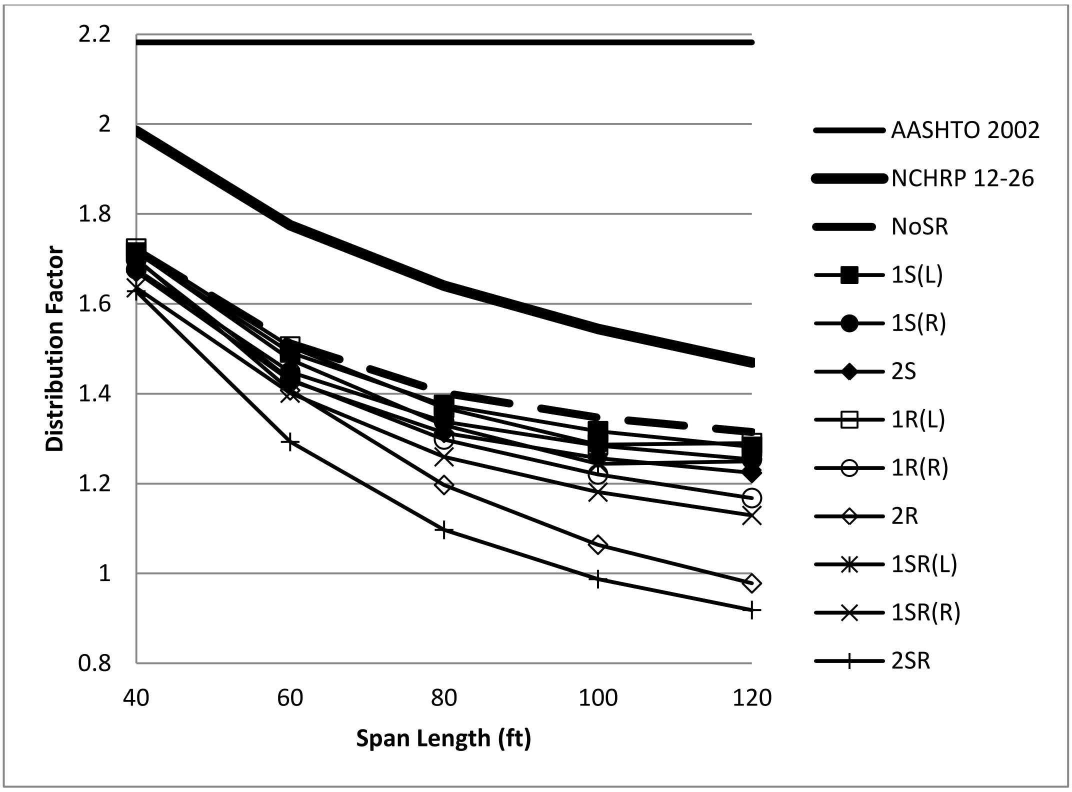


Figure 4.12. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12 ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.10 (considering steel beams only) as compared to Table 4.8 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.8 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: the FEA distribution factors are smaller than (1) by about 6 % for all spans.
2. Sidewalk on one side (left or right): (1) is about 8 % higher than FEA distribution factors for all spans.
3. Sidewalk on both sides: (1) is about 9 % higher than FEA distribution factors for spans up to 80 ft and about 11 % higher for spans between 80 and 120 ft.
4. Railing on one side (left or right): (1) is about 7 % higher than FEA distribution factors for spans up to 80 ft and about 11 % higher for spans between 80 and 120 ft.
5. Railing on both sides: (1) is about 16 % higher than FEA distribution factors.
6. Sidewalk and railing on one side (left or right): (1) is about 8 % higher than FEA distribution factors for spans up to 80 ft and about 14 % higher for spans between 80 and 120 ft.
7. Sidewalk and railing on both sides: (1) is about 12 % higher than FEA distribution factors for spans up to 80 ft and about 25 % higher for spans between 80 and 120 ft.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it is important to note that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a 3 % reduction in distribution factor is observed. Otherwise, the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 5 % when introducing sidewalks on both sides; 0 % for spans between 40 and 80 ft (12 and 24 m) and 0 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 6 ft and 3 % for spans between 80 and 120 ft with girder spacing between 6 and 12 ft when introducing railings on one side; 3 % for spans between 40 and 60 ft (12 and 18 m) and 8 % for spans between 60 and 80 ft (18 and 24 m) and 16 % for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on both sides; 2 % for spans between 40 and 80 ft (12 and 24 m) and 2 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 6ft and 6 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 6 and 12 ft for a combination of sidewalks and railings on one side; and 8 % for spans between 40 and 60 ft (12 and 18 m) and 13 % for spans between 60 and 80 ft (18 and 24 m) and 20 % for spans between 80 and 120 ft (24 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge.

However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.2.3. Four-Lane Bridges

Tables A.7-A.9 in the appendix show a summary of the bending moments calculated in the concrete slab and in steel girders for a one-span 4-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.7 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 18 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 46 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 52 %.

Since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur in either one of the two left side girders. When the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment occurred in the left exterior girder. However, using Tables A.7-A.9 to identify the maximum bending moments at critical sections (usually occurring in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge. It is worth mentioning that these maximum wheel load distributions have been reduced by 25 %, as permitted by the AASHTO Standard Specifications (2002) for 4-lane bridges, in order to account for the improbable situation of having all lanes loaded at the same time and at locations along the bridge deck producing the maximum bending moment in a bridge superstructure. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 12 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides. These maximum bending moments and FEA distribution factors are summarized in Table 4.11 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 4-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.12 for all the bridges.

Table 4.11. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	3362	3353	3362	3353	3385	3362	3385	3365	3363	3365
	8	2,698	4364	4354	4364	4344	4378	4363	4376	4373	4365	4366
	12	2,698	6458	6446	6440	6429	6486	6458	6486	6463	6443	6448
60	6	4,838	5733	5674	5707	5648	5708	5737	5711	5594	5716	5577
	8	4,838	7353	7220	7321	7189	7255	7361	7253	7155	7333	7076
	12	4,838	10479	10366	10363	10252	10438	10454	10414	10244	10313	10077
80	6	6,989	7783	7670	7709	7598	7536	7789	7543	7354	7705	7278
	8	6,989	9980	9751	9887	9661	9416	9989	9422	9104	9876	8993
	12	6,989	14045	13443	13921	13327	13135	14061	12915	12740	13902	12218
100	6	9,905	10608	10304	10540	10212	9796	10691	9751	9567	10610	9379
	8	9,905	13639	13092	13527	12938	12197	13721	12115	11817	13573	11551
	12	9,905	19038	18204	18803	17987	16746	18960	16063	16231	18635	15127
120	6	13,962	14950	14283	14807	14148	12841	15100	12657	12581	14907	12194
	8	13,962	18946	18085	18733	17889	15978	19043	15684	15563	18738	15008
	12	13,962	25907	24744	25502	24375	21708	25577	20716	21082	25009	19605

(b) Distribution factor = 0.75*DF = 0.75*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	0.93	0.93	0.93	0.93	0.94	0.93	0.94	0.94	0.94	0.94
	8	2,698	1.21	1.21	1.21	1.21	1.22	1.21	1.22	1.22	1.21	1.21
	12	2,698	1.80	1.79	1.79	1.79	1.80	1.80	1.80	1.80	1.79	1.79
60	6	4,838	0.89	0.88	0.88	0.88	0.88	0.89	0.89	0.87	0.89	0.86
	8	4,838	1.14	1.12	1.13	1.11	1.12	1.14	1.12	1.11	1.14	1.10
	12	4,838	1.62	1.61	1.61	1.59	1.62	1.62	1.61	1.59	1.60	1.56
80	6	6,989	0.84	0.82	0.83	0.82	0.81	0.84	0.81	0.79	0.83	0.78
	8	6,989	1.07	1.05	1.06	1.04	1.01	1.07	1.01	0.98	1.06	0.97
	12	6,989	1.51	1.44	1.49	1.43	1.41	1.51	1.39	1.37	1.49	1.31
100	6	9,905	0.80	0.78	0.80	0.77	0.74	0.81	0.74	0.72	0.80	0.71
	8	9,905	1.03	0.99	1.02	0.98	0.92	1.04	0.92	0.89	1.03	0.87
	12	9,905	1.44	1.38	1.42	1.36	1.27	1.44	1.22	1.23	1.41	1.15
120	6	13,962	0.80	0.77	0.80	0.76	0.69	0.81	0.68	0.68	0.80	0.66
	8	13,962	1.02	0.97	1.01	0.96	0.86	1.02	0.84	0.84	1.01	0.81
	12	13,962	1.39	1.33	1.37	1.31	1.17	1.37	1.11	1.13	1.34	1.05

Table 4.12. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-14	-14	-14	-14	-14	-14	-14	-14	-14	-14
	8	**1.46**	-17	-17	-17	-17	-17	-17	-17	-17	-17	-17
	12	**2.18**	-18	-18	-18	-18	-17	-18	-17	-18	-18	-18
60	**6**	**1.09**	-18	-19	-19	-20	-19	-18	-19	-20	-19	-21
	8	**1.46**	-22	-23	-22	-24	-23	-22	-23	-24	-22	-25
	12	**2.18**	-25	-26	-26	-27	-26	-26	-26	-27	-27	-28
80	**6**	**1.09**	-23	-24	-24	-25	-26	-23	-26	-28	-24	-28
	8	**1.46**	-27	-28	-27	-29	-31	-27	-31	-33	-27	-34
	12	**2.18**	-31	-34	-31	-34	-35	-31	-36	-37	-32	-40
100	**6**	**1.09**	-26	-28	-27	-29	-32	-26	-32	-34	-26	-35
	8	**1.46**	-29	-32	-30	-33	-37	-29	-37	-39	-30	-40
	12	**2.18**	-34	-37	-35	-38	-42	-34	-44	-44	-35	-47
120	**6**	**1.09**	-26	-30	-27	-30	-37	-26	-38	-38	-27	-40
	8	**1.46**	-30	-33	-31	-34	-41	-30	-42	-43	-31	-45
	12	**2.18**	-36	-39	-37	-40	-47	-37	-49	-48	-38	-52

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-22	-23	-22	-23	-22	-22	-22	-22	-22	-22
	8	**1.48**	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18
	12	**1.98**	-10	-10	-10	-10	-9	-10	-9	-9	-10	-10
60	**6**	**1.08**	-18	-19	-18	-19	-18	-18	-18	-20	-18	-20
	8	**1.32**	-14	-15	-14	-16	-15	-14	-15	-16	-14	-17
	12	**1.77**	-8	-9	-9	-10	-9	-9	-9	-11	-10	-12
80	**6**	**1.01**	-17	-18	-18	-19	-20	-17	-20	-22	-18	-22
	8	**1.23**	-13	-15	-14	-16	-18	-13	-18	-20	-14	-21
	12	**1.64**	-8	-12	-9	-13	-14	-8	-16	-17	-9	-20
100	**6**	**0.95**	-15	-18	-16	-19	-22	-15	-22	-24	-15	-25
	8	**1.16**	-11	-14	-12	-15	-20	-10	-21	-23	-11	-24
	12	**1.54**	-7	-11	-8	-12	-18	-7	-21	-20	-9	-26
120	**6**	**0.91**	-12	-15	-12	-16	-24	-11	-25	-26	-12	-28
	8	**1.10**	-8	-12	-9	-13	-22	-7	-24	-24	-9	-27
	12	**1.47**	-5	-10	-7	-11	-21	-7	-24	-23	-9	-28

Figures 4.13-4.15 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 4-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.13. It should be noted that Table 4.13 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.14 for all the bridges. Figures 4.16-4.18 show a trend similar to Figures 4.13-4.15, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

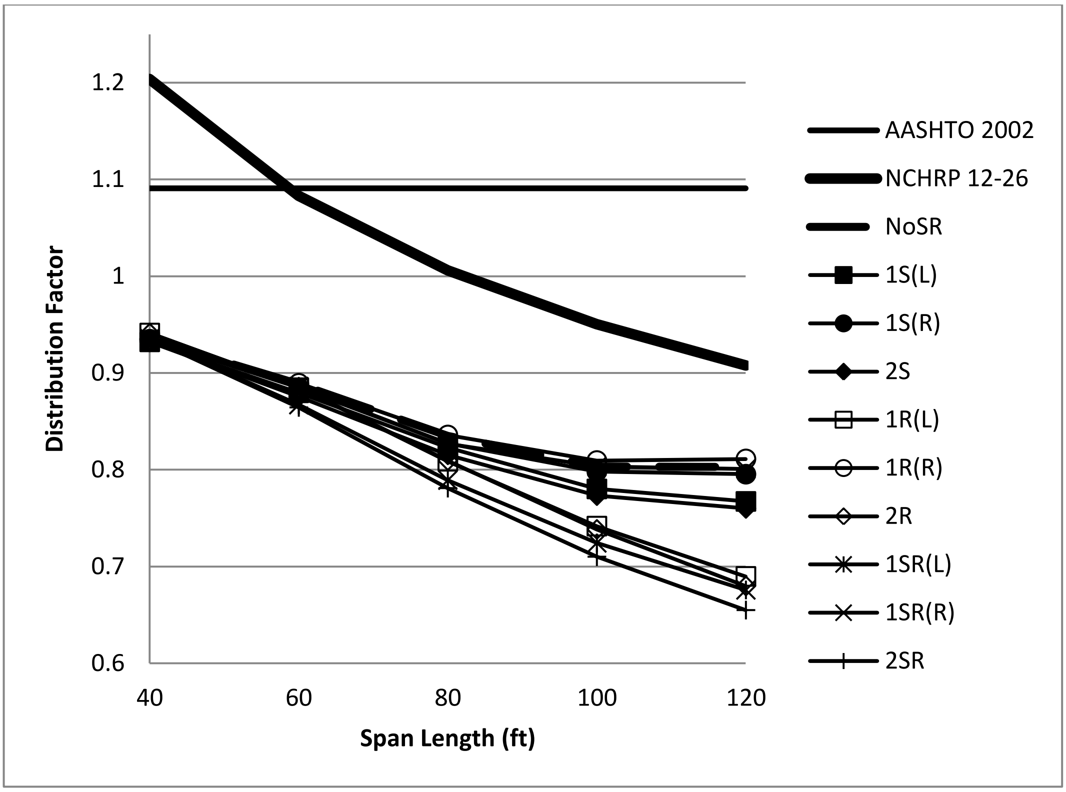


Figure 4.13. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

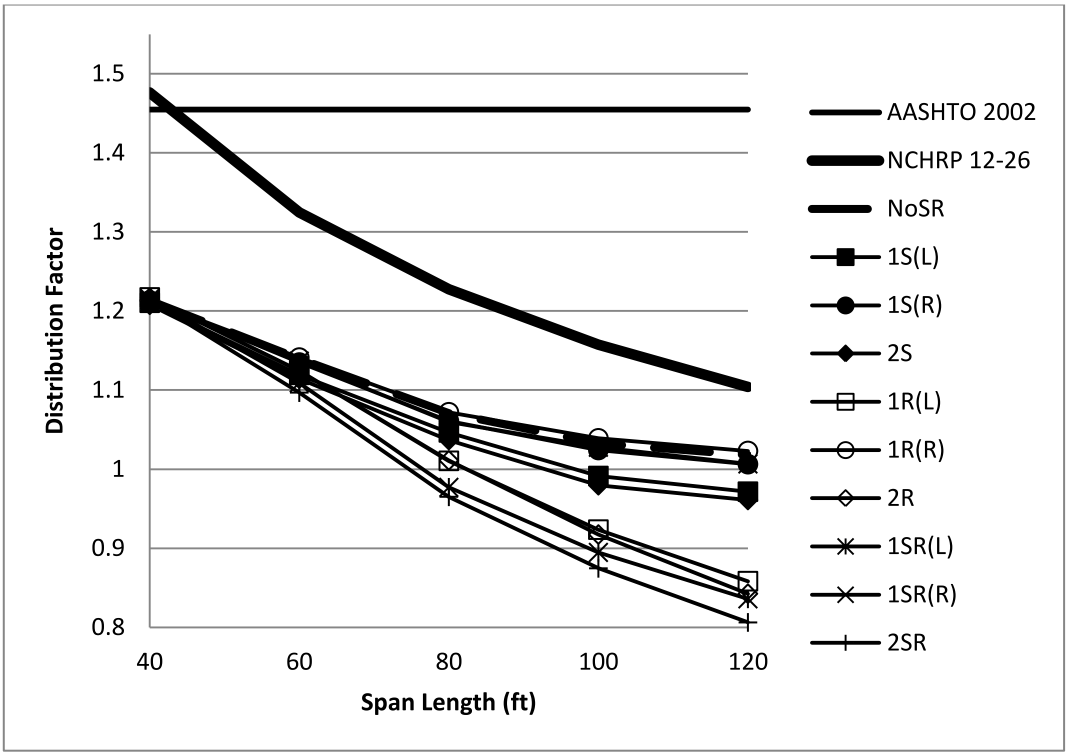


Figure 4.14. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

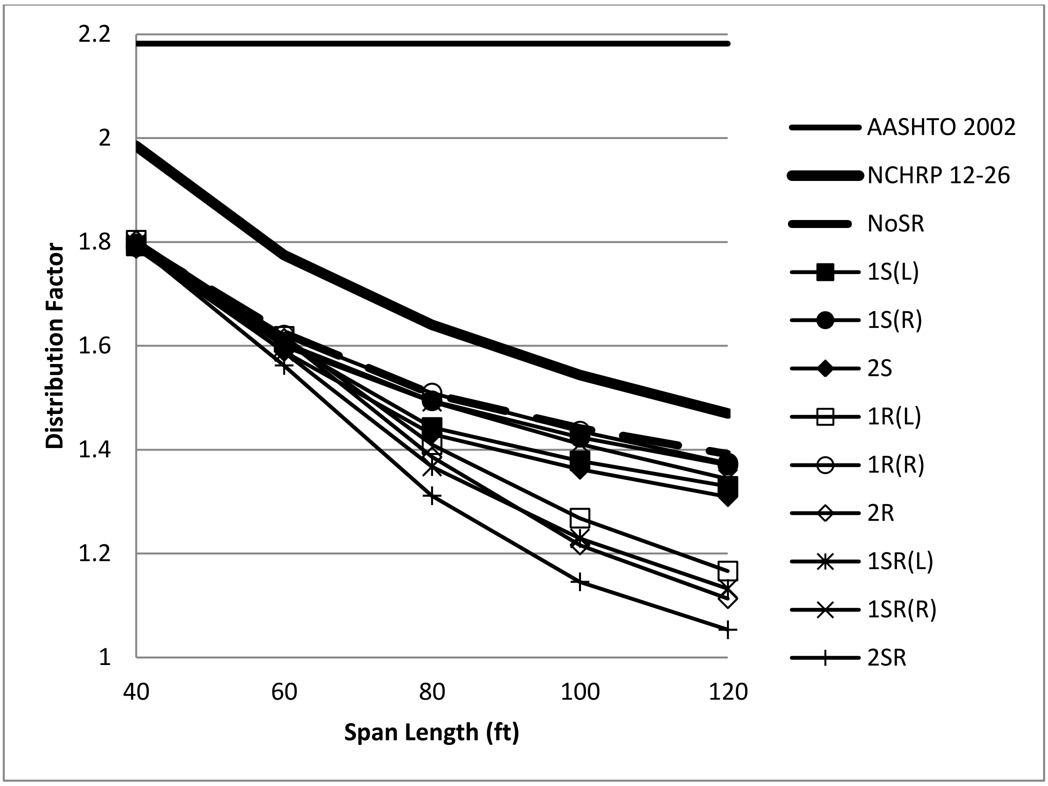


Figure 4.15. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.13. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	3137	3127	3137	3126	3158	3136	3157	3136	3137	3137
	8	2,698	3964	3954	3964	3944	3977	3964	3975	3971	3966	3963
	12	2,698	5607	5594	5590	5577	5630	5607	5630	5606	5592	5591
60	6	4,838	5388	5329	5363	5303	5352	5392	5355	5241	5370	5223
	8	4,838	6762	6634	6731	6604	6648	6769	6643	6549	6740	6470
	12	4,838	9239	9130	9132	9024	9178	9210	9149	8994	9108	8830
80	6	6,989	7314	7201	7265	7133	7049	7329	7051	6873	7282	6796
	8	6,989	9210	8985	9122	8900	8652	9211	8622	8464	9104	8217
	12	6,989	12569	12005	12455	11897	11711	12573	11482	11377	12425	10869
100	6	9,905	10112	9820	10021	9733	9203	10146	9149	8983	10069	8794
	8	9,905	12758	12179	12652	12078	11238	12830	11142	10962	12689	10614
	12	9,905	17219	16421	17003	16220	15018	17123	14328	14573	16826	13497
120	6	13,962	14474	13602	14358	13473	12118	14737	11955	11869	14579	11517
	8	13,962	18053	16980	17895	16793	14813	18406	14501	14427	18163	13870
	12	13,962	24079	22500	23835	22157	19595	24491	18547	19046	24069	17549

(b) Distribution factor = 0.75*DF = 0.75*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,698	0.87	0.87	0.87	0.87	0.88	0.87	0.88	0.87	0.87	0.87
	8	2,698	1.10	1.10	1.10	1.10	1.11	1.10	1.11	1.10	1.10	1.10
	12	2,698	1.56	1.56	1.55	1.55	1.57	1.56	1.57	1.56	1.55	1.55
60	6	4,838	0.84	0.83	0.83	0.82	0.83	0.84	0.83	0.81	0.83	0.81
	8	4,838	1.05	1.03	1.04	1.02	1.03	1.05	1.03	1.02	1.04	1.00
	12	4,838	1.43	1.42	1.42	1.40	1.42	1.43	1.42	1.39	1.41	1.37
80	6	6,989	0.78	0.77	0.78	0.77	0.76	0.79	0.76	0.74	0.78	0.73
	8	6,989	0.99	0.96	0.98	0.96	0.93	0.99	0.93	0.91	0.98	0.88
	12	6,989	1.35	1.29	1.34	1.28	1.26	1.35	1.23	1.22	1.33	1.17
100	6	9,905	0.77	0.74	0.76	0.74	0.70	0.77	0.69	0.68	0.76	0.67
	8	9,905	0.97	0.92	0.96	0.91	0.85	0.97	0.84	0.83	0.96	0.80
	12	9,905	1.30	1.24	1.29	1.23	1.14	1.30	1.08	1.10	1.27	1.02
120	6	13,962	0.78	0.73	0.77	0.72	0.65	0.79	0.64	0.64	0.78	0.62
	8	13,962	0.97	0.91	0.96	0.90	0.80	0.99	0.78	0.78	0.98	0.75
	12	13,962	1.29	1.21	1.28	1.19	1.05	1.32	1.00	1.02	1.29	0.94

Table 4.14. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-20	-20	-20	-20	-19	-20	-19	-20	-20	-20
	8	**1.46**	-25	-25	-25	-25	-24	-25	-24	-24	-24	-25
	12	**2.18**	-28	-29	-29	-29	-28	-28	-28	-29	-29	-29
60	**6**	**1.09**	-23	-24	-24	-25	-24	-23	-24	-25	-24	-26
	8	**1.46**	-28	-30	-29	-30	-29	-28	-29	-30	-28	-31
	12	**2.18**	-34	-35	-35	-36	-35	-35	-35	-36	-35	-37
80	**6**	**1.09**	-28	-29	-28	-30	-31	-28	-31	-32	-28	-33
	8	**1.46**	-32	-34	-33	-35	-36	-32	-37	-38	-33	-40
	12	**2.18**	-38	-41	-39	-41	-42	-38	-43	-44	-39	-46
100	**6**	**1.09**	-30	-32	-30	-32	-36	-30	-36	-38	-30	-39
	8	**1.46**	-34	-37	-34	-37	-42	-33	-42	-43	-34	-45
	12	**2.18**	-40	-43	-41	-44	-48	-41	-50	-49	-42	-53
120	**6**	**1.09**	-29	-33	-29	-34	-40	-27	-41	-42	-28	-43
	8	**1.46**	-34	-38	-34	-38	-45	-32	-47	-47	-33	-49
	12	**2.18**	-41	-45	-41	-45	-52	-40	-54	-53	-41	-57

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-28	-28	-28	-28	-27	-28	-27	-28	-28	-28
	8	**1.48**	-25	-26	-25	-26	-25	-25	-25	-25	-25	-25
	12	**1.98**	-21	-22	-22	-22	-21	-21	-21	-21	-22	-22
60	**6**	**1.08**	-23	-24	-23	-24	-23	-23	-23	-25	-23	-25
	8	**1.32**	-21	-22	-21	-23	-22	-21	-22	-23	-21	-24
	12	**1.77**	-19	-20	-20	-21	-20	-20	-20	-21	-20	-23
80	**6**	**1.01**	-22	-23	-22	-24	-25	-22	-25	-27	-22	-27
	8	**1.23**	-19	-21	-20	-22	-24	-19	-25	-26	-20	-28
	12	**1.64**	-18	-21	-19	-22	-23	-18	-25	-26	-19	-29
100	**6**	**0.95**	-19	-22	-20	-22	-27	-19	-27	-28	-20	-30
	8	**1.16**	-17	-20	-17	-21	-26	-16	-27	-28	-17	-31
	12	**1.54**	-16	-19	-17	-20	-26	-16	-30	-29	-17	-34
120	**6**	**0.91**	-14	-20	-15	-20	-28	-13	-29	-30	-14	-32
	8	**1.10**	-12	-17	-13	-18	-28	-10	-29	-30	-12	-33
	12	**1.47**	-12	-18	-13	-19	-28	-10	-32	-30	-12	-36

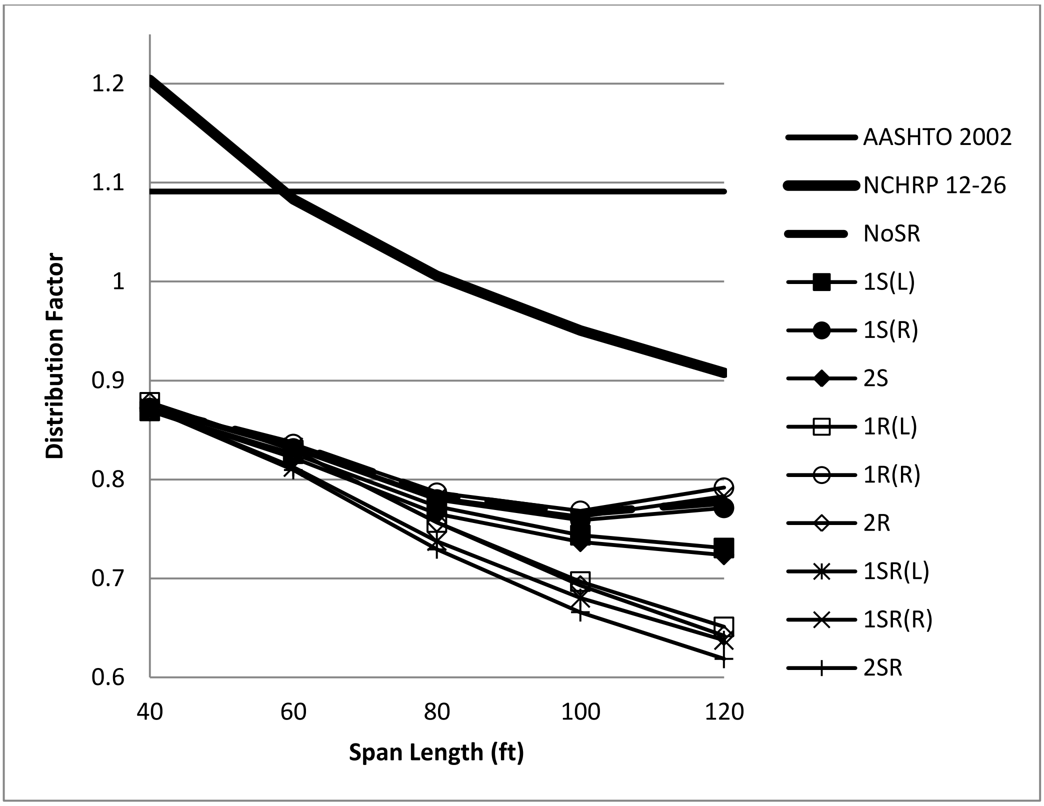


Figure 4.16. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6 ft)

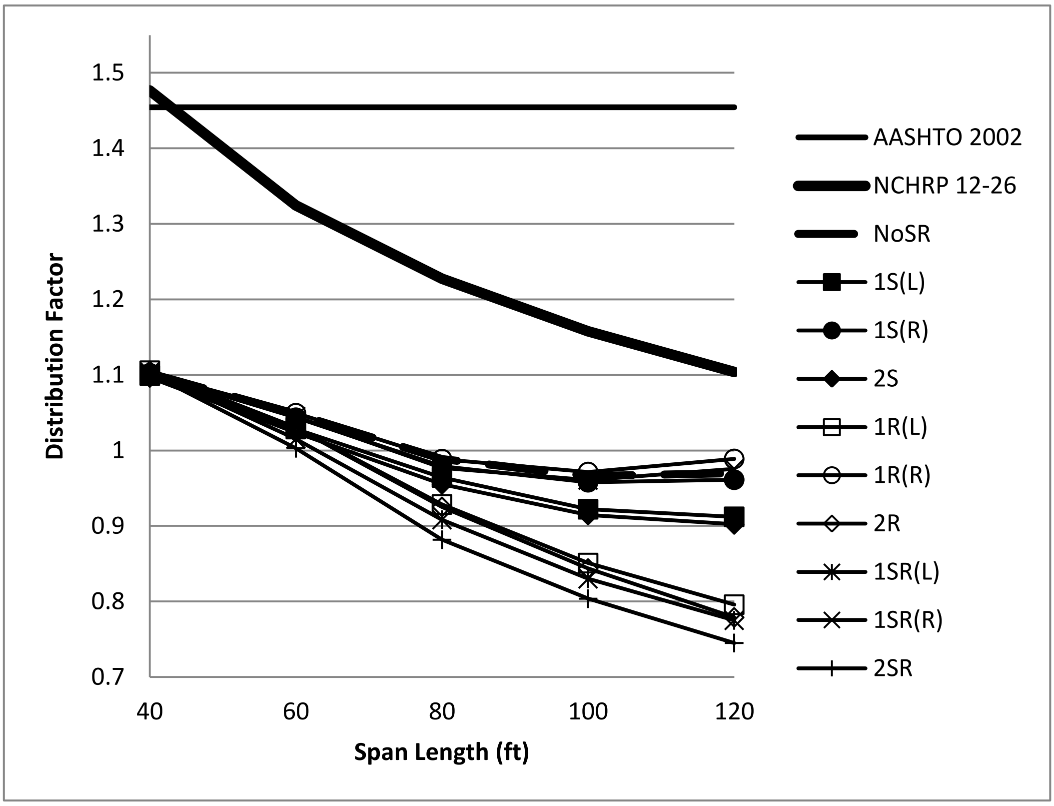


Figure 4.17. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8 ft)

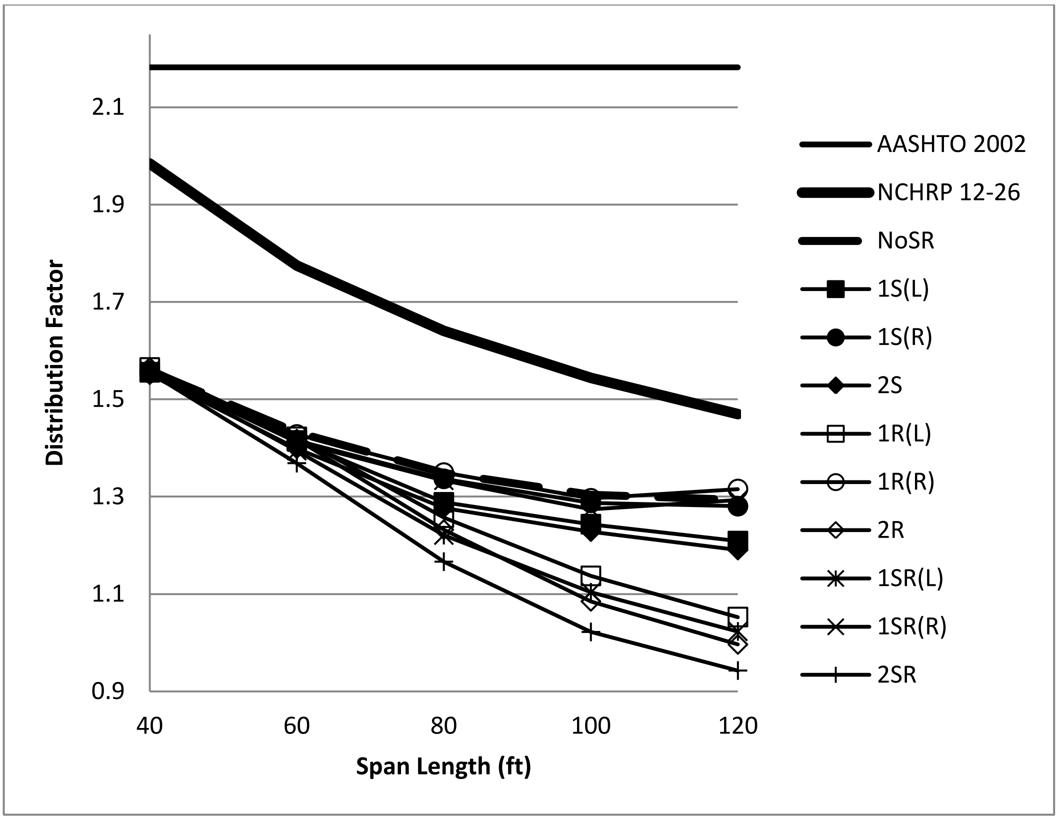


Figure 4.18. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12 ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.14 (considering steel beams only) as compared to Table 4.12 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.12 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: the FEA distribution factors are smaller than (1) by about 12 % for all spans.
2. Sidewalk on one side (left or right): (1) is about 14 % higher than FEA distribution factors for all spans.
3. Sidewalk on both sides: (1) is about 15 % higher than FEA distribution factors for all spans.
4. Railing on one side (left or right): (1) is about 15 % higher than FEA distribution factors for spans up to 80 ft and about 20 % higher for spans between 80 and 120 ft.
5. Railing on both sides: (1) is about 15 % higher than FEA distribution factors for spans up to 80 ft and about 20 % higher for spans between 80 and 120 ft.
6. Sidewalk and railing on one side (left or right): (1) is about 13 % higher than FEA for all spans.
7. Sidewalk and railing on both sides: (1) is about 17 % higher than FEA distribution factors for spans up to 80 ft and about 25 % higher for spans between 80 and 120 ft.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it's important to note that for spans up to 60 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a maximum of 4 % reduction in distribution factor is observed. Otherwise, the average reductions in FEA distribution factors when compared to the base case were 1 % when introducing sidewalks on one side; 3 % for spans between 60 and 80 ft (18 and 24 m) and 5 % for spans between 80 and 120 ft (24 and 36 m) when introducing sidewalks on both sides; 0 % when introducing railings on one side; 5 % for spans between 60 and 80 ft (18 and 24 m) and 10 % for spans between 80 and 100 ft (24 m and 30 m) and 16 % for spans between 100 and 120 ft (30 and 36 m) when introducing railings on both sides; 1% for a combination of sidewalks and railings on one side; and 8 % for spans between 60 and 80 ft (18 and 24 m) and 14 % for spans between 80 and 100 ft (24 and 30 m) and 20 % for spans between 100 and 120 ft (30 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.3. Two-Span Bridges - Positive Moment

4.3.1. Two-Lane Bridges

Tables A.10-A.12 in the appendix show a summary of the positive bending moments calculated in the concrete slab and in steel girders for a two-span 2-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.10 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 17 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 45 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 51 %.

Since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur in either one of the two left side girders. When the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment occurred in the left exterior girder. However, using Tables A.10-A.12 to identify the maximum bending moments at critical sections (usually occurring in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 10 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides. These maximum bending moments and FEA distribution factors are summarized in Table 4.15 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 2-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.16 for all the bridges.

Table 4.15. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	2360	2326	2315	2283	2366	2355	2360	2307	2306	2254
	8	2,150	3067	2953	3025	2914	2995	3067	2995	2820	3023	2778
	12	2,150	4348	4212	4213	4084	4306	4322	4280	4102	4154	3917
60	**6**	3,877	3758	3590	3695	3521	3543	3765	3474	3414	3685	3234
	8	3,877	4819	4605	4715	4510	4380	4785	4332	4096	4643	3937
	12	3,877	6450	6196	6195	5955	6061	6220	5804	5691	5872	5160
80	**6**	5,630	5247	4981	5139	4885	4609	5225	4371	4445	5070	4075
	8	5,630	6564	6252	6395	6099	5623	6415	5421	5317	6175	4973
	12	5,630	8593	8227	8221	7882	7662	7984	6979	7217	7512	6264
100	**6**	8,027	7250	6860	7076	6715	6075	7145	5653	5884	6879	5296
	8	8,027	8947	8501	8689	8277	7338	8575	6868	7079	8203	6384
	12	8,027	11596	11093	11082	10623	9869	10382	8559	9368	9780	7836
120	**6**	11,208	9895	9339	9618	9119	8112	9619	7458	7889	9195	7026
	8	11,208	12162	11535	11770	11203	9897	11424	8896	9572	10871	8350
	12	11,208	15764	15063	15047	14412	13005	13705	10903	12414	12925	10145

(b) Distribution factor = DF = Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	1.10	1.08	1.08	1.06	1.10	1.10	1.10	1.07	1.07	1.05
	8	2,150	1.43	1.37	1.41	1.36	1.39	1.43	1.39	1.31	1.41	1.29
	12	2,150	2.02	1.96	1.96	1.90	2.00	2.01	1.99	1.91	1.93	1.82
60	**6**	3,877	0.97	0.93	0.95	0.91	0.91	0.97	0.90	0.88	0.95	0.83
	8	3,877	1.24	1.19	1.22	1.16	1.13	1.23	1.12	1.06	1.20	1.02
	12	3,877	1.66	1.60	1.60	1.54	1.56	1.60	1.50	1.47	1.51	1.33
80	**6**	5,630	0.93	0.88	0.91	0.87	0.82	0.93	0.78	0.79	0.90	0.72
	8	5,630	1.17	1.11	1.14	1.08	1.00	1.14	0.96	0.94	1.10	0.88
	12	5,630	1.53	1.46	1.46	1.40	1.36	1.42	1.24	1.28	1.33	1.11
100	**6**	8,027	0.90	0.85	0.88	0.84	0.76	0.89	0.70	0.73	0.86	0.66
	8	8,027	1.11	1.06	1.08	1.03	0.91	1.07	0.86	0.88	1.02	0.80
	12	8,027	1.44	1.38	1.38	1.32	1.23	1.29	1.07	1.17	1.22	0.98
120	**6**	11,208	0.88	0.83	0.86	0.81	0.72	0.86	0.67	0.70	0.82	0.63
	8	11,208	1.09	1.03	1.05	1.00	0.88	1.02	0.79	0.85	0.97	0.74
	12	11,208	1.41	1.34	1.34	1.29	1.16	1.22	0.97	1.11	1.15	0.91

Table 4.16. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	1	-1	-1	-3	1	1	1	-2	-2	-4
	8	**1.46**	-2	-6	-4	-7	-5	-2	-5	-10	-4	-11
	12	**2.18**	-7	-10	-10	-13	-8	-8	-9	-12	-11	-16
60	**6**	**1.09**	-11	-15	-13	-17	-16	-11	-18	-19	-13	-23
	8	**1.46**	-15	-19	-17	-20	-23	-15	-23	-28	-18	-30
	12	**2.18**	-24	-27	-27	-30	-28	-26	-31	-33	-31	-39
80	**6**	**1.09**	-15	-19	-16	-20	-25	-15	-29	-28	-17	-34
	8	**1.46**	-20	-24	-22	-26	-32	-22	-34	-35	-25	-39
	12	**2.18**	-30	-33	-33	-36	-38	-35	-43	-41	-39	-49
100	**6**	**1.09**	-17	-22	-19	-23	-31	-18	-35	-33	-21	-39
	8	**1.46**	-24	-27	-26	-29	-37	-27	-41	-40	-30	-46
	12	**2.18**	-34	-37	-37	-39	-44	-41	-51	-46	-44	-55
120	**6**	**1.09**	-19	-24	-21	-25	-34	-21	-39	-35	-25	-42
	8	**1.46**	-26	-30	-28	-32	-40	-30	-46	-42	-34	-49
	12	**2.18**	-35	-38	-38	-41	-47	-44	-55	-49	-47	-58

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-9	-10	-11	-12	-9	-9	-9	-11	-11	-13
	8	**1.48**	-3	-7	-5	-8	-6	-3	-6	-11	-5	-12
	12	**1.98**	2	-1	-1	-4	1	1	0	-4	-3	-8
60	**6**	**1.08**	-11	-15	-12	-16	-16	-10	-17	-19	-12	-23
	8	**1.32**	-6	-10	-8	-12	-15	-7	-16	-20	-10	-23
	12	**1.77**	-6	-10	-10	-13	-12	-10	-16	-17	-15	-25
80	**6**	**1.01**	-7	-12	-9	-14	-19	-8	-23	-22	-10	-28
	8	**1.23**	-5	-10	-7	-12	-19	-7	-22	-23	-11	-28
	12	**1.64**	-7	-11	-11	-15	-17	-14	-24	-22	-19	-32
100	**6**	**0.95**	-5	-10	-7	-12	-20	-6	-26	-23	-10	-31
	8	**1.16**	-4	-9	-6	-11	-21	-8	-26	-24	-12	-31
	12	**1.54**	-6	-10	-11	-14	-20	-16	-31	-24	-21	-37
120	**6**	**0.91**	-3	-8	-5	-10	-20	-5	-27	-22	-10	-31
	8	**1.10**	-2	-7	-5	-9	-20	-8	-28	-23	-12	-33
	12	**1.47**	-4	-9	-9	-13	-21	-17	-34	-25	-22	-38

Figures 4.19-4.21 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 3-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.17. It should be noted that Table 4.17 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.18 for all the bridges. Figures 4.22-4.24 show a trend similar to Figures 4.19-4.21, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

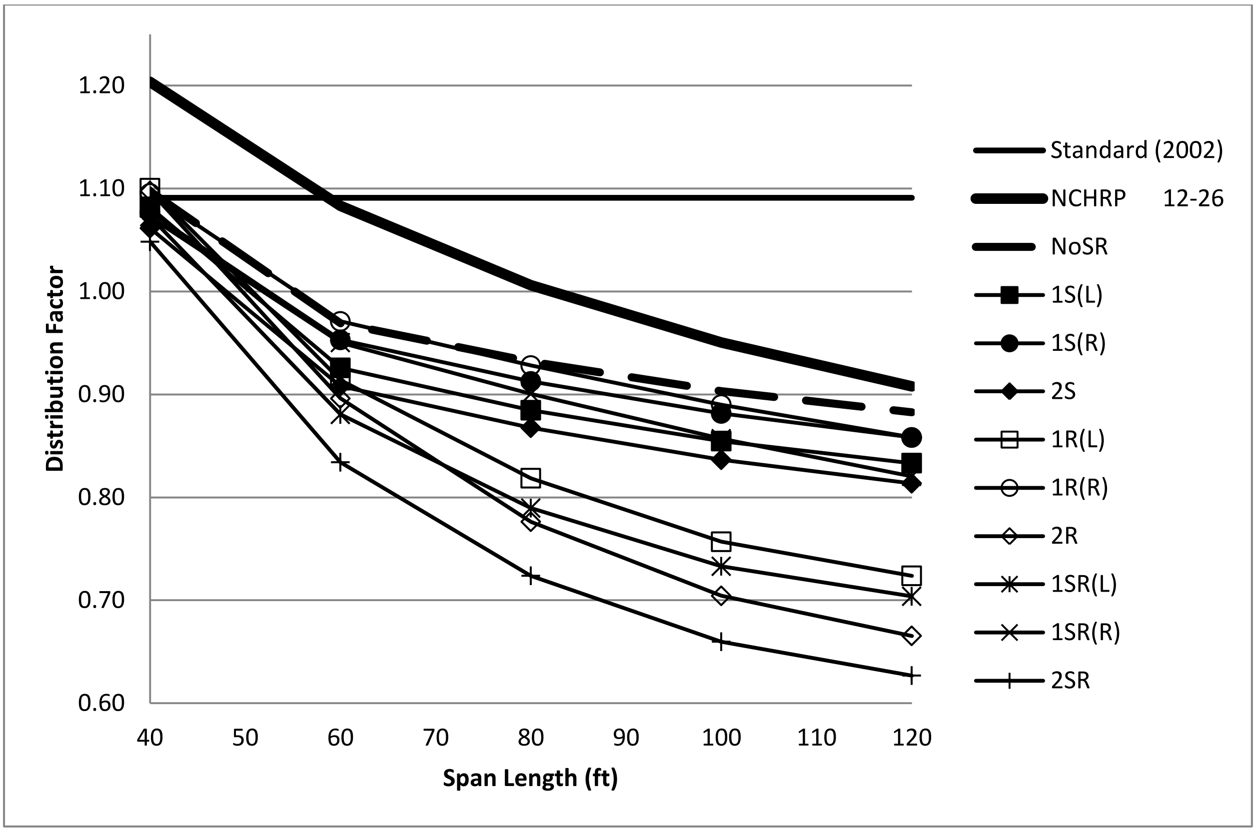


Figure 4.19. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

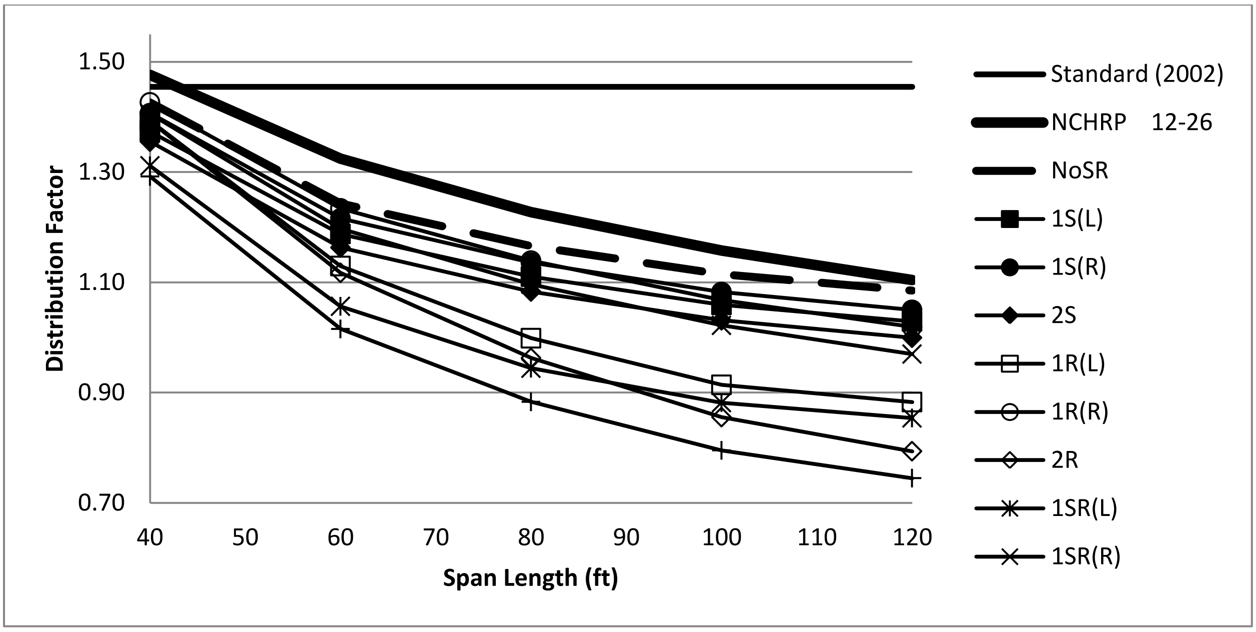


Figure 4.20. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

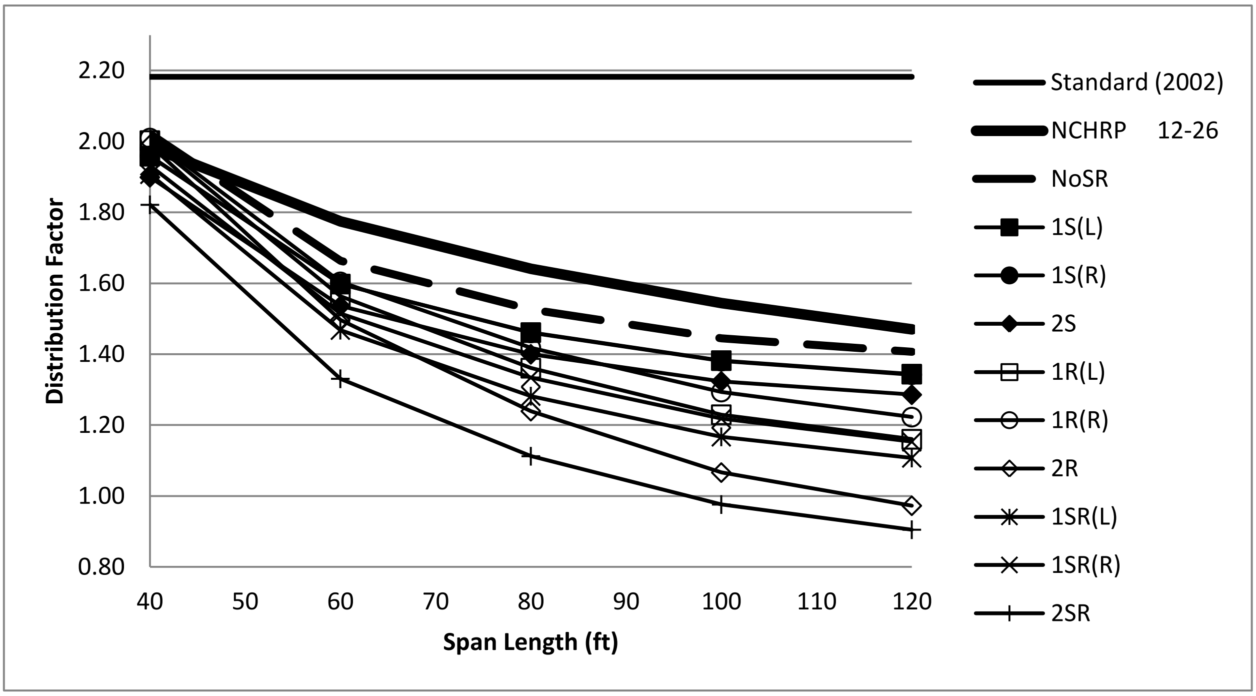


Figure 4.21. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.17. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	2184	2151	2142	2110	2187	2179	2181	2129	2132	2078
	8	2,150	2768	2664	2728	2627	2692	2767	2691	2531	2724	2490
	12	2,150	3721	3599	3602	3486	3674	3695	3647	3490	3543	3320
60	**6**	3,877	3495	3344	3435	3270	3286	3499	3214	3164	3423	2988
	8	3,877	4403	4204	4307	4114	3961	4365	3908	3704	4233	3546
	12	3,877	5610	5377	5386	5166	5218	5381	4961	4889	5074	4390
80	**6**	5,630	4938	4668	4849	4577	4300	5047	4057	4146	4915	3782
	8	5,630	6057	5759	5900	5616	5124	6094	4915	4851	5904	4509
	12	5,630	7634	7246	7450	6940	6675	7705	6007	6283	7407	5380
100	**6**	8,027	7147	6535	6980	6409	5735	7304	5296	5554	7023	4961
	8	8,027	8651	7931	8437	7720	6790	8779	6318	6539	8400	5871
	12	8,027	11053	9990	10743	9740	8783	11022	7517	8335	10474	6873
120	**6**	11,208	9896	9025	9610	8827	7695	10037	7028	7484	9543	6617
	8	11,208	11989	10887	11624	10623	9216	12018	8239	8912	11367	7730
	12	11,208	15341	13853	14817	13448	11695	15003	9682	11167	14092	9005

(b) Distribution factor = DF = Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	1.02	1.00	1.00	0.98	1.02	1.01	1.01	0.99	0.99	0.97
	8	2,150	1.29	1.24	1.27	1.22	1.25	1.29	1.25	1.18	1.27	1.16
	12	2,150	1.73	1.67	1.68	1.62	1.71	1.72	1.70	1.62	1.65	1.54
60	**6**	3,877	0.90	0.86	0.89	0.84	0.85	0.90	0.83	0.82	0.88	0.77
	8	3,877	1.14	1.08	1.11	1.06	1.02	1.13	1.01	0.96	1.09	0.91
	12	3,877	1.45	1.39	1.39	1.33	1.35	1.39	1.28	1.26	1.31	1.13
80	**6**	5,630	0.88	0.83	0.86	0.81	0.76	0.90	0.72	0.74	0.87	0.67
	8	5,630	1.08	1.02	1.05	1.00	0.91	1.08	0.87	0.86	1.05	0.80
	12	5,630	1.36	1.29	1.32	1.23	1.19	1.37	1.07	1.12	1.32	0.96
100	**6**	8,027	0.89	0.81	0.87	0.80	0.71	0.91	0.66	0.69	0.87	0.62
	8	8,027	1.08	0.99	1.05	0.96	0.85	1.09	0.79	0.81	1.05	0.73
	12	8,027	1.38	1.24	1.34	1.21	1.09	1.37	0.94	1.04	1.30	0.86
120	**6**	11,208	0.88	0.81	0.86	0.79	0.69	0.90	0.63	0.67	0.85	0.59
	8	11,208	1.07	0.97	1.04	0.95	0.82	1.07	0.74	0.80	1.01	0.69
	12	11,208	1.37	1.24	1.32	1.20	1.04	1.34	0.86	1.00	1.26	0.80

Table 4.18. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-7	-8	-9	-10	-7	-7	-7	-9	-9	-11
	8	**1.46**	-12	-15	-13	-16	-14	-12	-14	-19	-13	-21
	12	**2.18**	-21	-23	-23	-26	-22	-21	-22	-26	-24	-29
60	**6**	**1.09**	-17	-21	-19	-23	-22	-17	-24	-25	-19	-29
	8	**1.46**	-22	-26	-24	-27	-30	-23	-31	-35	-25	-37
	12	**2.18**	-34	-36	-36	-39	-38	-36	-41	-42	-40	-48
80	**6**	**1.09**	-20	-24	-21	-25	-30	-18	-34	-32	-20	-38
	8	**1.46**	-26	-30	-28	-32	-38	-26	-40	-41	-28	-45
	12	**2.18**	-38	-41	-39	-43	-46	-37	-51	-49	-40	-56
100	**6**	**1.09**	-18	-25	-20	-27	-34	-17	-39	-37	-20	-43
	8	**1.46**	-26	-32	-28	-34	-42	-25	-46	-44	-28	-50
	12	**2.18**	-37	-43	-39	-44	-50	-37	-57	-52	-40	-61
120	**6**	**1.09**	-19	-26	-21	-28	-37	-18	-42	-39	-22	-46
	8	**1.46**	-27	-33	-29	-35	-44	-27	-50	-46	-31	-53
	12	**2.18**	-37	-43	-39	-45	-52	-39	-60	-54	-42	-63

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-16	-17	-17	-18	-15	-16	-16	-18	-18	-20
	8	**1.48**	-13	-16	-14	-17	-15	-13	-15	-20	-14	-22
	12	**1.98**	-13	-16	-16	-18	-14	-13	-15	-18	-17	-22
60	**6**	**1.08**	-17	-20	-18	-22	-22	-17	-23	-25	-18	-29
	8	**1.32**	-14	-18	-16	-20	-23	-15	-24	-28	-18	-31
	12	**1.77**	-18	-22	-22	-25	-24	-22	-28	-29	-26	-36
80	**6**	**1.01**	-13	-18	-14	-19	-24	-11	-28	-27	-13	-33
	8	**1.23**	-12	-17	-15	-19	-26	-12	-29	-30	-15	-35
	12	**1.64**	-17	-22	-19	-25	-28	-17	-35	-32	-20	-42
100	**6**	**0.95**	-6	-14	-9	-16	-25	-4	-31	-27	-8	-35
	8	**1.16**	-7	-15	-9	-17	-27	-6	-32	-30	-10	-37
	12	**1.54**	-11	-19	-13	-21	-29	-11	-39	-33	-15	-45
120	**6**	**0.91**	-3	-11	-6	-13	-24	-1	-31	-26	-6	-35
	8	**1.10**	-3	-12	-6	-14	-26	-3	-33	-28	-8	-38
	12	**1.47**	-7	-16	-10	-18	-29	-9	-41	-32	-14	-45

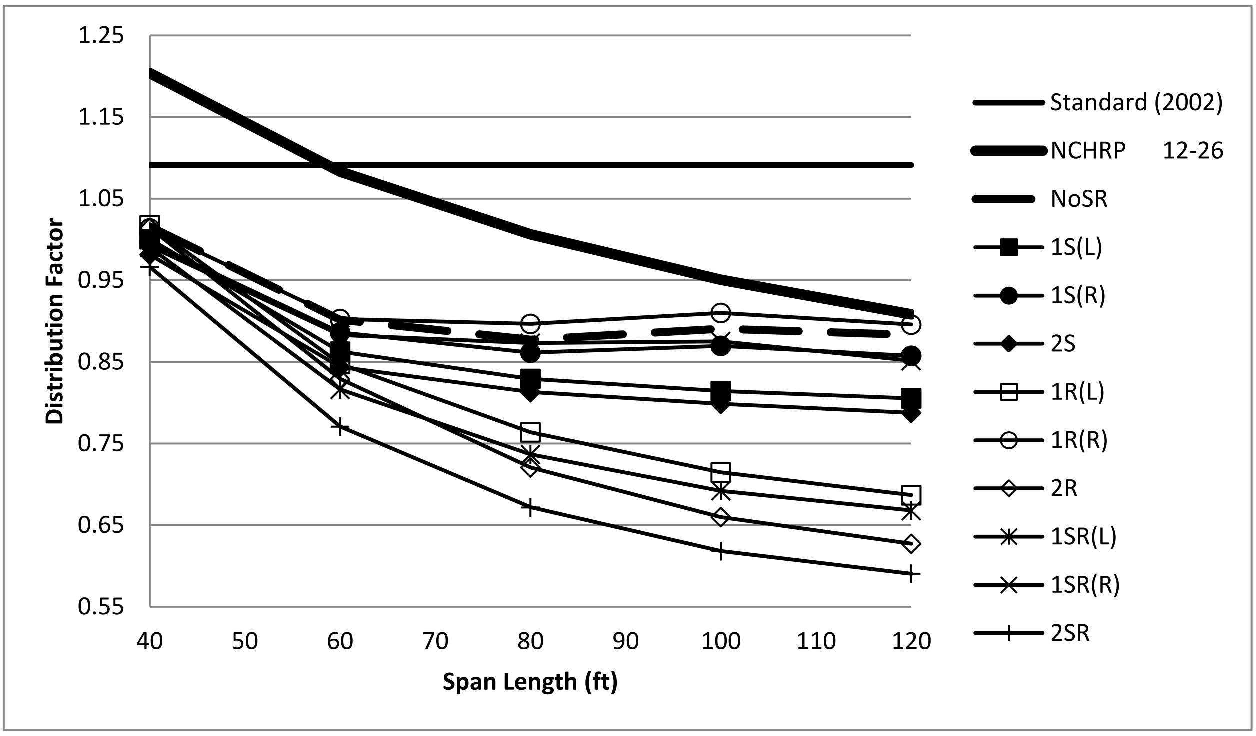


Figure 4.22. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6 ft)

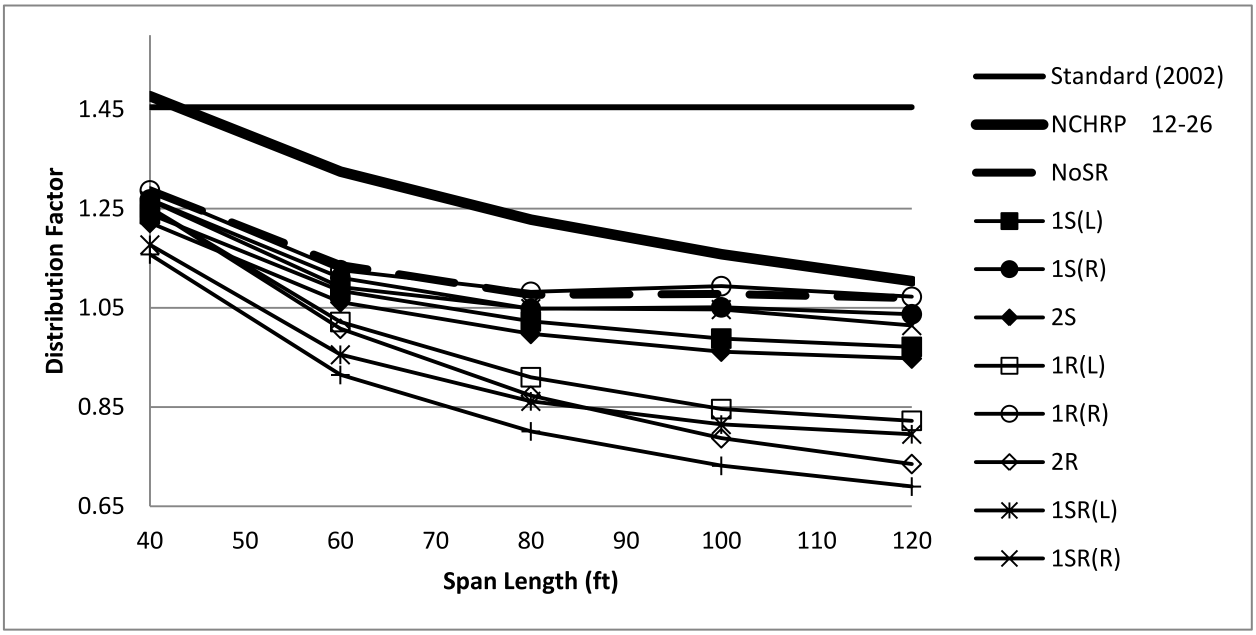


Figure 4.23. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8 ft)

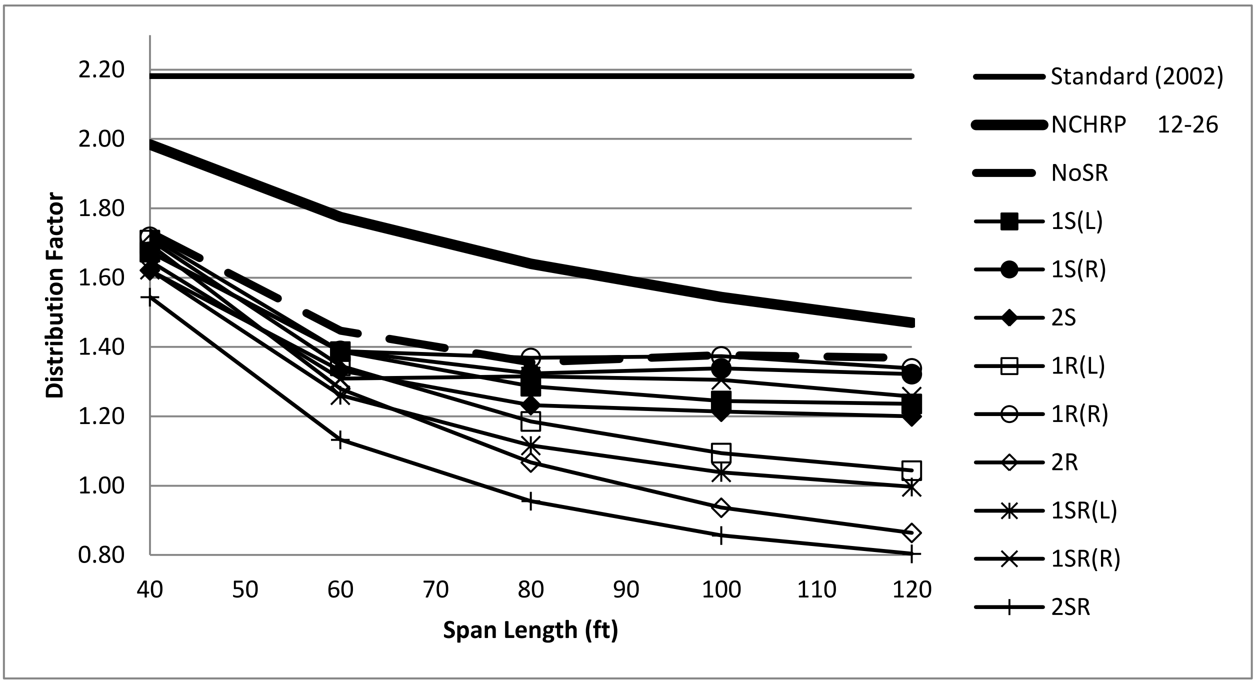


Figure 4.24. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12 ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.18 (considering steel beams only) as compared to Table 4.16 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.16 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: the FEA distribution factors are smaller than (1) by about 3 % for spans up to 40 ft as well as for spans between 100 and 120 ft, and FEA distribution factors are smaller than (1) by about 6 % for spans between 40 and 100 ft.
2. Sidewalk on one side (left or right): for spans up to 40 ft, (1) is about 7 % higher than the FEA distribution factors for girder spacings up to 8 ft, while (1) is nearly equal to the FEA results for girder spacings between 8 and 12 ft. However, (1) is 10 % higher than FEA results for spans between 40 and 80 and about 7 % higher for spans between 80 and 120 ft.
3. Sidewalk on both sides: for spans up to 40ft, (1) is about 8 % higher than FEA distribution factors for girder spacings up to 8ft and about 4 % higher for girder spacings between 8 and 12 ft. However, (1) is 12 % higher than FEA distribution factors for spans between 40 and 80 ft, and about 10 % higher for spans for spans between 80 and 120 ft.
4. Railing on one side (left or right): for spans up to 40 ft, (1) is about 9 % higher than FEA distribution factors for girder spacings up to 6ft and about 3

% higher for girder sapcings between 6 and 8 ft and are nearly equal to the FEA distribution factors for girder spacings between 8 and 12 ft. Furthermore, (1) is about 7 % higher than FEA distribution factors for spans between 40 and 60 ft. On the same hand, for spans between 60 and 120 ft, (1) is 7 % higher than the FEA distribution factors for girder spacings up to 8ft and about 14 % higher for girder spacings between 8 and 12 ft.

5. Railing on both sides: for spans up to 40 ft, (1) is about 6 % higher than FEA distribution factors for girder spacings up to 8 ft while (1) is nearly equal to the FEA distribution factors for girder spacings between 8 and 12 ft. Otherwise, (1) is about 16 % higher than FEA distribution factors for spans between 40 and 60 ft and about 22 % higher for spans between 60 and 80 ft, and about 26 % higher for spans between 80 and 120 ft.
6. Sidewalk and railing on one side (left or right): for spans up to 40 ft, (1) is about 11 % higher than FEA distribution factors for girder spacings up to 6 ft, and about 3 % higher for girder spacings between 6 and 12 ft. For spans between 40 and 60 ft, (1) is about 10 % higher than FEA distribution factors for girder spacings up to 8 ft, and about 15 % higher for girder spacings between 8 and 12 ft. For spans between 60 and 120 ft, (1) is about 10 % higher than FEA distribution factors for girder spacings up to 8 ft, and about 19 % higher for girder spacings between 8 and 12 ft.
7. Sidewalk and railing on both sides: (1) is about 10 % higher than FEA distribution factors for spans up to 40 ft, and about 23 % higher for spans between 40 and 60 ft, and about 28 % higher for spans between 60 and 80 ft, and about 31 % higher for spans between 80 and 120 ft.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it's important to note that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a 9 % reduction in distribution factor is observed. Otherwise, the average reductions in FEA distribution factors when compared to the base case were 3 % when introducing sidewalks on one side; 7 % when introducing sidewalks on both sides; 1 % for spans between 40 and 120 ft (12 and 36 m) with girder spacing up to 8ft and 4 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing between 8 and 12 ft and 10 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 8 and 12 ft when introducing railings on one side; 10 % for spans between 40 and 60 ft (12 and 18 m) and 17 % for spans between 60 and 80 ft (18 m and 24 m) and 23 % for spans between 80 and 120 ft (24 and 36 m) when introducing railings on both sides; 3 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing up to 8 ft and 9 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing between 8 and 12 ft and 6 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 8 ft and 16 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 8 and 12 ft for a combination of sidewalks and railings on one side; and 14 % for spans between 40 and 60 ft (12 and 18 m) and 23 % for spans between 60 and 80 ft (18 and 24 m) and 29 % for spans between 80 and 120 ft

(24 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.3.2. Three-Lane Bridges

Tables A.13-A.15 in the appendix show a summary of the positive bending moments calculated in the concrete slab and in steel girders for a two-span 3-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.13 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 17 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 43 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 50 %.

Unlike the two-lane case, the discussion of the three-lane case is handled in two parts. First, we'll consider the three-lane bridges with a 6ft girder spacing separately. Similarly to the previous discussion for the 2-lane case; since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur in one of the three left side girders. When the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment occurred in the left exterior girder. However, using Table A.13 to identify the maximum bending moments at critical sections (usually occurring in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge.

However, Tables A.14 and A.15 show that for three-lane bridge cases with a girder spacing of 8ft and 12 ft, the maximum bending moment (girder+slab) is mostly observed in one of the two inner girders unlike the two-lane cases in which the maximum moment used to be observed in one of the two adjacent exterior girders regardless of girder spacing. This is mainly due to the transverse (lateral) positioning of truck loads on the bridge cross-section which differs between the two-lane and three-lane cases (reflected in figure 3.8). However, the latter has no significant effect on our approach to the analysis as we're already excluding the exterior girders which used to produce the highest total moment (girder+slab) in the 2-lane case for any girder spacing. In this context, although the maximum moment is mostly occurring in one of the interior girders even after placing sidewalks and/or railing on the heavily loaded edge; excluding the exterior girders moment would affect nothing but only make sure that if for any bridge case, the addition of sidewalks and railings increases the moment in the exterior girder such that it exceeds the maximum moment in any one of the interior girders; this exterior girder moment is excluded from the analysis as it is an overestimate of the maximum design bending moment. Furthermore, it's important to note that although the maximum moment was still observed in an interior girder, it's worth to be noted that its value was significantly reduced by the placing of sidewalks and/or railings, which is to be discussed further later on. It is worth mentioning that all the maximum wheel load distributions of the 3-lane bridge cases have been reduced by 10 %, as permitted by the AASHTO Standard Specifications (2002) for 3-lane bridges, in order to account for the improbable situation of having all lanes loaded at the same time and at locations along the bridge deck producing the maximum bending moment in a bridge superstructure. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 12 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides. These maximum bending moments and FEA distribution factors are summarized in Table 4.19 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 3-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.20 for all the bridges.

Table 4.19. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,150	2611	2589	2607	2571	2618	2610	2614	2600	2608	2582
	8	2,150	3427	3355	3410	3338	3423	3427	3424	3346	3413	3332
	12	2,150	4841	4815	4696	4671	4846	4809	4813	4821	4617	4598
60	6	3,877	4355	4263	4313	4223	4251	4358	4242	4166	4316	4080
	8	3,877	5530	5364	5432	5271	5451	5528	5449	5279	5419	5137
	12	3,877	7689	7575	7388	7283	7685	7376	7367	7537	7191	6819
80	6	5,630	5936	5788	5855	5711	5539	5932	5529	5409	5833	5248
	8	5,630	7240	7024	7087	6881	6981	7182	6875	6792	6989	6434
	12	5,630	10210	10020	9774	9599	10086	9606	9174	9816	9339	8437
100	6	8,027	8185	7808	8078	7693	7187	8244	7107	6998	8099	6758
	8	8,027	9479	9223	9266	9023	8898	9256	8535	8638	8974	8029
	12	8,027	13603	13324	13007	12752	13190	12538	11313	12786	12161	10485
120	6	11,208	11275	10695	11087	10533	9389	11338	9164	9151	11067	8754
	8	11,208	12422	12084	12163	11843	11533	12001	10600	11231	11672	10063
	12	11,208	18221	17802	17411	17030	17309	16457	14186	16723	15934	13283

(b) Distribution factor = 0.9*DF = 0.9*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,150	1.09	1.08	1.09	1.08	1.10	1.09	1.09	1.09	1.09	1.08
	8	2,150	1.43	1.40	1.43	1.40	1.43	1.43	1.43	1.40	1.43	1.39
	12	2,150	2.03	2.02	1.97	1.96	2.03	2.01	2.01	2.02	1.93	1.92
60	6	3,877	1.01	0.99	1.00	0.98	0.99	1.01	0.98	0.97	1.00	0.95
	8	3,877	1.28	1.25	1.26	1.22	1.27	1.28	1.26	1.23	1.26	1.19
	12	3,877	1.78	1.76	1.72	1.69	1.78	1.71	1.71	1.75	1.67	1.58
80	6	5,630	0.95	0.93	0.94	0.91	0.89	0.95	0.88	0.86	0.93	0.84
	8	5,630	1.16	1.12	1.13	1.10	1.12	1.15	1.10	1.09	1.12	1.03
	12	5,630	1.63	1.60	1.56	1.53	1.61	1.54	1.47	1.57	1.49	1.35
100	6	8,027	0.92	0.88	0.91	0.86	0.81	0.92	0.80	0.78	0.91	0.76
	8	8,027	1.06	1.03	1.04	1.01	1.00	1.04	0.96	0.97	1.01	0.90
	12	8,027	1.53	1.49	1.46	1.43	1.48	1.41	1.27	1.43	1.36	1.18
120	6	11,208	0.91	0.86	0.89	0.85	0.75	0.91	0.74	0.73	0.89	0.70
	8	11,208	1.00	0.97	0.98	0.95	0.93	0.96	0.85	0.90	0.94	0.81
	12	11,208	1.46	1.43	1.40	1.37	1.39	1.32	1.14	1.34	1.28	1.07

Table 4.20. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	0	-1	0	-1	1	0	0	0	0	-1
	8	**1.46**	-2	-4	-2	-4	-2	-2	-2	-4	-2	-4
	12	**2.18**	-7	-8	-10	-10	-7	-8	-8	-7	-11	-12
60	**6**	**1.09**	-7	-9	-8	-10	-9	-7	-10	-11	-8	-13
	8	**1.46**	-12	-15	-14	-16	-13	-12	-13	-16	-14	-18
	12	**2.18**	-18	-19	-21	-22	-18	-21	-22	-20	-23	-27
80	**6**	**1.09**	-13	-15	-14	-16	-19	-13	-19	-21	-14	-23
	8	**1.46**	-21	-23	-22	-25	-24	-21	-25	-26	-23	-30
	12	**2.18**	-25	-27	-28	-30	-26	-30	-33	-28	-32	-38
100	**6**	**1.09**	-16	-20	-17	-21	-26	-15	-27	-28	-17	-30
	8	**1.46**	-27	-29	-29	-31	-32	-29	-34	-34	-31	-38
	12	**2.18**	-30	-31	-33	-34	-32	-36	-42	-34	-37	-46
120	**6**	**1.09**	-17	-21	-18	-22	-31	-16	-32	-33	-18	-36
	8	**1.46**	-32	-34	-33	-35	-37	-34	-42	-38	-36	-45
	12	**2.18**	-33	-34	-36	-37	-36	-39	-48	-38	-41	-51

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-9	-10	-9	-11	-9	-9	-9	-10	-9	-10
	8	**1.48**	-3	-5	-3	-5	-3	-3	-3	-5	-3	-6
	12	**1.98**	2	2	-1	-1	2	1	2	2	-3	-3
60	**6**	**1.08**	-7	-9	-8	-9	-9	-7	-9	-11	-7	-13
	8	**1.32**	-3	-6	-5	-8	-4	-3	-5	-7	-5	-10
	12	**1.77**	1	-1	-3	-5	1	-4	-4	-1	-6	-11
80	**6**	**1.01**	-6	-8	-7	-9	-12	-6	-12	-14	-7	-17
	8	**1.23**	-6	-9	-8	-10	-9	-6	-10	-12	-9	-16
	12	**1.64**	0	-2	-5	-6	-2	-6	-11	-4	-9	-18
100	**6**	**0.95**	-3	-8	-5	-9	-15	-3	-16	-17	-4	-20
	8	**1.16**	-8	-11	-10	-13	-14	-10	-17	-16	-13	-22
	12	**1.54**	-1	-3	-6	-7	-4	-9	-18	-7	-12	-24
120	**6**	**0.91**	0	-5	-2	-7	-17	0	-19	-19	-2	-23
	8	**1.10**	-10	-12	-12	-14	-16	-13	-23	-18	-15	-27
	12	**1.47**	0	-3	-5	-7	-5	-10	-22	-9	-13	-27

Figures 4.25-4.27 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 3-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.21. It should be noted that Table 4.21 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.22 for all the bridges. Figures 4.28-4.30 show a trend similar to Figures 4.25-4.27, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

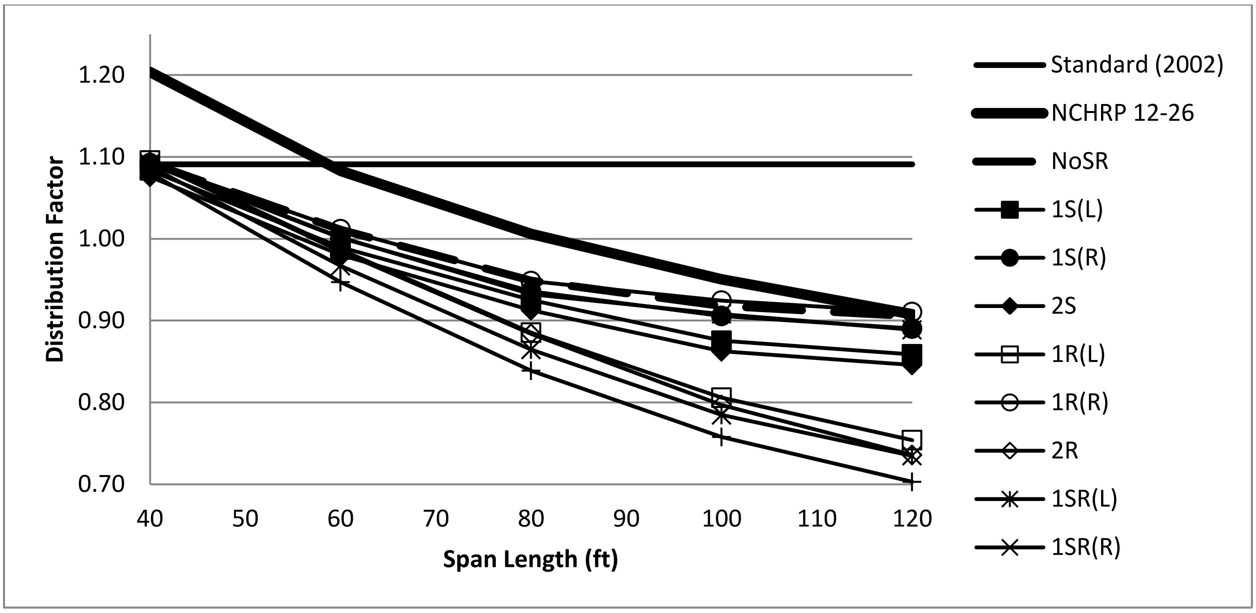


Figure 4.25. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

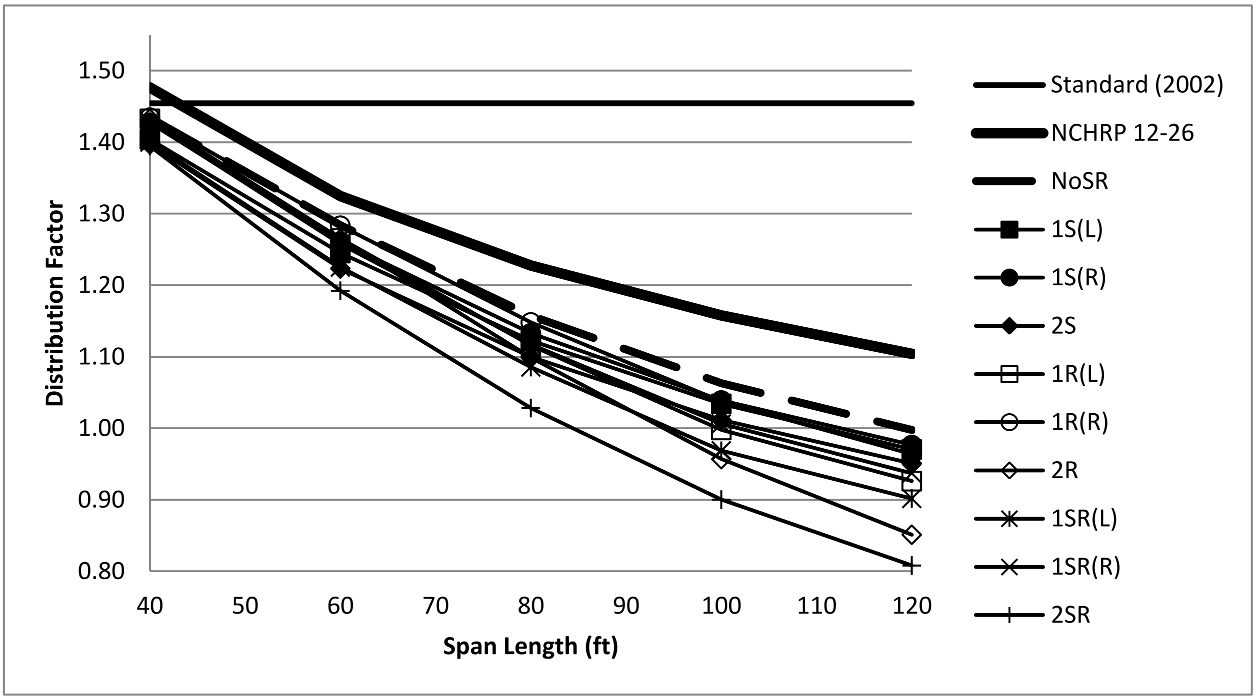


Figure 4.26. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

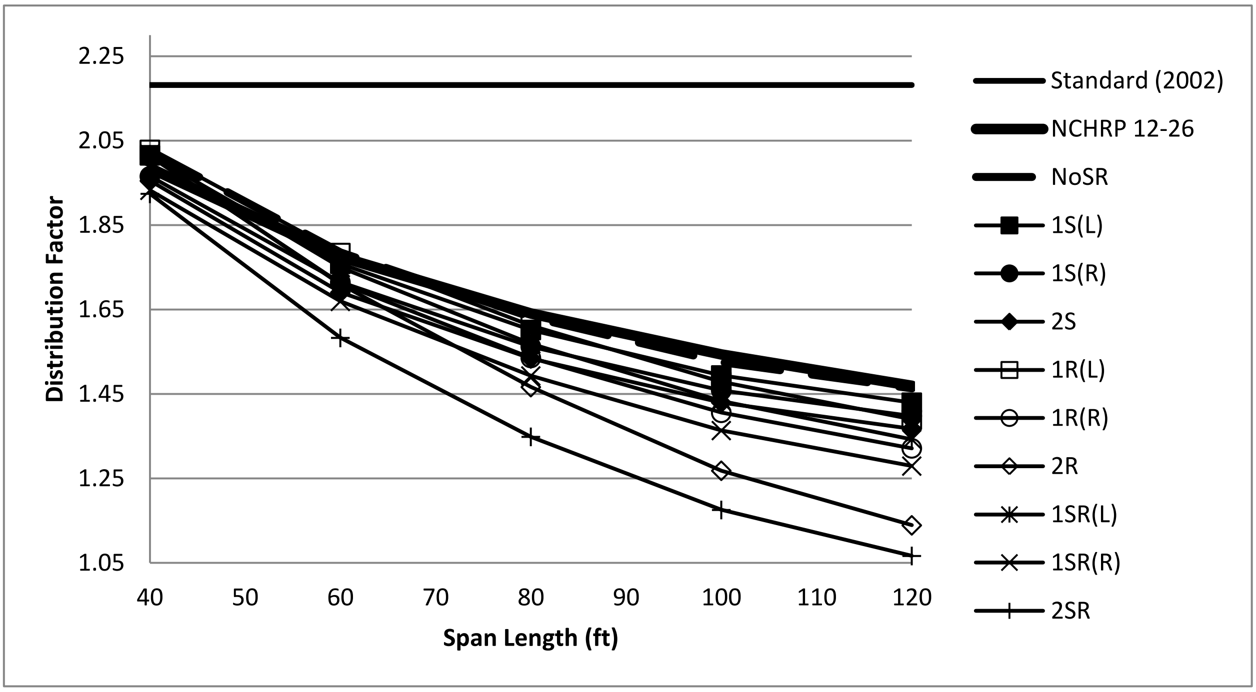


Figure 4.27. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.21. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	2437	2413	2432	2397	2440	2435	2437	2422	2433	2404
	8	2,150	3120	3052	3103	3035	3115	3120	3116	3041	3106	3027
	12	2,150	4193	4167	4064	4039	4196	4158	4162	4172	3986	3965
60	**6**	3,877	4088	4000	4047	3960	3973	4090	3963	3890	4049	3806
	8	3,877	5063	4910	4970	4821	4979	5058	4973	4786	4953	4681
	12	3,877	6752	6646	6486	6387	6740	6437	6420	6602	6284	5928
80	**6**	5,630	5593	5448	5516	5375	5183	5586	5169	5058	5491	4905
	8	5,630	6648	6450	6505	6316	6343	6582	6267	6167	6402	5863
	12	5,630	9046	8871	8655	8492	8909	8465	8008	8662	8221	7356
100	**6**	8,027	7782	7408	7679	7301	6779	7905	6691	6597	7790	6363
	8	8,027	8765	8518	8578	8342	8215	8542	7806	7988	8297	7348
	12	8,027	12252	11994	11700	11460	11828	11218	9998	11464	10876	9266
120	**6**	11,208	11057	10239	10878	10091	8882	11308	8646	8660	11043	8261
	8	11,208	11577	11257	11336	11030	10649	11192	9759	10372	10883	9274
	12	11,208	16530	16142	15772	15413	16046	14829	12610	15534	14356	11813

(b) Distribution factor = 0.9*DF = 0.9*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	1.02	1.01	1.02	1.00	1.02	1.02	1.02	1.01	1.02	1.01
	8	2,150	1.31	1.28	1.30	1.27	1.30	1.31	1.30	1.27	1.30	1.27
	12	2,150	1.76	1.74	1.70	1.69	1.76	1.74	1.74	1.75	1.67	1.66
60	**6**	3,877	0.95	0.93	0.94	0.92	0.92	0.95	0.92	0.90	0.94	0.88
	8	3,877	1.18	1.14	1.15	1.12	1.16	1.17	1.15	1.11	1.15	1.09
	12	3,877	1.57	1.54	1.51	1.48	1.56	1.49	1.49	1.53	1.46	1.38
80	**6**	5,630	0.89	0.87	0.88	0.86	0.83	0.89	0.83	0.81	0.88	0.78
	8	5,630	1.06	1.03	1.04	1.01	1.01	1.05	1.00	0.99	1.02	0.94
	12	5,630	1.45	1.42	1.38	1.36	1.42	1.35	1.28	1.38	1.31	1.18
100	**6**	8,027	0.87	0.83	0.86	0.82	0.76	0.89	0.75	0.74	0.87	0.71
	8	8,027	0.98	0.96	0.96	0.94	0.92	0.96	0.88	0.90	0.93	0.82
	12	8,027	1.37	1.34	1.31	1.28	1.33	1.26	1.12	1.29	1.22	1.04
120	**6**	11,208	0.89	0.82	0.87	0.81	0.71	0.91	0.69	0.70	0.89	0.66
	8	11,208	0.93	0.90	0.91	0.89	0.86	0.90	0.78	0.83	0.87	0.74
	12	11,208	1.33	1.30	1.27	1.24	1.29	1.19	1.01	1.25	1.15	0.95

Table 4.22. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-6	-7	-7	-8	-6	-6	-6	-7	-7	-8
	8	**1.46**	-11	-13	-11	-13	-11	-11	-11	-13	-11	-13
	12	**2.18**	-19	-20	-22	-22	-19	-20	-20	-20	-23	-24
60	**6**	**1.09**	-13	-15	-14	-16	-15	-13	-16	-17	-14	-19
	8	**1.46**	-20	-22	-21	-23	-21	-20	-21	-24	-21	-26
	12	**2.18**	-28	-29	-31	-32	-28	-31	-32	-30	-33	-37
80	**6**	**1.09**	-18	-20	-19	-21	-24	-18	-24	-26	-19	-28
	8	**1.46**	-27	-29	-29	-31	-31	-28	-31	-32	-30	-36
	12	**2.18**	-34	-35	-37	-38	-35	-38	-41	-36	-40	-46
100	**6**	**1.09**	-20	-24	-21	-25	-30	-19	-31	-32	-20	-35
	8	**1.46**	-33	-35	-34	-36	-37	-34	-40	-39	-36	-44
	12	**2.18**	-37	-38	-40	-41	-39	-42	-49	-41	-44	-52
120	**6**	**1.09**	-19	-25	-20	-26	-35	-17	-36	-36	-19	-39
	8	**1.46**	-36	-38	-38	-39	-41	-38	-46	-43	-40	-49
	12	**2.18**	-39	-41	-42	-43	-41	-45	-54	-43	-47	-56

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-15	-16	-15	-17	-15	-15	-15	-16	-15	-16
	8	**1.48**	-12	-13	-12	-14	-12	-12	-12	-14	-12	-14
	12	**1.98**	-12	-12	-14	-15	-11	-12	-12	-12	-16	-16
60	**6**	**1.08**	-12	-14	-13	-15	-15	-12	-15	-17	-13	-18
	8	**1.32**	-11	-14	-13	-16	-13	-11	-13	-16	-13	-18
	12	**1.77**	-12	-13	-15	-16	-12	-16	-16	-14	-18	-22
80	**6**	**1.01**	-11	-13	-12	-15	-18	-11	-18	-20	-13	-22
	8	**1.23**	-13	-16	-15	-18	-17	-14	-18	-20	-17	-24
	12	**1.64**	-12	-14	-16	-17	-13	-18	-22	-16	-20	-28
100	**6**	**0.95**	-8	-13	-9	-14	-20	-7	-21	-22	-8	-25
	8	**1.16**	-15	-18	-17	-19	-20	-17	-24	-23	-20	-29
	12	**1.54**	-11	-13	-15	-17	-14	-19	-27	-17	-21	-33
120	**6**	**0.91**	-2	-9	-4	-11	-21	0	-24	-23	-2	-27
	8	**1.10**	-16	-18	-18	-20	-23	-19	-29	-25	-21	-33
	12	**1.47**	-10	-12	-14	-16	-12	-19	-31	-15	-22	-35

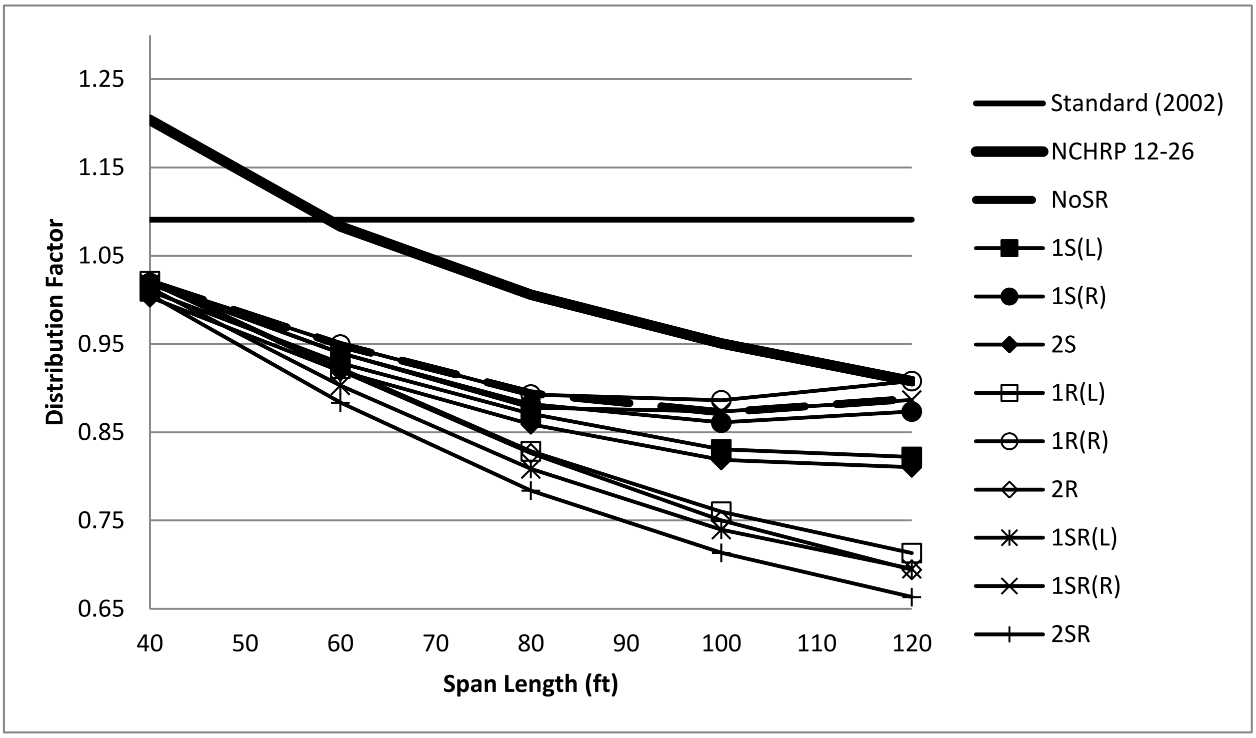


Figure 4.28. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6ft)

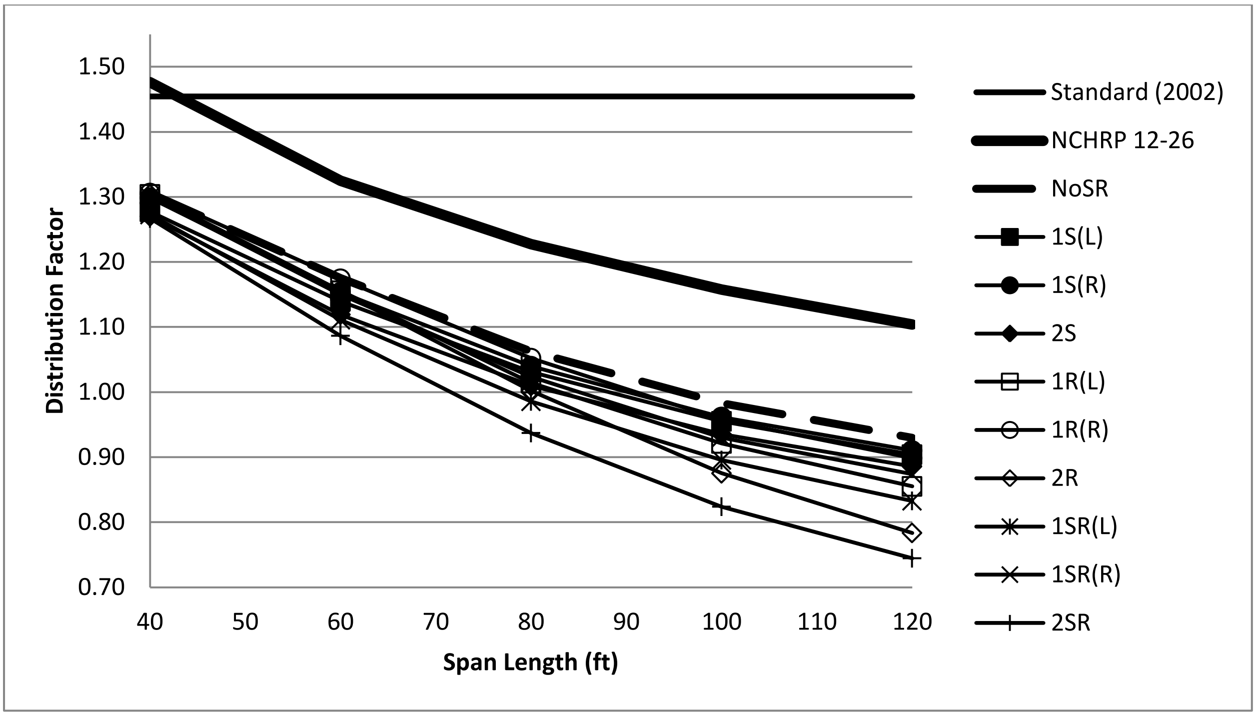


Figure 4.29. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8ft)

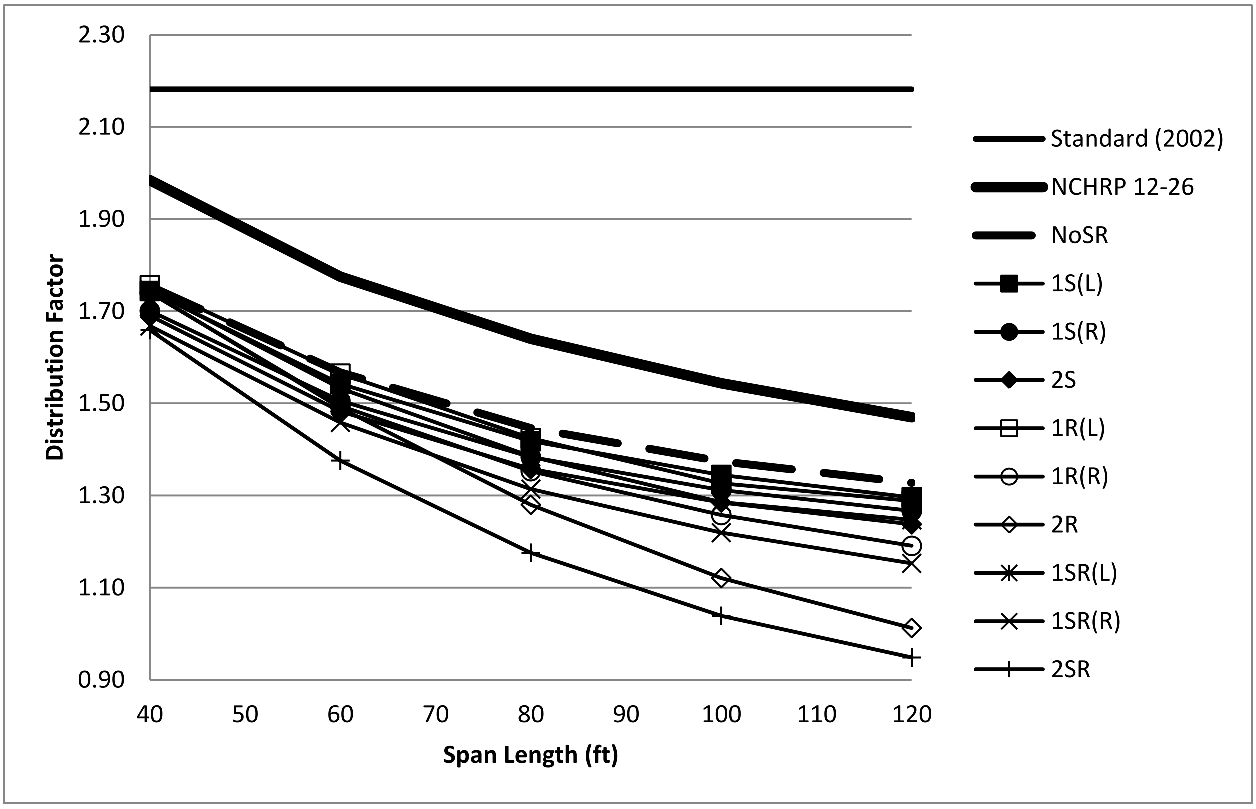


Figure 4.30. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.22 (considering steel beams only) as compared to Table 4.20 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.20 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: the FEA distribution factors are smaller than (1) by about 3 % for girder spacings up to 8 ft while (1) is nearly equal to the FEA distribution factors for spacings between 8 and 12 ft.
2. Sidewalk on one side (left or right): (1) is about 3 % higher than FEA distribution factors for spans up 60 ft and about 6 % higher for spans between 60 and 120 ft with girder spacings up to 8ft and about 2 % higher for spans between 60 and 120 ft with girder spacings between 80 and 120 ft.
3. Sidewalk on both sides: (1) is about 7 % higher than FEA distribution factors for girder spacings up to 8ft and about 4 % higher for girder spacings between 8 and 12 ft.
4. Railing on one side (left or right): for spans up to 80 ft, (1) is about 6 % higher than FEA distribution factors for girder spacings up to 6 ft and about 3 % higher for girder spacings between 6 and 8 ft while (1) is equal to the FEA results for spacings between 8 and 12 ft. For spans between 80 and 120 ft, (1) is about 3 % higher than FEA distribution factors for girder spacings

up to 6 ft and about 10 % higher for girder spacings between 6 and 8 ft and about 4 % higher for girder spacings between 8 and 12 ft.

5. Railing on both sides: for spans up to 40 ft, (1) is about 9 % higher than FEA distribution factors up to a girder spacing of 6ft while (1) is nearly equal to the FEA distribution factors for girder spacings between 6 and 12 ft. Otherwise, (1) is about 5 % higher than FEA distribution factors for spans between 40 and 60 ft and about 10 % higher for spans between 60 and 80 ft and about 17 % higher for spans between 80 and 120 ft.
6. Sidewalk and railing on one side (left or right): for spans up to 80 ft, (1) is about 5 % higher than FEA distribution factors for girder spacings up to 8 ft while (1) is equal to the FEA results for spacings between 8 and 12 ft. For spans between 80 and 120 ft, (1) is about 2 % higher than FEA distribution factors for girder spacings up to 6 ft and about 9 % higher for girder spacings between 6 and 12 ft.
7. Sidewalk and railing on both sides: (1) is about 6 % higher than FEA distribution factors for spans up to 40 ft and about 10% higher for spans between 40 and 60 ft and about 16 % higher for spans between 60 and 80 ft and about 22% higher for spans between 80 and 120 ft.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it is important to note that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a 3 % reduction in distribution factor is observed. Otherwise, the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 5 % when introducing sidewalks on both sides; 0 % for spans between 40 and 80 ft (12 and 24 m) and 0 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 6 ft and 3 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 6 and 12 ft when introducing railings on one side; 3 % for spans between 40 and 60 ft (12 and 18 m) and 7 % for spans between 60 and 80 ft (18 and 24 m) and 13% for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on both sides; 2 % for spans between 40 and 80 ft (12 and 24 m) and 2 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 6ft and 6 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 6 and 12 ft for a combination of sidewalks and railings on one side; and 7 % for spans between 40 and 60 ft (12 and 18 m) and 11 % for spans between 60 and 80 ft (18 and 24 m) and 17 % for spans between 80 and 120 ft (24 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.3.3. Four-Lane Bridges

Tables A.16-A.18 in the appendix show a summary of the positive bending moments calculated in the concrete slab and in steel girders for a two-span 4-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.16 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 17 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 41 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 49 %.

Since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur mostly in either one of the three left side girders. When the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment occurred mostly in the left exterior girder. However, using Tables A.16-A.18 to identify the maximum bending moments at critical sections (usually occurring in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge. It is worth mentioning that all maximum wheel load distributions of the 4-lane bridge cases have been reduced by 25 %, as permitted by the AASHTO Standard Specifications (2002) for 4-lane bridges, in order to account for the improbable situation of having all lanes loaded at the same time and at locations along the bridge deck producing the maximum bending moment in a bridge superstructure. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 12 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides. These maximum bending moments and FEA distribution factors are summarized in Table 4.23 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 4-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.24 for all the bridges.

Table 4.23. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	2671	2661	2671	2661	2689	2671	2689	2672	2672	2672
	8	2,150	3487	3478	3488	3469	3503	3487	3502	3491	3488	3483
	12	2,150	5207	5187	5185	5165	5227	5204	5224	5201	5186	5180
60	**6**	3,877	4654	4606	4633	4586	4658	4654	4658	4571	4637	4554
	8	3,877	6010	5907	5983	5880	5977	6013	5972	5899	5990	5829
	12	3,877	8680	8581	8564	8467	8687	8666	8672	8526	8537	8384
80	**6**	5,630	6405	6319	6343	6260	6275	6408	6279	6133	6344	6074
	8	5,630	8251	8075	8171	7998	7955	8258	7919	7801	8170	7621
	12	5,630	11580	11348	11471	11144	11321	11597	11207	11032	11468	10668
100	**6**	8,027	8723	8490	8642	8411	8203	8748	8186	8018	8655	7882
	8	8,027	11127	10856	10985	10721	10285	11140	10225	10069	11024	9756
	12	8,027	15714	15026	15510	14836	14389	15693	13972	13999	15423	13230
120	**6**	11,208	11998	11520	11881	11389	10609	12109	10510	10387	11962	10117
	8	11,208	15301	14596	15123	14431	13234	15395	13072	12929	15153	12487
	12	11,208	21139	20179	20805	19873	18265	20980	17281	17771	20516	16364

(b) Distribution factor = 0.75*DF = 0.75*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	2,150	0.93	0.93	0.93	0.93	0.94	0.93	0.94	0.93	0.93	0.93
	8	2,150	1.22	1.21	1.22	1.21	1.22	1.22	1.22	1.22	1.22	1.21
	12	2,150	1.82	1.81	1.81	1.80	1.82	1.82	1.82	1.81	1.81	1.81
60	**6**	3,877	0.90	0.89	0.90	0.89	0.90	0.90	0.90	0.88	0.90	0.88
	8	3,877	1.16	1.14	1.16	1.14	1.16	1.16	1.16	1.14	1.16	1.13
	12	3,877	1.68	1.66	1.66	1.64	1.68	1.68	1.68	1.65	1.65	1.62
80	**6**	5,630	0.85	0.84	0.85	0.83	0.84	0.85	0.84	0.82	0.85	0.81
	8	5,630	1.10	1.08	1.09	1.07	1.06	1.10	1.05	1.04	1.09	1.02
	12	5,630	1.54	1.51	1.53	1.48	1.51	1.54	1.49	1.47	1.53	1.42
100	**6**	8,027	0.81	0.79	0.81	0.79	0.77	0.82	0.76	0.75	0.81	0.74
	8	8,027	1.04	1.01	1.03	1.00	0.96	1.04	0.96	0.94	1.03	0.91
	12	8,027	1.47	1.40	1.45	1.39	1.34	1.47	1.31	1.31	1.44	1.24
120	**6**	11,208	0.80	0.77	0.80	0.76	0.71	0.81	0.70	0.70	0.80	0.68
	8	11,208	1.02	0.98	1.01	0.97	0.89	1.03	0.87	0.87	1.01	0.84
	12	11,208	1.41	1.35	1.39	1.33	1.22	1.40	1.16	1.19	1.37	1.10

Table 4.24. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-15	-15	-15	-15	-14	-15	-14	-14	-14	-14
	8	**1.46**	-17	-17	-17	-17	-16	-17	-16	-17	-17	-17
	12	**2.18**	-17	-17	-17	-17	-16	-17	-16	-17	-17	-17
60	**6**	**1.09**	-17	-18	-18	-19	-17	-17	-17	-19	-18	-19
	8	**1.46**	-20	-22	-21	-22	-21	-20	-21	-22	-21	-23
	12	**2.18**	-23	-24	-24	-25	-23	-23	-23	-24	-24	-26
80	**6**	**1.09**	-22	-23	-22	-23	-23	-22	-23	-25	-22	-26
	8	**1.46**	-25	-26	-25	-27	-27	-25	-28	-29	-25	-30
	12	**2.18**	-29	-31	-30	-32	-31	-29	-32	-33	-30	-35
100	**6**	**1.09**	-25	-27	-26	-28	-30	-25	-30	-31	-26	-32
	8	**1.46**	-29	-31	-30	-31	-34	-29	-35	-36	-29	-38
	12	**2.18**	-33	-36	-34	-36	-38	-33	-40	-40	-34	-43
120	**6**	**1.09**	-26	-29	-27	-30	-35	-26	-35	-36	-27	-38
	8	**1.46**	-30	-33	-31	-34	-39	-29	-40	-41	-31	-43
	12	**2.18**	-35	-38	-36	-39	-44	-36	-47	-45	-37	-50

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-23	-23	-23	-23	-22	-23	-22	-23	-23	-23
	8	**1.48**	-18	-18	-18	-18	-17	-18	-17	-18	-18	-18
	12	**1.98**	-8	-9	-9	-9	-8	-9	-8	-9	-9	-9
60	**6**	**1.08**	-17	-18	-17	-18	-17	-17	-17	-18	-17	-19
	8	**1.32**	-12	-14	-13	-14	-13	-12	-13	-14	-13	-15
	12	**1.77**	-5	-6	-7	-8	-5	-6	-5	-7	-7	-9
80	**6**	**1.01**	-15	-16	-16	-17	-17	-15	-17	-19	-16	-20
	8	**1.23**	-10	-12	-11	-13	-14	-10	-14	-15	-11	-17
	12	**1.64**	-6	-8	-7	-9	-8	-6	-9	-10	-7	-13
100	**6**	**0.95**	-14	-17	-15	-17	-19	-14	-20	-21	-15	-23
	8	**1.16**	-10	-12	-11	-13	-17	-10	-17	-19	-11	-21
	12	**1.54**	-5	-9	-6	-10	-13	-5	-15	-15	-7	-20
120	**6**	**0.91**	-12	-15	-12	-16	-22	-11	-23	-23	-12	-25
	8	**1.10**	-7	-12	-8	-13	-20	-7	-21	-22	-8	-24
	12	**1.47**	-4	-8	-5	-10	-17	-4	-21	-19	-7	-25

Figures 4.31-4.33 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 4-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.25. It should be noted that Table 4.25 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.26 for all the bridges. Figures 4.34-4.36 show a trend similar to Figures 4.31-4.33, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

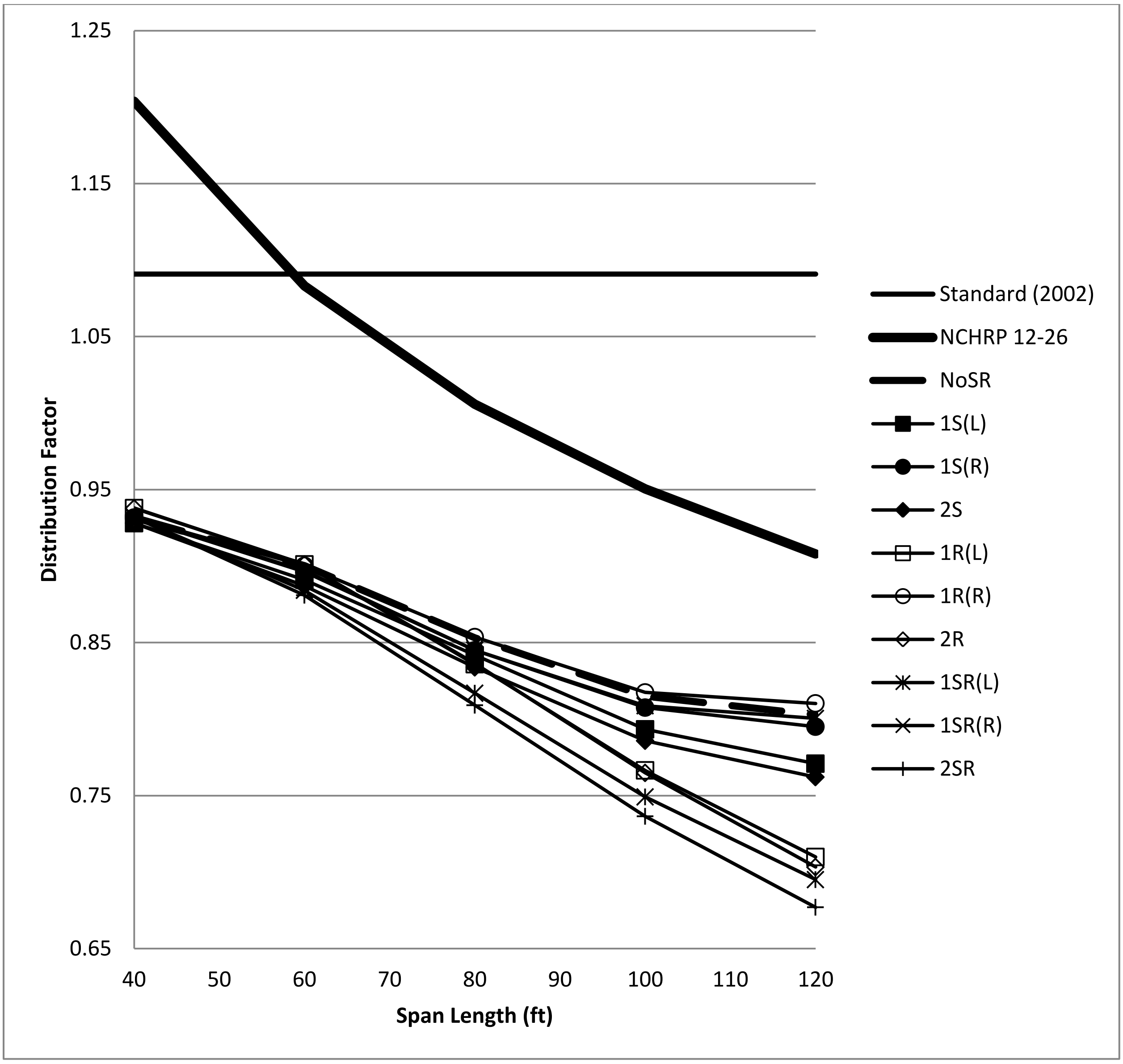


Figure 4.31. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

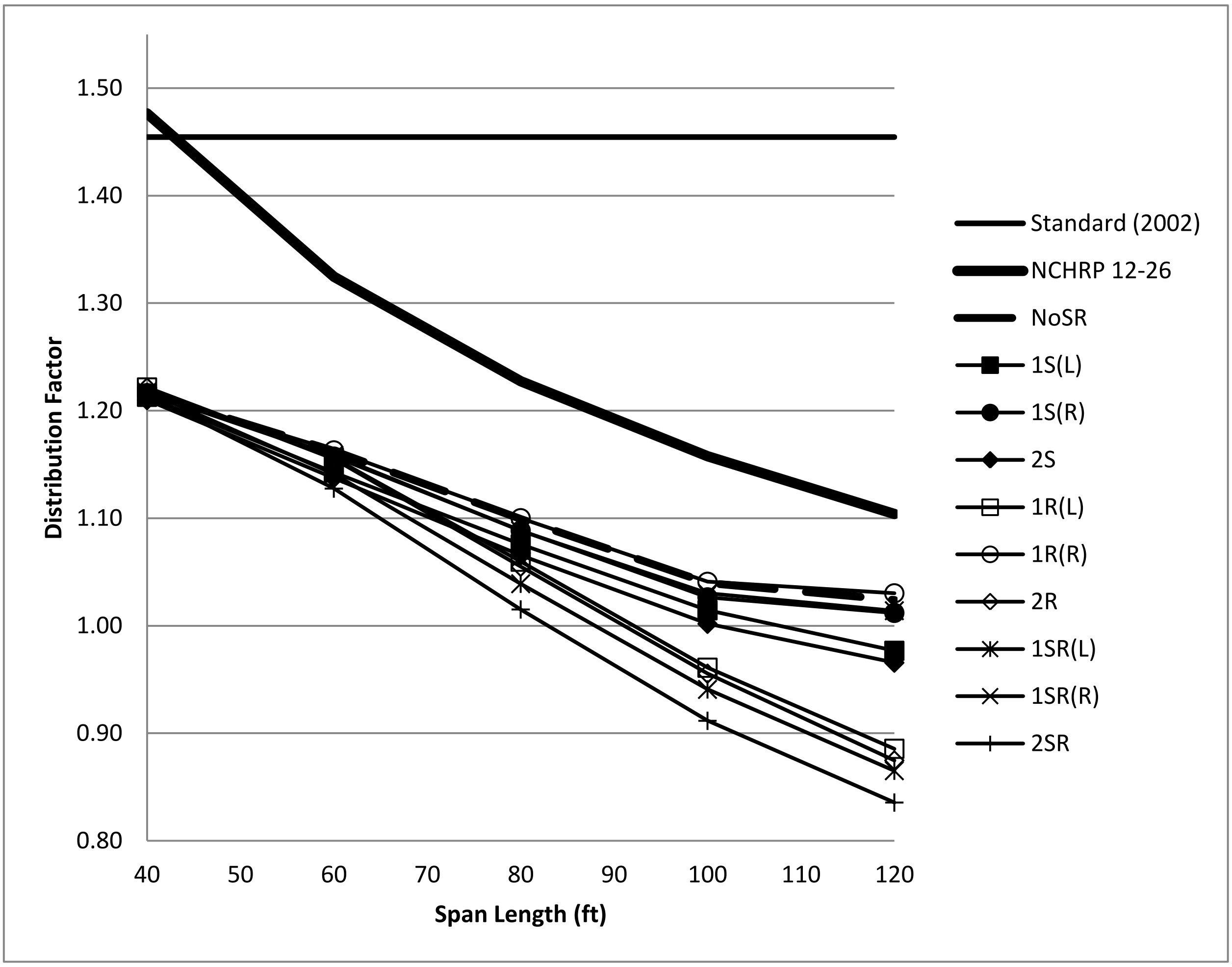


Figure 4.32. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

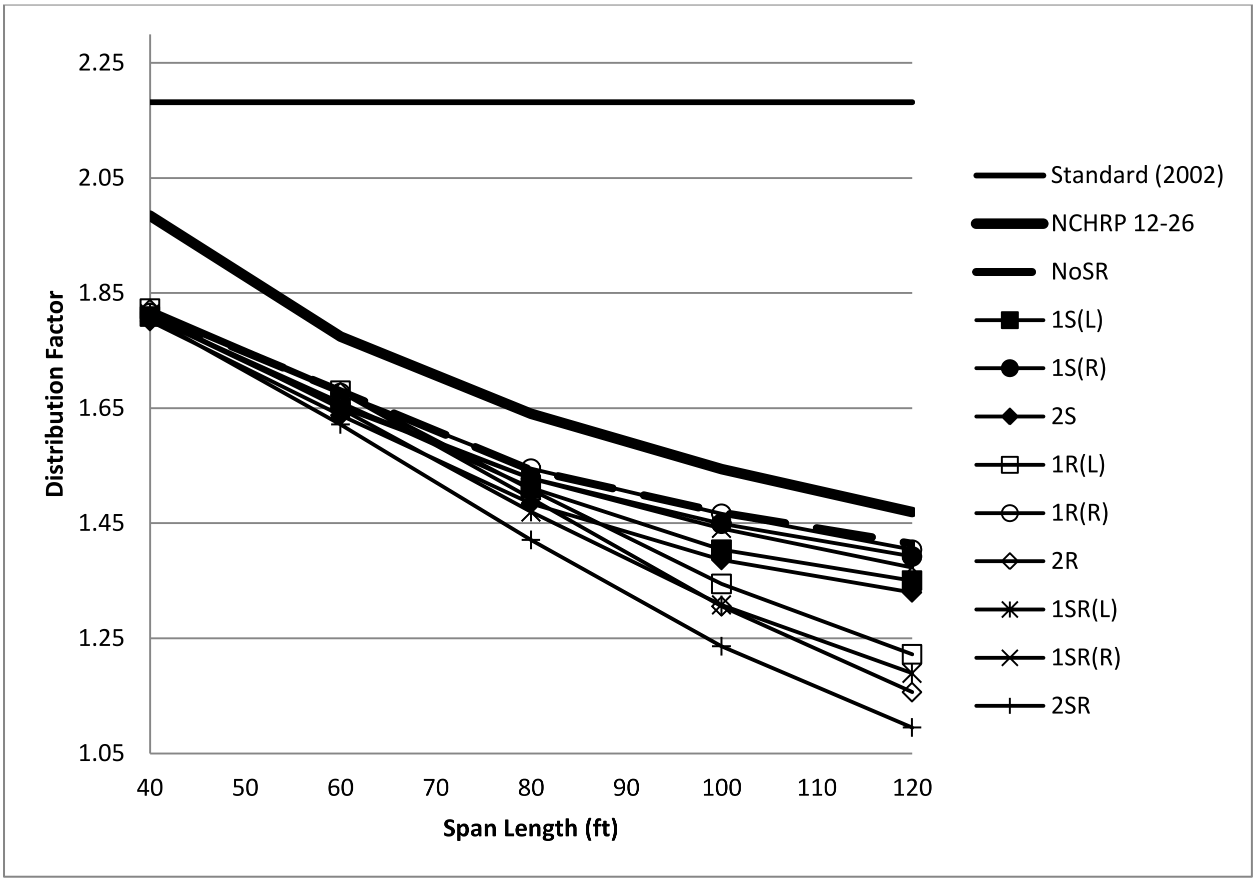


Figure 4.33. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.25. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,150	2502	2491	2502	2491	2518	2501	2517	2500	2502	2500
	8	2,150	3143	3132	3144	3124	3155	3143	3155	3144	3144	3136
	12	2,150	4467	4448	4446	4426	4485	4464	4482	4458	4447	4437
60	6	3,877	4385	4338	4364	4318	4381	4385	4381	4296	4368	4279
	8	3,877	5503	5404	5476	5378	5457	5505	5451	5381	5482	5312
	12	3,877	7606	7511	7500	7406	7598	7590	7581	7445	7470	7309
80	6	5,630	6045	5961	5986	5903	5900	6047	5902	5765	5985	5706
	8	5,630	7598	7429	7522	7355	7280	7602	7240	7133	7517	6957
	12	5,630	10334	10002	10231	9818	9940	10343	9818	9674	10221	9324
100	6	8,027	8305	8074	8228	8000	7751	8327	7727	7576	8237	7439
	8	8,027	10397	10086	10307	9959	9487	10450	9423	9284	10340	8988
	12	8,027	14217	13571	14029	13395	12775	14181	12348	12421	13932	11681
120	6	11,208	11461	10972	11369	10847	10029	11639	9919	9820	11525	9550
	8	11,208	14401	13716	14233	13559	12245	14575	12066	11951	14390	11526
	12	11,208	19223	18308	18928	18025	16285	19473	15425	15840	19137	14539

(b) Distribution factor = 0.75*DF = 0.75*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	2,150	0.87	0.87	0.87	0.87	0.88	0.87	0.88	0.87	0.87	0.87
	8	2,150	1.10	1.09	1.10	1.09	1.10	1.10	1.10	1.10	1.10	1.09
	12	2,150	1.56	1.55	1.55	1.54	1.56	1.56	1.56	1.56	1.55	1.55
60	6	3,877	0.85	0.84	0.84	0.84	0.85	0.85	0.85	0.83	0.85	0.83
	8	3,877	1.06	1.05	1.06	1.04	1.06	1.06	1.05	1.04	1.06	1.03
	12	3,877	1.47	1.45	1.45	1.43	1.47	1.47	1.47	1.44	1.45	1.41
80	6	5,630	0.81	0.79	0.80	0.79	0.79	0.81	0.79	0.77	0.80	0.76
	8	5,630	1.01	0.99	1.00	0.98	0.97	1.01	0.96	0.95	1.00	0.93
	12	5,630	1.38	1.33	1.36	1.31	1.32	1.38	1.31	1.29	1.36	1.24
100	6	8,027	0.78	0.75	0.77	0.75	0.72	0.78	0.72	0.71	0.77	0.70
	8	8,027	0.97	0.94	0.96	0.93	0.89	0.98	0.88	0.87	0.97	0.84
	12	8,027	1.33	1.27	1.31	1.25	1.19	1.32	1.15	1.16	1.30	1.09
120	6	11,208	0.77	0.73	0.76	0.73	0.67	0.78	0.66	0.66	0.77	0.64
	8	11,208	0.96	0.92	0.95	0.91	0.82	0.98	0.81	0.80	0.96	0.77
	12	11,208	1.29	1.23	1.27	1.21	1.09	1.30	1.03	1.06	1.28	0.97

Table 4.26. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-20	-20	-20	-20	-19	-20	-19	-20	-20	-20
	8	**1.46**	-25	-25	-25	-25	-25	-25	-25	-25	-25	-25
	12	**2.18**	-29	-29	-29	-29	-28	-29	-28	-29	-29	-29
60	**6**	**1.09**	-22	-23	-23	-23	-22	-22	-22	-24	-22	-24
	8	**1.46**	-27	-28	-27	-29	-28	-27	-28	-29	-27	-30
	12	**2.18**	-33	-33	-33	-34	-33	-33	-33	-34	-34	-35
80	**6**	**1.09**	-26	-27	-27	-28	-28	-26	-28	-30	-27	-30
	8	**1.46**	-31	-32	-31	-33	-34	-31	-34	-35	-31	-37
	12	**2.18**	-37	-39	-37	-40	-39	-37	-40	-41	-38	-43
100	**6**	**1.09**	-29	-31	-29	-31	-34	-29	-34	-35	-29	-36
	8	**1.46**	-33	-35	-34	-36	-39	-33	-40	-41	-34	-42
	12	**2.18**	-39	-42	-40	-43	-45	-39	-47	-47	-40	-50
120	**6**	**1.09**	-30	-33	-30	-33	-38	-29	-39	-40	-29	-41
	8	**1.46**	-34	-37	-35	-38	-44	-33	-45	-45	-34	-47
	12	**2.18**	-41	-44	-42	-45	-50	-40	-53	-51	-41	-55

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-28	-28	-28	-28	-27	-28	-27	-28	-27	-28
	8	**1.48**	-26	-26	-26	-26	-25	-26	-25	-26	-26	-26
	12	**1.98**	-21	-22	-22	-22	-21	-22	-21	-22	-22	-22
60	**6**	**1.08**	-22	-23	-22	-23	-22	-22	-22	-23	-22	-24
	8	**1.32**	-20	-21	-20	-21	-20	-20	-20	-21	-20	-22
	12	**1.77**	-17	-18	-18	-19	-17	-17	-17	-19	-19	-20
80	**6**	**1.01**	-20	-21	-21	-22	-22	-20	-22	-24	-21	-24
	8	**1.23**	-18	-19	-18	-20	-21	-17	-21	-23	-18	-24
	12	**1.64**	-16	-19	-17	-20	-19	-16	-20	-21	-17	-24
100	**6**	**0.95**	-18	-21	-19	-21	-24	-18	-24	-26	-19	-27
	8	**1.16**	-16	-19	-17	-20	-23	-16	-24	-25	-17	-27
	12	**1.54**	-14	-18	-15	-19	-23	-14	-25	-25	-16	-29
120	**6**	**0.91**	-16	-19	-16	-20	-26	-14	-27	-28	-15	-30
	8	**1.10**	-13	-17	-14	-18	-26	-12	-27	-28	-13	-30
	12	**1.47**	-12	-17	-14	-18	-26	-11	-30	-28	-13	-34

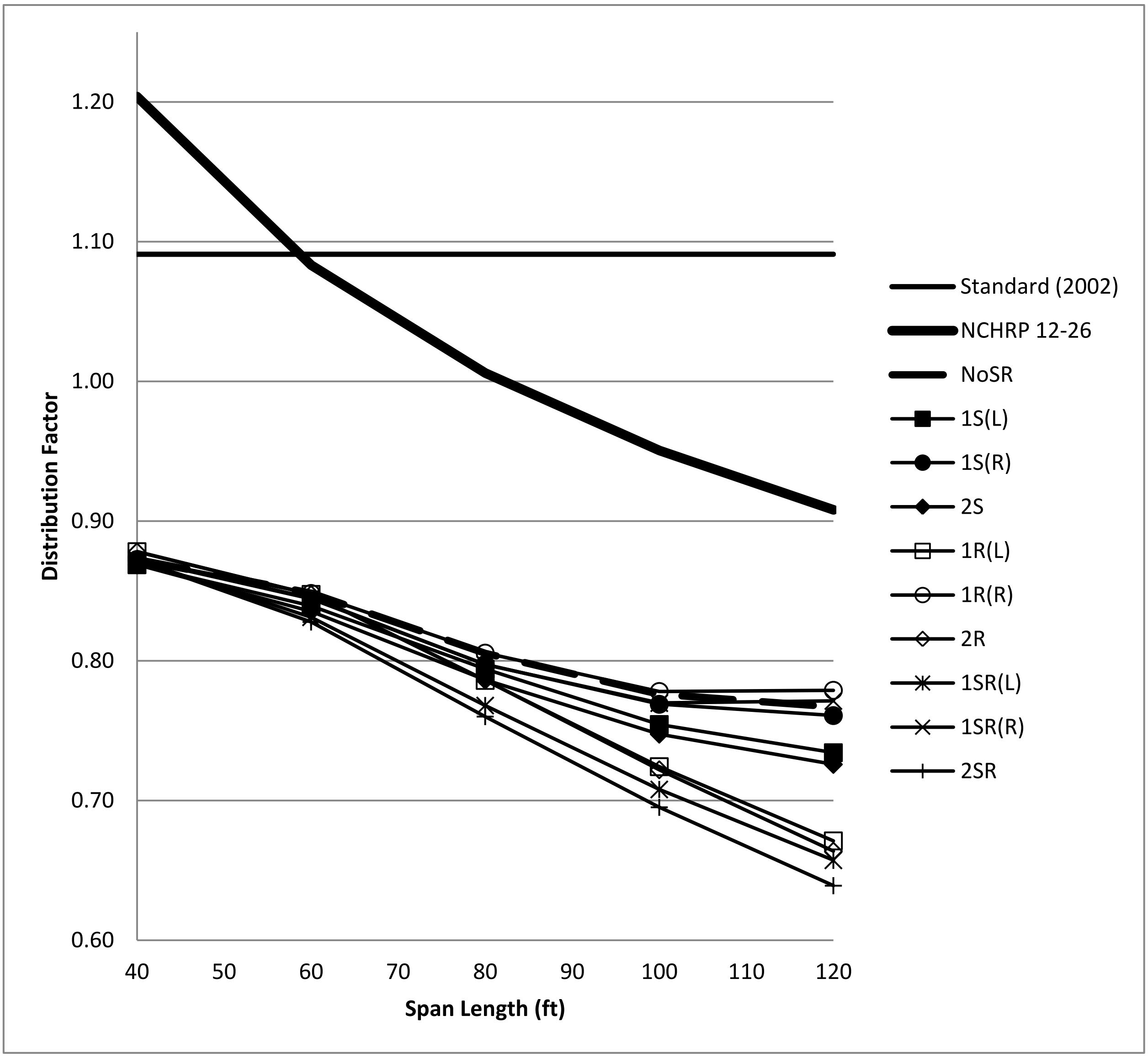


Figure 4.34. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6ft)

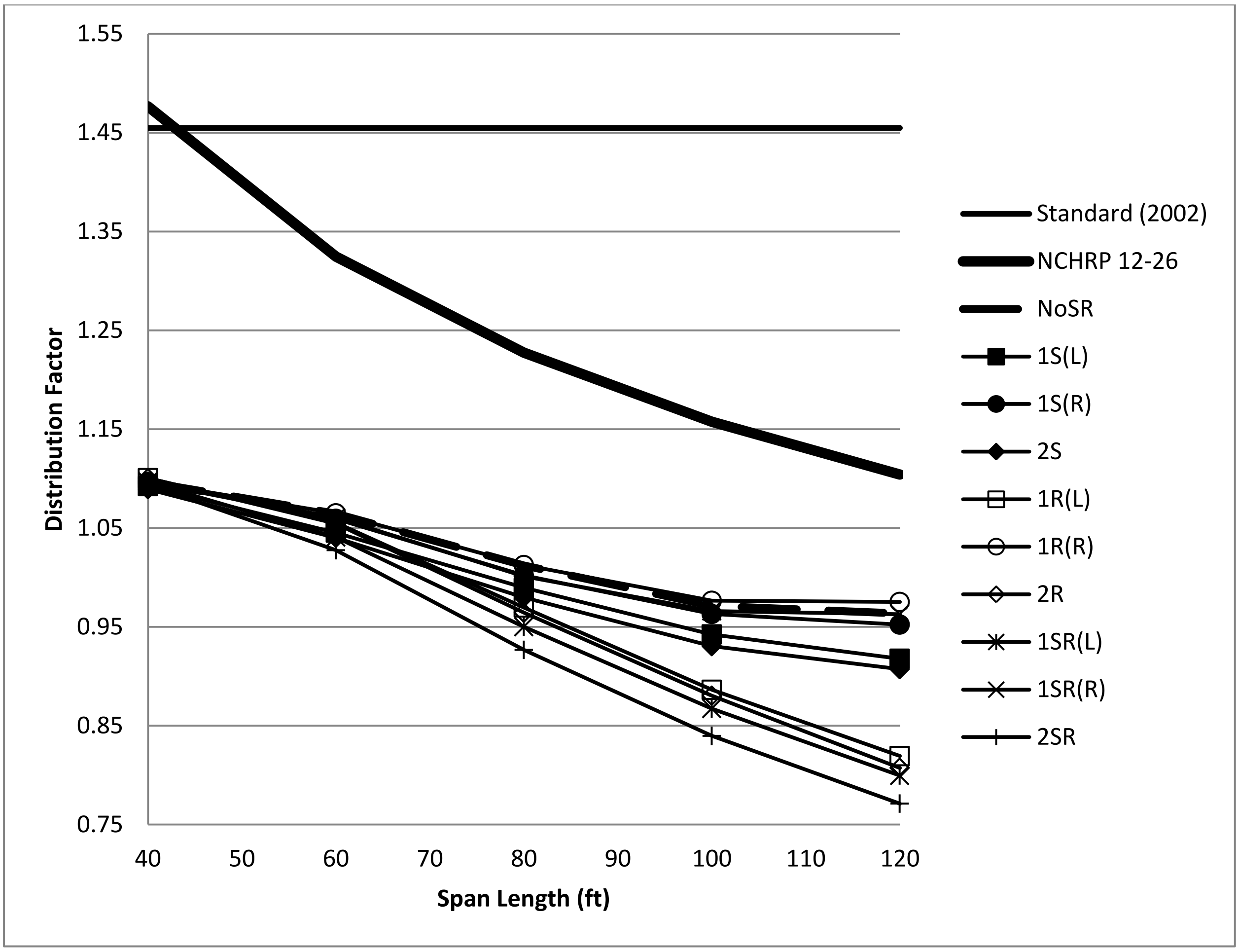


Figure 4.35. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8ft)

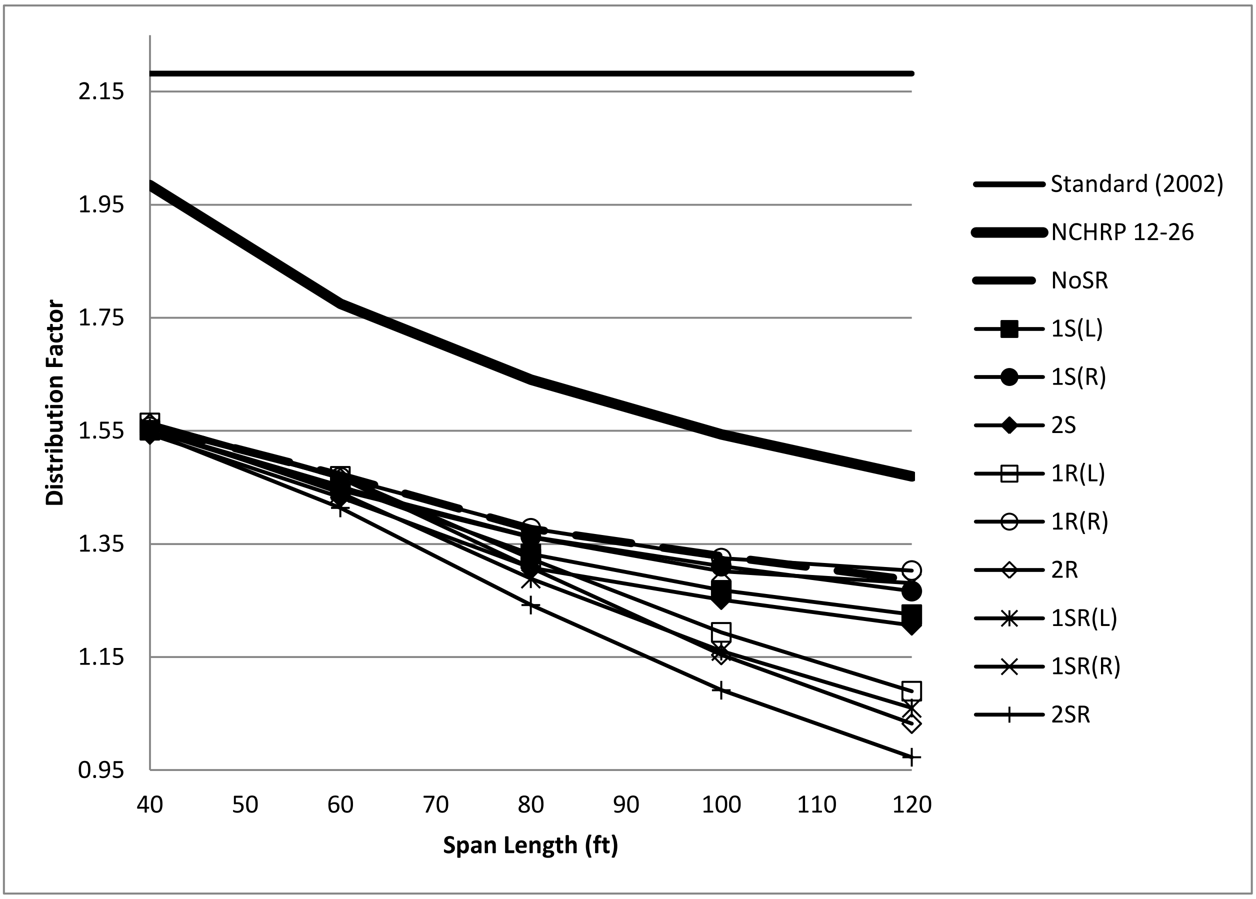


Figure 4.36. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.26 (considering steel beams only) as compared to Table 4.24 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.24 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: for spans up to 40 ft, (1) is about 18 % higher than FEA distribution factors for girder spacings up to 8 ft and about 8 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 10 % higher than FEA distribution factors for girder spacings up to 8 ft and about 4 % higher for girder spacings between 8 and 12 ft.
2. Sidewalk on one side (left or right): for spans up to 40 ft, (1) is about 18 % higher than FEA distribution factors for girder spacings up to 8 ft and about 9 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 11 % higher than FEA distribution factors for girder spacings up to 8 ft and about 5 % higher for girder spacings between 8 and 12 ft.
3. Sidewalk on both sides: for spans up to 40 ft, (1) is about 18 % higher than FEA distribution factors for girder spacings up to 8 ft and about 9 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 13 % higher than FEA distribution factors for girder spacings up to 8 ft and about 8 % higher for girder spacings between 8 and 12 ft.

4. Railing on one side (left or right): for spans up to 40 ft, (1) is about 18 % higher than FEA distribution factors for girder spacings up to 8 ft and about 8 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 10 % higher than FEA distribution factors for girder spacings up to 8 ft and about 5 % higher for girder spacings between 8 and 12 ft.
5. Railing on both sides: for spans up to 40 ft, (1) is about 18 % higher than FEA distribution factors for girder spacings up to 8 ft and about 8 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 80 ft, (1) is about 13 % higher than FEA distribution factors for girder spacings up to 8 ft and about 5 % higher for girder spacings between 8 and 12 ft. For spans between 80 and 120 ft, (1) is about 17 % higher than FEA distribution factors for all girder spacings.
6. Sidewalk and railing on one side (left or right): for spans up to 40 ft, (1) is about 18 % higher than FEA distribution factors for girder spacings up to 8 ft and about 9 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 11 % higher than FEA distribution factors for girder spacings up to 8 ft and about 7 % higher for girder spacings between 8 and 12 ft.
7. Sidewalk and railing on both sides: for spans up to 80 ft, (1) is about 18 % higher than FEA distribution factors for girder spacings up to 8 ft and about 9 % higher for girder spacings between 8 and 12 ft. For spans between 80 and 120 ft, (1) is about 21 % higher than FEA distribution factors for all girder spacings.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it's important to note that for spans up to 60 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a 2 % reduction in distribution factor is observed. Otherwise (for spans between 60 and 120 ft), the average reductions in FEA distribution factors when compared to the base case were 1 % when introducing sidewalks on one side; 3 % for spans between 60 and 80 ft (18 and 24 m) and 5 % for spans between 80 and 120 ft (24 and 36 m) when introducing sidewalks on both sides; 0 % when introducing railings on one side; 3 % for spans between 60 and 80 ft (18 and 24 m) and 8 % for spans between 80 and 100 ft (24 m and 30 m) and 13 % for spans between 100 and 120 ft (30 and 36 m) when introducing railings on both sides; 1 % for a combination of sidewalks and railings on one side; and 6 % for spans between 60 and 80 ft (18 and 24 m) and 12 % for spans between 80 and 100 ft (24 and 30 m) and 18 % for spans between 100 and 120 ft (30 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.4. Two-Span Bridges - Negative Moments

4.4.1. Two-Lane Bridges

Tables A.19-A.21 in the appendix show a summary of the negative bending moments calculated in the concrete slab and in steel girders for a two-span 2-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.19 shows that the contribution of bending moment from the concrete slab is about 5% when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 18 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 34 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 44 %.

Since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will mostly occur in either one of the two left side girders. When the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment occurred in the left exterior girder. However, using Tables A.19-A.21 to identify the maximum bending moments at critical sections (usually occurring in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 9 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides. These maximum negative bending moments and FEA distribution factors are summarized in Table 4.27 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 2-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.28 for all the bridges.

Table 4.27. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	2147	2123	2123	2088	2166	2143	2162	2127	2112	2091
	8	1,874	2828	2778	2801	2751	2777	2827	2775	2693	2805	2670
	12	1,874	4168	4130	4129	4092	4146	4160	4138	4070	4104	4007
60	**6**	4,394	4475	4316	4419	4260	4332	4487	4301	4175	4427	4012
	8	4,394	5804	5607	5681	5487	5333	5800	5324	5129	5653	4975
	12	4,394	8027	7776	7848	7599	7647	7842	7437	7331	7600	6890
80	**6**	6,545	6271	5964	6172	5867	5691	6279	5523	5504	6152	5179
	8	6,545	7948	7590	7766	7414	6893	7873	6764	6680	7637	6340
	12	6,545	10731	10319	10478	10066	9753	10183	9084	9397	9905	8494
100	**6**	9,936	9471	8877	9303	8722	8102	9437	7607	7872	9200	7205
	8	9,936	11786	11110	11509	10845	9762	11516	9245	9466	11151	8698
	12	9,936	15627	14901	15266	14506	13524	14347	11934	13105	14069	11282
120	**6**	14,432	13476	12504	13201	12267	11145	13334	10182	10854	12939	9701
	8	14,432	16634	15534	16221	15146	13617	16040	12351	13220	15504	11598
	12	14,432	21895	20715	21369	20106	18335	19602	15545	17790	19312	14709

(b) Distribution factor = DF = Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	1.15	1.13	1.13	1.11	1.16	1.14	1.15	1.13	1.13	1.12
	8	1,874	1.51	1.48	1.49	1.47	1.48	1.51	1.48	1.44	1.50	1.42
	12	1,874	2.22	2.20	2.20	2.18	2.21	2.22	2.21	2.17	2.19	2.14
60	**6**	4,394	1.02	0.98	1.01	0.97	0.99	1.02	0.98	0.95	1.01	0.91
	8	4,394	1.32	1.28	1.29	1.25	1.21	1.32	1.21	1.17	1.29	1.13
	12	4,394	1.83	1.77	1.79	1.73	1.74	1.78	1.69	1.67	1.73	1.57
80	**6**	6,545	0.96	0.91	0.94	0.90	0.87	0.96	0.84	0.84	0.94	0.79
	8	6,545	1.21	1.16	1.19	1.13	1.05	1.20	1.03	1.02	1.17	0.97
	12	6,545	1.64	1.58	1.60	1.54	1.49	1.56	1.39	1.44	1.51	1.30
100	**6**	9,936	0.95	0.89	0.94	0.88	0.82	0.95	0.77	0.79	0.93	0.73
	8	9,936	1.19	1.12	1.16	1.09	0.98	1.16	0.93	0.95	1.12	0.88
	12	9,936	1.57	1.50	1.54	1.46	1.36	1.44	1.20	1.32	1.42	1.14
120	**6**	14,432	0.93	0.87	0.91	0.85	0.77	0.92	0.71	0.75	0.90	0.67
	8	14,432	1.15	1.08	1.12	1.05	0.94	1.11	0.86	0.92	1.07	0.80
	12	14,432	1.52	1.44	1.48	1.39	1.27	1.36	1.08	1.23	1.34	1.02

Table 4.28. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	5	4	4	2	6	5	6	4	3	2
	8	**1.46**	3	2	2	1	1	3	1	-2	3	-2
	12	**2.18**	2	1	1	0	1	2	1	0	0	-2
60	**6**	**1.09**	-7	-10	-8	-11	-10	-6	-10	-13	-8	-16
	8	**1.46**	-10	-13	-11	-14	-17	-10	-17	-20	-12	-22
	12	**2.18**	-16	-19	-18	-21	-20	-18	-22	-23	-21	-28
80	**6**	**1.09**	-12	-16	-13	-18	-20	-12	-23	-23	-14	-27
	8	**1.46**	-17	-21	-19	-22	-28	-18	-29	-30	-20	-34
	12	**2.18**	-25	-28	-27	-29	-32	-29	-36	-34	-31	-40
100	**6**	**1.09**	-13	-18	-14	-19	-25	-13	-30	-27	-15	-33
	8	**1.46**	-19	-23	-21	-25	-33	-21	-36	-35	-23	-40
	12	**2.18**	-28	-31	-30	-33	-38	-34	-45	-39	-35	-48
120	**6**	**1.09**	-14	-21	-16	-22	-29	-15	-35	-31	-18	-38
	8	**1.46**	-21	-26	-23	-28	-35	-24	-41	-37	-26	-45
	12	**2.18**	-30	-34	-32	-36	-42	-38	-51	-43	-39	-53

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-5	-6	-6	-7	-4	-5	-4	-6	-6	-7
	8	**1.48**	2	0	1	-1	0	2	0	-3	1	-4
	12	**1.98**	12	11	11	10	11	12	11	9	10	8
60	**6**	**1.08**	-6	-9	-7	-10	-9	-6	-10	-12	-7	-16
	8	**1.32**	0	-4	-2	-6	-8	0	-9	-12	-3	-15
	12	**1.77**	3	0	1	-3	-2	1	-5	-6	-3	-12
80	**6**	**1.01**	-5	-9	-6	-11	-14	-5	-16	-16	-7	-21
	8	**1.23**	-1	-6	-3	-8	-14	-2	-16	-17	-5	-21
	12	**1.64**	0	-4	-2	-6	-9	-5	-15	-12	-8	-21
100	**6**	**0.95**	0	-6	-1	-8	-14	0	-19	-17	-3	-24
	8	**1.16**	2	-3	0	-6	-15	0	-20	-18	-3	-24
	12	**1.54**	2	-3	0	-5	-12	-6	-22	-15	-8	-26
120	**6**	**0.91**	3	-5	1	-6	-15	2	-22	-17	-1	-26
	8	**1.10**	4	-3	2	-5	-15	1	-22	-17	-3	-27
	12	**1.47**	3	-2	1	-5	-14	-8	-27	-16	-9	-31

Figures 4.37-4.39 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 2-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.29. It should be noted that Table 4.29 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.30 for all the bridges. Figures 4.40-4.42 show a trend similar to Figures 4.37-4.39, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

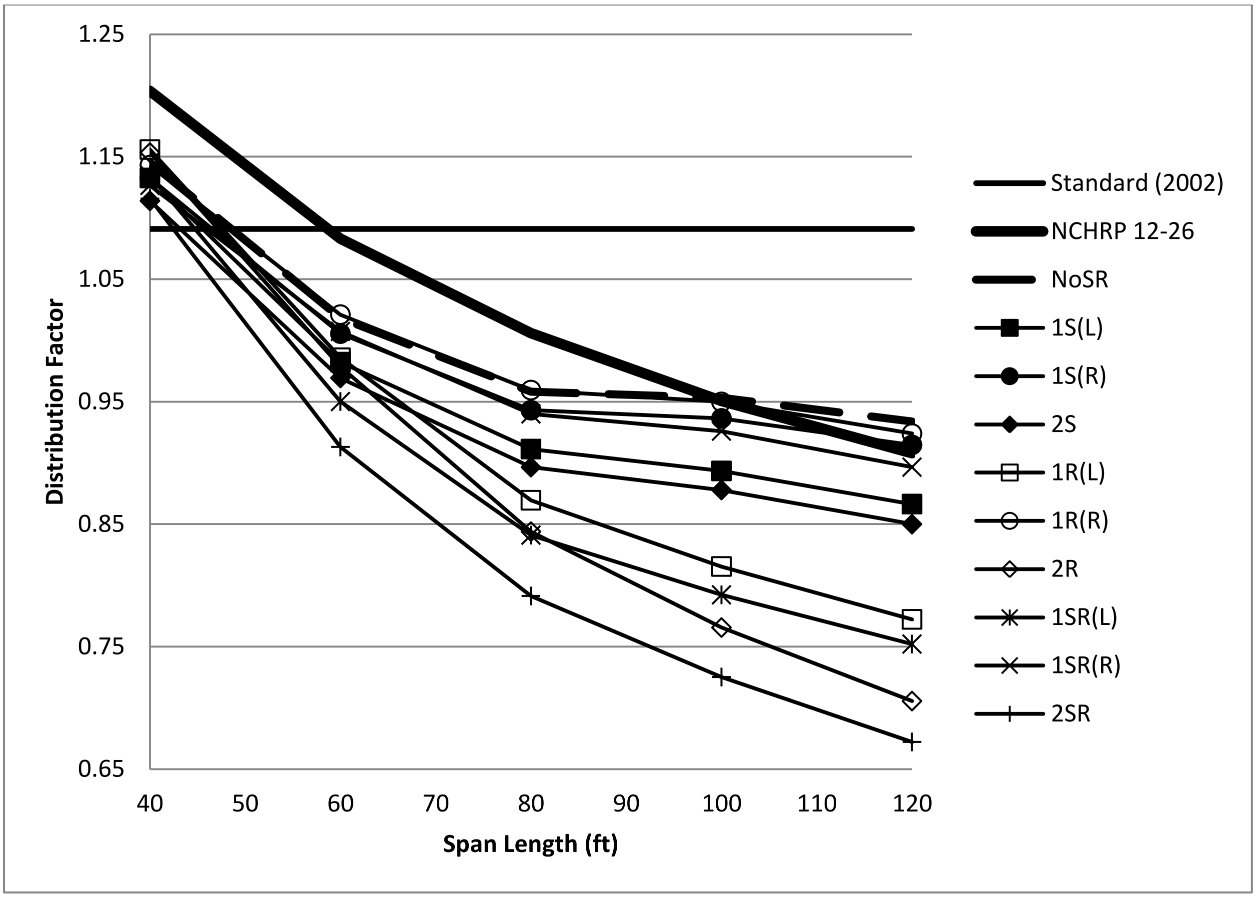


Figure 4.37. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

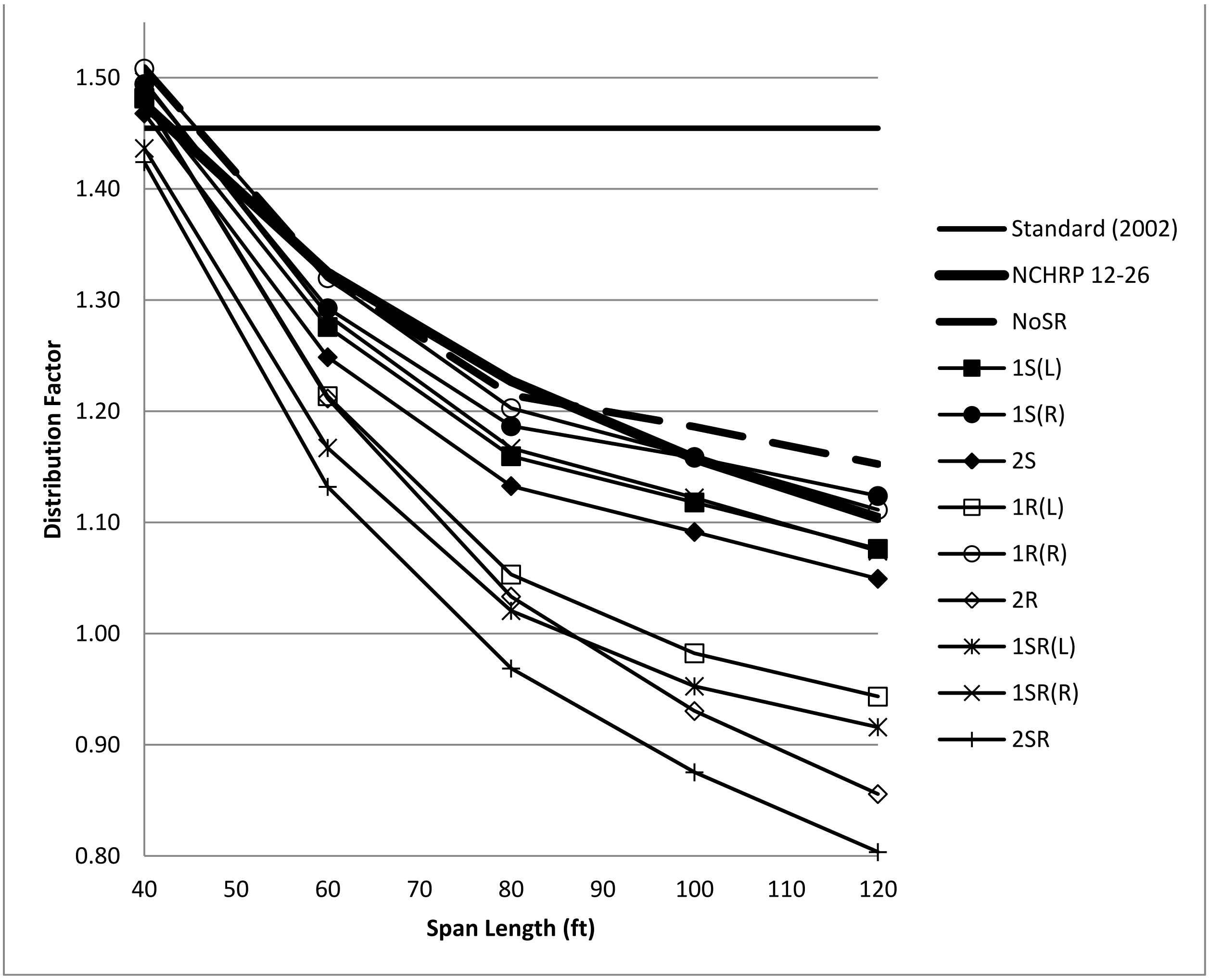


Figure 4.38. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

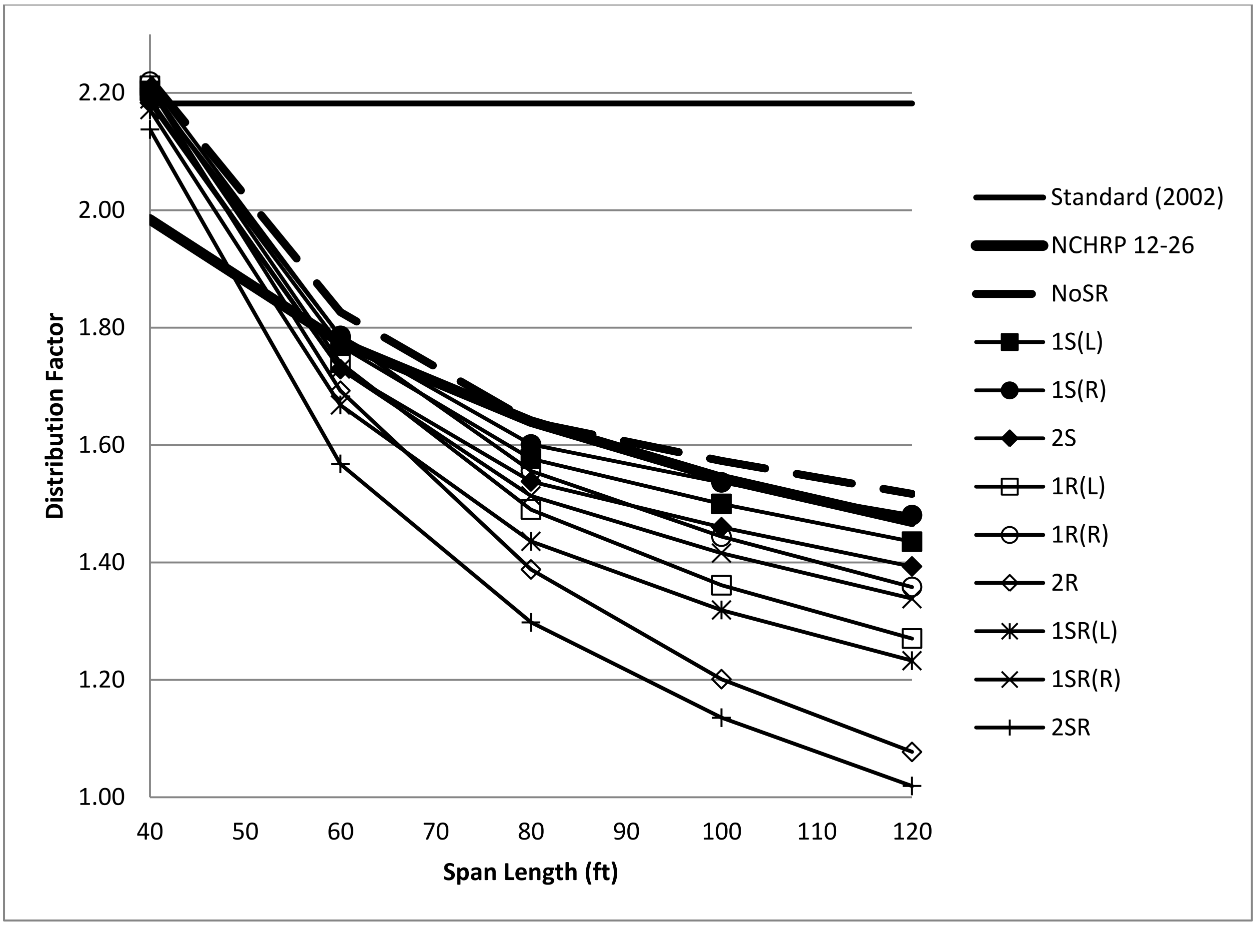


Figure 4.39. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.29. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	2020	1997	1997	1965	2038	2016	2034	2002	1987	1969
	8	1,874	2627	2580	2603	2556	2582	2626	2581	2502	2607	2481
	12	1,874	3795	3761	3759	3725	3779	3789	3772	3708	3737	3650
60	**6**	4,394	4231	4082	4178	4028	4098	4242	4069	3950	4185	3796
	8	4,394	5420	5236	5306	5125	4987	5416	4979	4791	5280	4649
	12	4,394	7338	7108	7170	6943	6998	7171	6811	6705	6945	6300
80	**6**	6,545	5934	5646	5840	5555	5384	5942	5225	5208	5821	4900
	8	6,545	7424	7092	7254	6928	6445	7353	6323	6241	7132	5924
	12	6,545	9799	9425	9563	9190	8913	9301	8306	8583	9039	7757
100	**6**	9,936	9005	8446	8845	8298	7704	9012	7233	7485	8807	6851
	8	9,936	11066	10437	10807	10190	9164	10935	8693	8887	10650	8175
	12	9,936	14343	13681	14004	13313	12425	13955	10974	12033	13500	10362
120	**6**	14,432	12824	11909	12562	11683	10605	13252	10166	10328	12789	9229
	8	14,432	15629	14606	15241	14242	12790	16018	11806	12419	15403	10909
	12	14,432	20097	19023	19605	18459	16841	20310	14289	16333	19402	13511

(b) Distribution factor = DF = Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	1.08	1.07	1.07	1.05	1.09	1.08	1.09	1.07	1.06	1.05
	8	1,874	1.40	1.38	1.39	1.36	1.38	1.40	1.38	1.34	1.39	1.32
	12	1,874	2.03	2.01	2.01	1.99	2.02	2.02	2.01	1.98	1.99	1.95
60	**6**	4,394	0.96	0.93	0.95	0.92	0.93	0.97	0.93	0.90	0.95	0.86
	8	4,394	1.23	1.19	1.21	1.17	1.14	1.23	1.13	1.09	1.20	1.06
	12	4,394	1.67	1.62	1.63	1.58	1.59	1.63	1.55	1.53	1.58	1.43
80	**6**	6,545	0.91	0.86	0.89	0.85	0.82	0.91	0.80	0.80	0.89	0.75
	8	6,545	1.13	1.08	1.11	1.06	0.98	1.12	0.97	0.95	1.09	0.91
	12	6,545	1.50	1.44	1.46	1.40	1.36	1.42	1.27	1.31	1.38	1.19
100	**6**	9,936	0.91	0.85	0.89	0.84	0.78	0.91	0.73	0.75	0.89	0.69
	8	9,936	1.11	1.05	1.09	1.03	0.92	1.10	0.87	0.89	1.07	0.82
	12	9,936	1.44	1.38	1.41	1.34	1.25	1.40	1.10	1.21	1.36	1.04
120	**6**	14,432	0.89	0.83	0.87	0.81	0.73	0.92	0.70	0.72	0.89	0.64
	8	14,432	1.08	1.01	1.06	0.99	0.89	1.11	0.82	0.86	1.07	0.76
	12	14,432	1.39	1.32	1.36	1.28	1.17	1.41	0.99	1.13	1.34	0.94

Table 4.30. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1.09	-1	-2	-2	-4	0	-1	0	-2	-3	-4
	8	1.46	-4	-6	-5	-7	-6	-4	-6	-9	-5	-9
	12	2.18	-7	-8	-8	-9	-7	-7	-8	-9	-9	-11
60	6	1.09	-12	-15	-13	-16	-14	-11	-15	-18	-13	-21
	8	1.46	-16	-18	-17	-20	-22	-16	-22	-25	-18	-28
	12	2.18	-23	-26	-25	-28	-27	-25	-29	-30	-27	-34
80	6	1.09	-17	-21	-18	-22	-25	-17	-27	-27	-18	-31
	8	1.46	-22	-26	-24	-27	-33	-23	-34	-35	-25	-38
	12	2.18	-31	-34	-33	-36	-38	-35	-42	-40	-37	-46
100	6	1.09	-17	-22	-18	-23	-29	-17	-33	-31	-19	-37
	8	1.46	-24	-28	-26	-30	-37	-25	-40	-39	-27	-44
	12	2.18	-34	-37	-35	-39	-43	-36	-49	-44	-38	-52
120	6	1.09	-18	-24	-20	-26	-33	-16	-35	-34	-19	-41
	8	1.46	-26	-31	-28	-32	-39	-24	-44	-41	-27	-48
	12	2.18	-36	-40	-38	-41	-46	-35	-55	-48	-38	-57

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1.20	-10	-11	-11	-13	-10	-11	-10	-11	-12	-13
	8	1.48	-5	-7	-6	-8	-7	-5	-7	-10	-6	-10
	12	1.98	2	1	1	0	2	2	1	0	0	-2
60	6	1.08	-11	-14	-12	-15	-14	-11	-14	-17	-12	-20
	8	1.32	-7	-10	-9	-12	-14	-7	-14	-18	-9	-20
	12	1.77	-6	-9	-8	-11	-10	-8	-13	-14	-11	-19
80	6	1.01	-10	-14	-11	-16	-18	-10	-21	-21	-12	-26
	8	1.23	-8	-12	-10	-14	-20	-8	-21	-22	-11	-26
	12	1.64	-9	-12	-11	-14	-17	-13	-23	-20	-16	-28
100	6	0.95	-5	-11	-6	-12	-18	-5	-23	-21	-7	-27
	8	1.16	-4	-9	-6	-11	-20	-5	-24	-23	-7	-29
	12	1.54	-6	-11	-9	-13	-19	-9	-28	-22	-12	-32
120	6	0.91	-2	-9	-4	-11	-19	1	-22	-21	-2	-30
	8	1.10	-2	-8	-4	-11	-20	1	-26	-22	-3	-32
	12	1.47	-5	-10	-8	-13	-21	-4	-33	-23	-9	-36

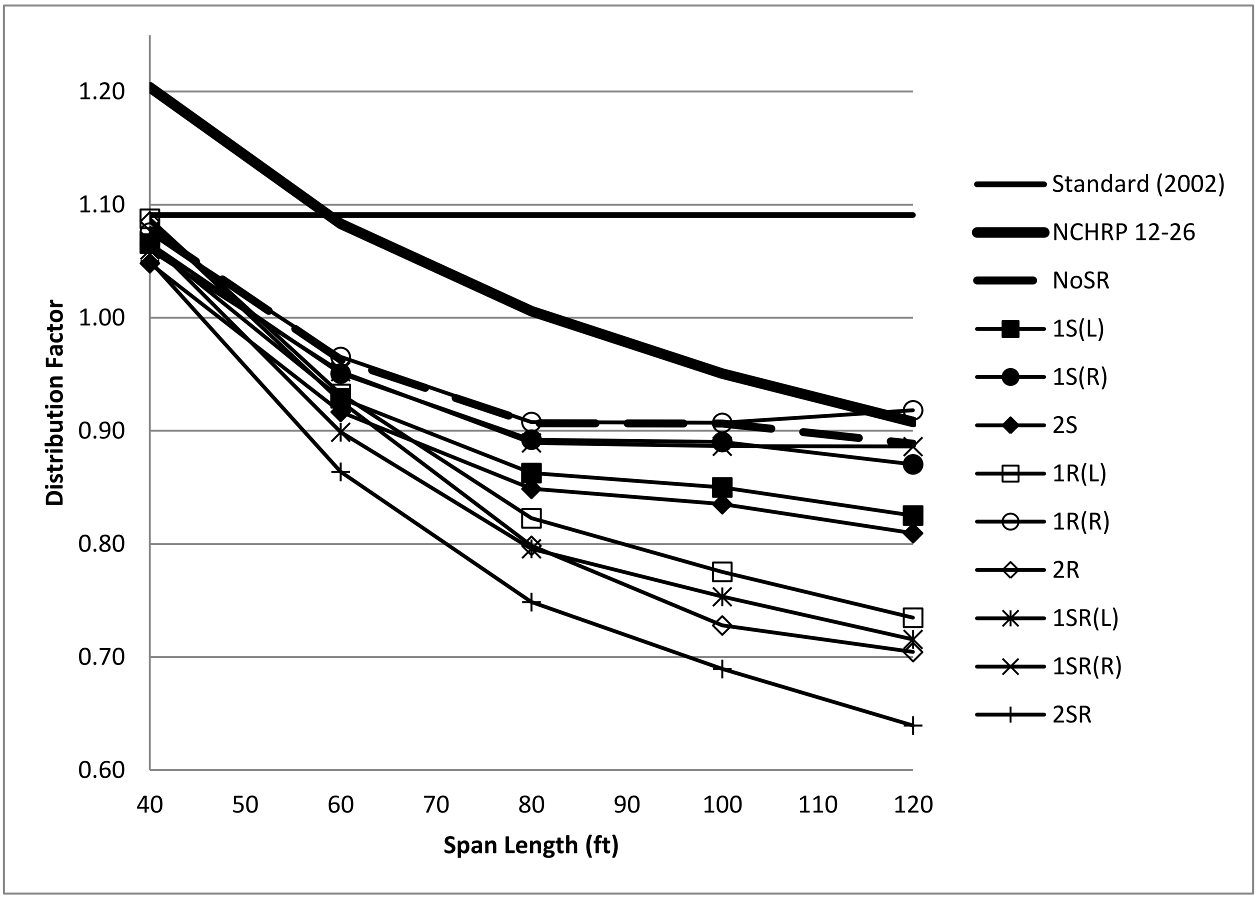


Figure 4.40. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6 ft)

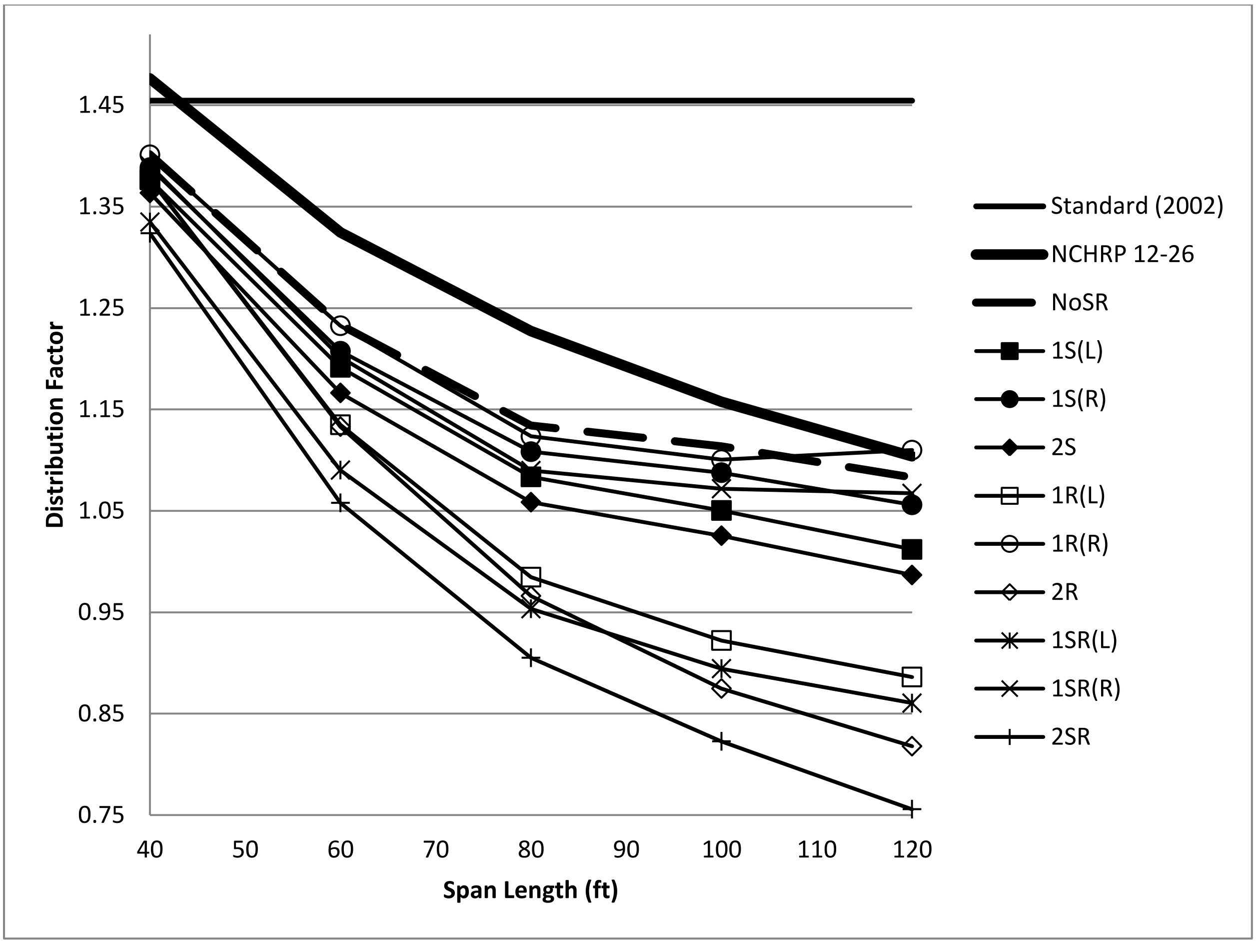


Figure 4.41. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8 ft)

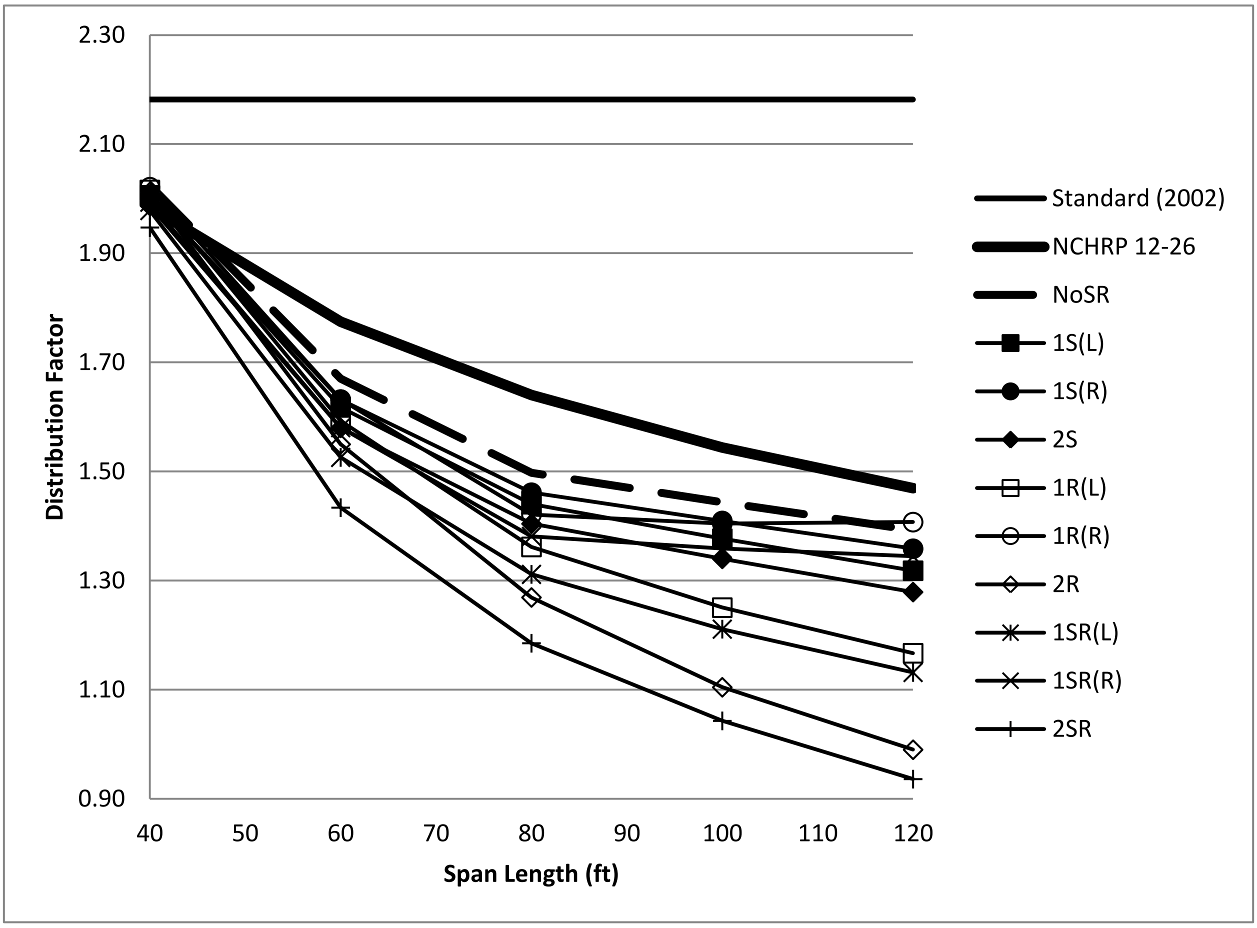


Figure 4.42. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12 ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.30 (considering steel beams only) as compared to Table 4.28 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.28 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: for spans up to 40 ft, (1) is approximately equal to FEA distribution factors for a girder spacing up to 8ft while FEA distribution factors are about 12 % higher than (1) for girder spacings between 8 and 12 ft. Otherwise, (1) is nearly equal to FEA distribution factors for spans between 40 and 120 ft.
2. Sidewalk on one side (left or right): for spans up to 40 ft, (1) is approximately equal to FEA distribution factors for a girder spacing up to 8ft while FEA distribution factors are about 11 % higher than (1) for girder spacings between 8 and 12 ft. Otherwise, (1) is approximately equal to the FEA distribution factors for spans between 40 and 120 ft.
3. Sidewalk on both sides: for spans up to 40 ft, (1) is approximately equal to FEA distribution factors for a girder spacing up to 8ft while FEA distribution factors are about 10 % higher than (1) for girder spacings between 8 and 12 ft. Otherwise, (1) is about 5 % higher than FEA distribution factors for spans between 40 and 120 ft.

4. Railing on one side (left or right): for spans up to 40 ft, (1) is approximately equal to FEA distribution factors for a girder spacing up to 8ft while FEA distribution factors are about 11 % higher than (1) for girder spacings between 8 and 12 ft. For spans between 40 and 60 ft, (1) is approximately equal to FEA distribution factors. Otherwise (for spans between 60 and 120 ft), (1) is approximately equal to the FEA distribution factors for girder spacings up to 8 ft while (1) is about 5 % higher than FEA values for girder spacings between 8 and 12 ft.
5. Railing on both sides: for spans up to 40 ft, (1) is approximately equal to FEA distribution factors for a girder spacing up to 8ft while FEA distribution factors are about 11 % higher than (1) for girder spacings between 8 and 12 ft. Otherwise, (1) is about 5 % higher than FEA distribution factors for spans between 40 and 60 ft and about 15 % higher for spans between 60 and 80 ft and about 20 % higher for spans between 80 and 120 ft.
6. Sidewalk and railing on one side (left or right): for spans up to 40 ft, (1) is approximately equal to the FEA distribution factors for a girder spacing up to 8ft while FEA distribution factors are about 9 % higher than (1) for girder spacings between 8 and 12 ft. Otherwise, (1) is about 3 % higher than FEA distribution factors for spans between 40 and 120 ft.
7. Sidewalk and railing on both sides: for spans up to 40 ft, (1) is about 4 % higher than FEA distribution factors for a girder spacing up to 8ft while FEA distribution factors are about 8 % higher than (1) for girder spacings between 8 and 12 ft. Otherwise, (1) is about 15 % higher than FEA distribution factors for spans between 40 and 60 ft and about 21 % higher for spans

between 60 and 80 ft and about 24 % higher for spans between 80 and 120 ft.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it's important to note that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a 4 % reduction in distribution factor is observed. Otherwise, the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 6 % when introducing sidewalks on both sides; 1 % for spans between 40 and 120 ft (12 and 36 m) with girder spacing up to 8 ft and 3 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing between 8 and 12 ft and 8 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 8 and 12 ft when introducing railings on one side; 7 % for spans between 40 and 60 ft (12 and 18 m) and 12 % for spans between 60 and 80 ft (18 m and 24 m) and 22 % for spans between 80 and 120 ft (24 and 36 m) when introducing railings on both sides; 2 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing up to 8 ft and 6 % for spans between 40 and 80 ft (12 and 24 m) with girder spacing between 8 and 12 ft and 4 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 8 ft and 10 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 8 and 12 ft for a combination of sidewalks and railings on one side; and 10 % for spans between 40 and 60 ft (12 and 18 m) and 17 % for spans between 60 and 80 ft (18 m and 24 m) and 25 % for spans between 80 and 120 ft (24 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.4.2. Three-Lane Bridges

Tables A.22-A.24 in the appendix show a summary of the negative bending moments calculated in the concrete slab and in steel girders for a two-span 3-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.22 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 17 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 32 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 43 %.

Unlike the two-lane case, the discussion of the three-lane case is handled in two parts. First, we'll consider the three-lane bridges with a 6ft girder spacing separately. Similarly to the previous discussion for the 2-lane case, since the AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur in one of the three left side girders. When the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment occurred in some cases in the left exterior girder. However, using Table A.22 to identify the maximum bending moments at critical sections (occurring sometimes in the exterior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge.

On the same hand, Tables A.23 and A.24 show that for three-lane bridge cases with a girder spacing of 8ft and 12 ft, the maximum bending moment (girder+slab) is always observed in one of the two inner girders unlike the two-lane cases in which the maximum moment used to be observed in one of the two adjacent exterior girders regardless of girder spacing. This is mainly due to the transverse (lateral) positioning of truck loads on the bridge cross-section which differs between the two-lane and three-lane cases (reflected in figure 3.8). However, the latter has no significant effect on our approach to the analysis as we're already excluding the exterior girders which used to produce the highest total moment (girder+slab) in the 2-lane case for any girder spacing. In this context, although the maximum moment is mostly occurring in one of the interior girders even after placing sidewalks and/or railing on the heavily loaded edge; excluding the exterior girders moment would affect nothing but only make sure that if for any bridge case, the addition of sidewalks and railings increases the moment in the exterior girder such that it exceeds the maximum moment in any one of the interior girders; this exterior girder moment is excluded from the analysis as it is an overestimate of the maximum design bending moment. Furthermore, it's important to note that although the maximum moment was still observed in an interior girder, it's worth to be noted that its value was significantly reduced by the placing of sidewalks and/or railings, which is to be discussed further later on. Furthermore, it is worth mentioning that all the maximum wheel load distributions of the 3-lane bridge cases have been reduced by 10 %, as permitted by the AASHTO Standard Specifications (2002) for 3-lane bridges, in order to account for the improbable situation of having all lanes loaded at the same time and at locations along the bridge deck producing the maximum bending moment in a bridge superstructure. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 9 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides.

These maximum bending moments and FEA distribution factors are summarized in Table 4.31 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 3-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.32 for all the bridges.

Table 4.31. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	2332	2333	2332	2327	2350	2331	2349	2346	2332	2340
	8	1,874	3101	3045	3101	3044	3101	3101	3102	3047	3103	3049
	12	1,874	4509	4498	4480	4469	4512	4496	4499	4505	4442	4438
60	**6**	4,394	5162	5059	5136	5033	5137	5163	5131	5068	5144	5005
	8	4,394	6717	6501	6642	6429	6687	6724	6694	6442	6651	6378
	12	4,394	9465	9360	9260	9156	9494	9215	9243	9376	8939	8847
80	**6**	6,545	7123	6967	7053	6899	6875	7131	6862	6745	7058	6595
	8	6,545	8989	8713	8838	8571	8818	8982	8810	8522	8809	8313
	12	6,545	12725	12529	12410	12218	12723	12068	11921	12459	11800	11343
100	**6**	9,936	10535	10262	10405	10137	9738	10528	9708	9551	10412	9303
	8	9,936	12807	12454	12576	12236	12247	12692	12093	11971	12400	11437
	12	9,936	18392	18076	17850	17538	18191	17217	16109	17732	16774	15374
120	**6**	14,432	15074	14293	14904	14102	13163	15217	13016	12909	14984	12495
	8	14,432	17426	16963	17121	16673	16415	17060	15731	16060	16672	14995
	12	14,432	25455	24979	24532	24053	24819	23457	20886	24149	22831	19983

(b) Distribution factor = 0.9*DF = 0.9*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	1.12	1.12	1.12	1.12	1.13	1.12	1.13	1.13	1.12	1.12
	8	1,874	1.49	1.46	1.49	1.46	1.49	1.49	1.49	1.46	1.49	1.46
	12	1,874	2.17	2.16	2.15	2.15	2.17	2.16	2.16	2.16	2.13	2.13
60	**6**	4,394	1.06	1.04	1.05	1.03	1.05	1.06	1.05	1.04	1.05	1.03
	8	4,394	1.38	1.33	1.36	1.32	1.37	1.38	1.37	1.32	1.36	1.31
	12	4,394	1.94	1.92	1.90	1.88	1.94	1.89	1.89	1.92	1.83	1.81
80	**6**	6,545	0.98	0.96	0.97	0.95	0.95	0.98	0.94	0.93	0.97	0.91
	8	6,545	1.24	1.20	1.22	1.18	1.21	1.24	1.21	1.17	1.21	1.14
	12	6,545	1.75	1.72	1.71	1.68	1.75	1.66	1.64	1.71	1.62	1.56
100	**6**	9,936	0.95	0.93	0.94	0.92	0.88	0.95	0.88	0.87	0.94	0.84
	8	9,936	1.16	1.13	1.14	1.11	1.11	1.15	1.10	1.08	1.12	1.04
	12	9,936	1.67	1.64	1.62	1.59	1.65	1.56	1.46	1.61	1.52	1.39
120	**6**	14,432	0.94	0.89	0.93	0.88	0.82	0.95	0.81	0.81	0.93	0.78
	8	14,432	1.09	1.06	1.07	1.04	1.02	1.06	0.98	1.00	1.04	0.94
	12	14,432	1.59	1.56	1.53	1.50	1.55	1.46	1.30	1.51	1.42	1.25

Table 4.32. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1.09	3	3	3	3	4	3	3	3	3	3
	8	1.46	2	0	2	0	2	2	2	0	2	0
	12	2.18	-1	-1	-1	-2	-1	-1	-1	-1	-2	-2
60	6	1.09	-3	-5	-3	-5	-3	-3	-4	-5	-3	-6
	8	1.46	-6	-9	-7	-10	-6	-6	-6	-10	-7	-11
	12	2.18	-11	-12	-13	-14	-11	-13	-13	-12	-16	-17
80	6	1.09	-10	-12	-11	-13	-13	-10	-13	-15	-11	-17
	8	1.46	-15	-18	-17	-19	-17	-15	-17	-20	-17	-22
	12	2.18	-20	-21	-22	-23	-20	-24	-25	-21	-26	-28
100	6	1.09	-12	-15	-14	-16	-19	-13	-19	-21	-13	-23
	8	1.46	-21	-23	-22	-24	-24	-21	-25	-26	-23	-29
	12	2.18	-24	-25	-26	-27	-24	-28	-33	-26	-30	-36
120	6	1.09	-14	-18	-15	-19	-25	-13	-26	-26	-14	-29
	8	1.46	-26	-28	-27	-29	-30	-27	-33	-31	-29	-36
	12	2.18	-27	-29	-30	-31	-29	-33	-40	-31	-35	-43

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1.20	-7	-7	-7	-7	-6	-7	-6	-6	-7	-7
	8	1.48	1	-1	1	-1	1	1	1	-1	1	-1
	12	1.98	9	9	8	8	9	9	9	9	7	7
60	6	1.08	-2	-4	-3	-5	-3	-2	-3	-4	-3	-5
	8	1.32	4	1	3	-1	3	4	4	0	3	-1
	12	1.77	9	8	7	6	10	6	7	8	3	2
80	6	1.01	-3	-5	-4	-6	-6	-3	-6	-8	-4	-10
	8	1.23	1	-2	-1	-4	-1	1	-1	-5	-1	-7
	12	1.64	7	5	4	2	7	1	0	4	-1	-5
100	6	0.95	0	-2	-1	-3	-7	0	-7	-9	-1	-11
	8	1.16	0	-3	-2	-4	-4	-1	-5	-6	-3	-11
	12	1.54	8	6	5	3	7	1	-5	4	-2	-10
120	6	0.91	4	-2	2	-3	-10	5	-11	-11	3	-14
	8	1.10	-2	-4	-3	-6	-7	-4	-11	-9	-6	-15
	12	1.47	8	6	4	2	5	0	-11	2	-3	-15

Figures 4.43-4.45 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 3-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.33. It should be noted that Table 4.33 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.34 for all the bridges. Figures 4.46-4.48 show a trend similar to Figures 4.43-4.45, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

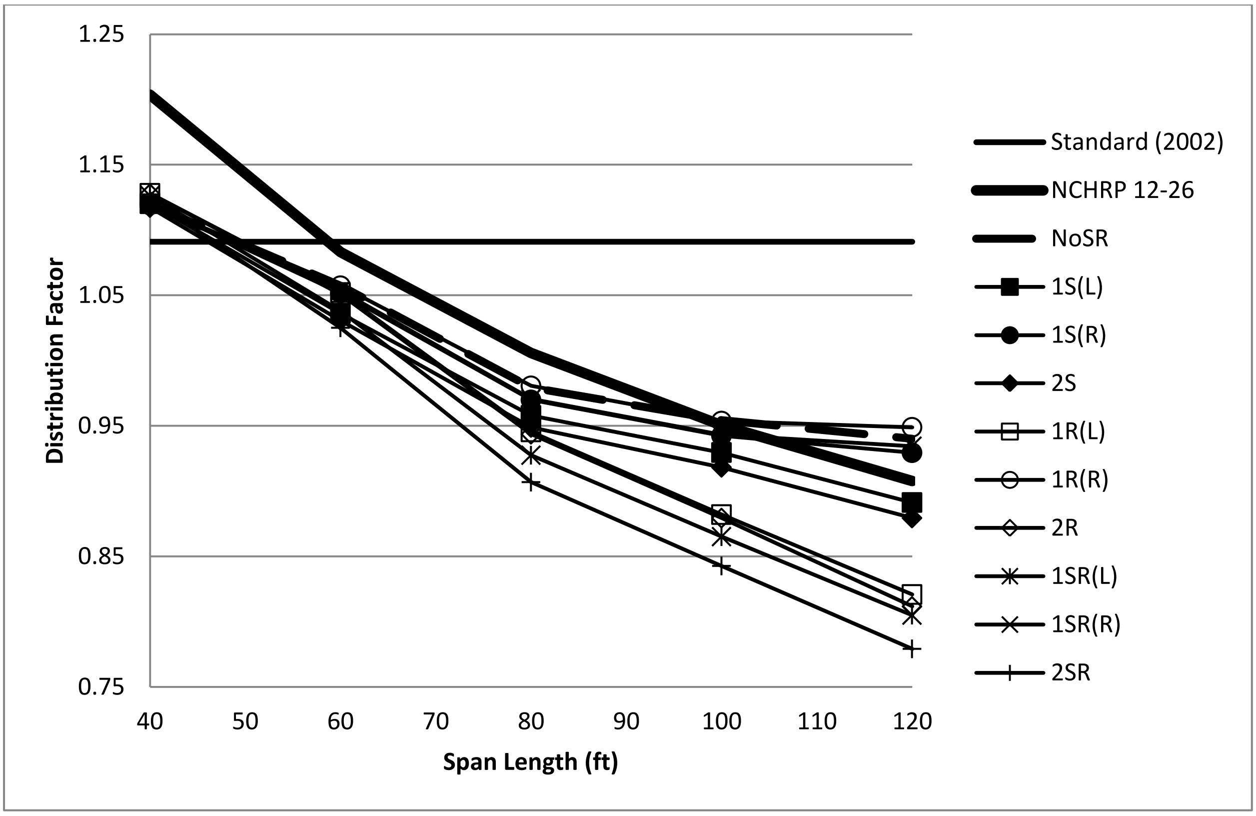


Figure 4.43. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

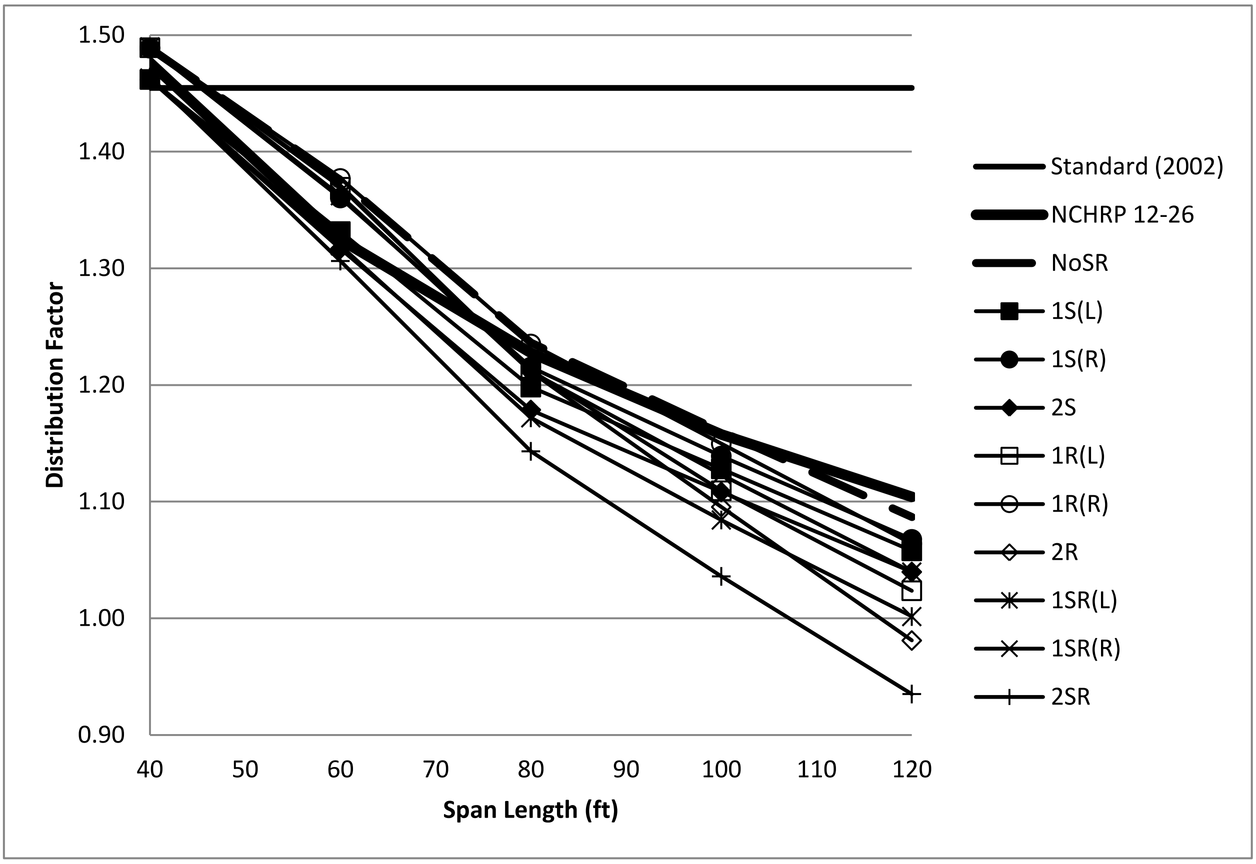


Figure 4.44. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

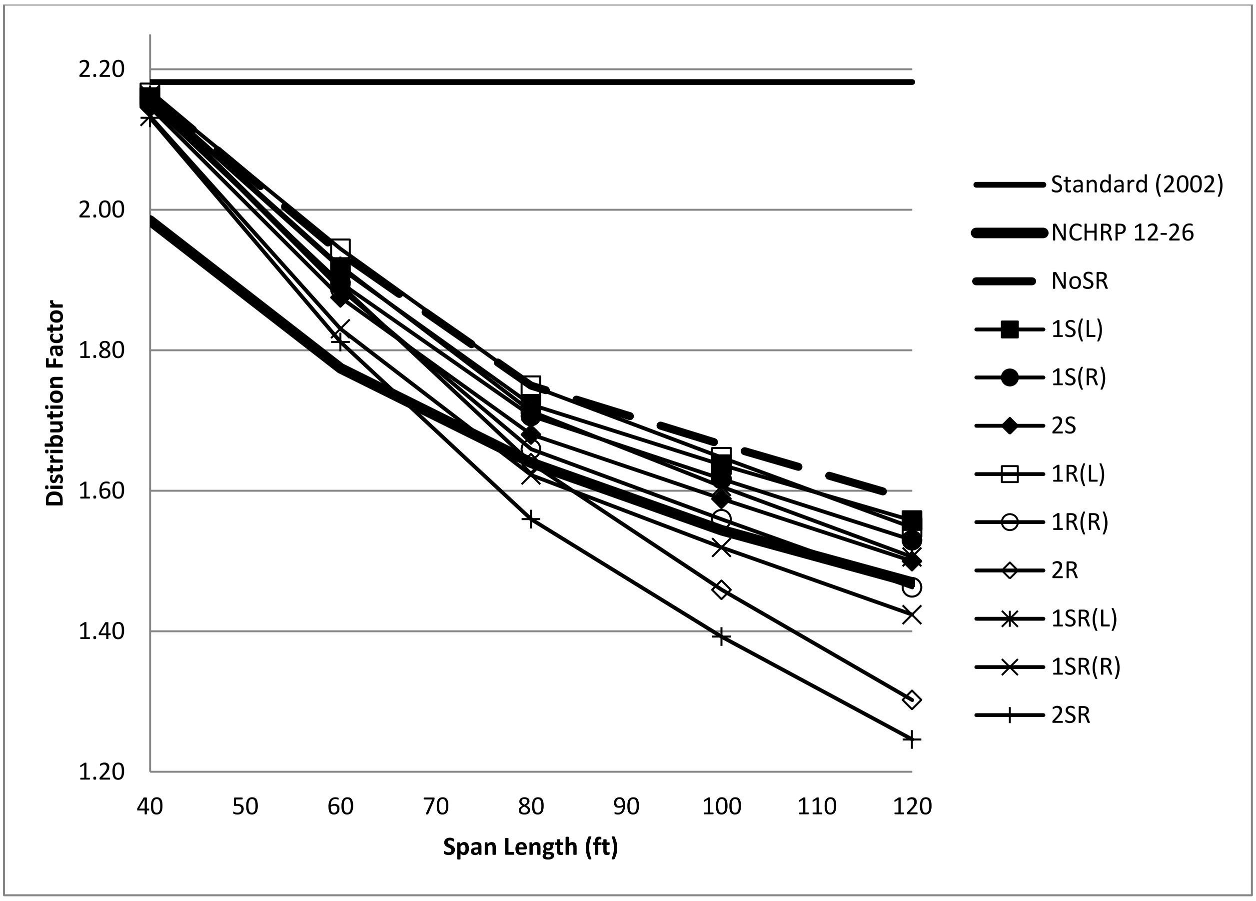


Figure 4.45. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.33. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1,874	2191	2192	2191	2186	2208	2190	2207	2204	2191	2199
	8	1,874	2878	2826	2877	2826	2878	2878	2878	2829	2879	2831
	12	1,874	4098	4089	4070	4061	4101	4088	4091	4096	4037	4034
60	6	4,394	4881	4784	4856	4759	4858	4881	4852	4793	4863	4734
	8	4,394	6273	6073	6203	6006	6246	6280	6253	6018	6212	5959
	12	4,394	8649	8554	8457	8364	8676	8425	8451	8569	8165	8084
80	6	6,545	6739	6592	6673	6528	6505	6747	6492	6381	6678	6239
	8	6,545	8397	8140	8256	8007	8237	8391	8229	7961	8228	7765
	12	6,545	11620	11442	11329	11154	11619	11022	10891	11378	10778	10355
100	6	9,936	10018	9759	9894	9640	9259	10012	9231	9081	9901	8846
	8	9,936	12028	11697	11810	11492	11502	11920	11356	11242	11645	10740
	12	9,936	16884	16595	16382	16097	16699	15806	14799	16279	15401	14113
120	6	14,432	14354	13610	14191	13427	12531	14489	12391	12289	14268	11894
	8	14,432	16384	15950	16096	15677	15433	16038	14785	15098	15673	14094
	12	14,432	23385	22949	22534	22097	22799	21548	19198	22184	20974	18356

(b) Distribution factor = 0.9*DF = 0.9*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1,874	1.05	1.05	1.05	1.05	1.06	1.05	1.06	1.06	1.05	1.06
	8	1,874	1.38	1.36	1.38	1.36	1.38	1.38	1.38	1.36	1.38	1.36
	12	1,874	1.97	1.96	1.95	1.95	1.97	1.96	1.96	1.97	1.94	1.94
60	6	4,394	1.00	0.98	0.99	0.97	1.00	1.00	0.99	0.98	1.00	0.97
	8	4,394	1.28	1.24	1.27	1.23	1.28	1.29	1.28	1.23	1.27	1.22
	12	4,394	1.77	1.75	1.73	1.71	1.78	1.73	1.73	1.76	1.67	1.66
80	6	6,545	0.93	0.91	0.92	0.90	0.89	0.93	0.89	0.88	0.92	0.86
	8	6,545	1.15	1.12	1.14	1.10	1.13	1.15	1.13	1.09	1.13	1.07
	12	6,545	1.60	1.57	1.56	1.53	1.60	1.52	1.50	1.56	1.48	1.42
100	6	9,936	0.91	0.88	0.90	0.87	0.84	0.91	0.84	0.82	0.90	0.80
	8	9,936	1.09	1.06	1.07	1.04	1.04	1.08	1.03	1.02	1.05	0.97
	12	9,936	1.53	1.50	1.48	1.46	1.51	1.43	1.34	1.47	1.39	1.28
120	6	14,432	0.90	0.85	0.88	0.84	0.78	0.90	0.77	0.77	0.89	0.74
	8	14,432	1.02	0.99	1.00	0.98	0.96	1.00	0.92	0.94	0.98	0.88
	12	14,432	1.46	1.43	1.41	1.38	1.42	1.34	1.20	1.38	1.31	1.14

Table 4.34. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	**S (ft)**	**AASHTO**	**No**	**1S(L)**	**1S(R)**	**2S**	**1R(L)**	**1R(R)**	**2R**	**1SR(L)**	**1SR(R)**	**2SR**
	6	**1.09**	-3	-3	-3	-4	-3	-4	-3	-3	-3	-3
40	**8**	**1.46**	-5	-7	-5	-7	-5	-5	-5	-7	-5	-7
	12	**2.18**	-10	-10	-10	-11	-10	-10	-10	-10	-11	-11
	6	**1.09**	-8	-10	-9	-11	-9	-8	-9	-10	-9	-11
60	**8**	**1.46**	-12	-15	-13	-16	-12	-12	-12	-16	-13	-16
	12	**2.18**	-19	-20	-21	-21	-18	-21	-21	-19	-23	-24
	6	**1.09**	-15	-17	-16	-18	-18	-15	-18	-19	-16	-21
80	**8**	**1.46**	-21	-23	-22	-25	-22	-21	-22	-25	-23	-27
	12	**2.18**	-27	-28	-29	-30	-27	-30	-31	-28	-32	-35
	6	**1.09**	-17	-19	-18	-20	-23	-17	-23	-25	-18	-26
100	**8**	**1.46**	-25	-27	-27	-29	-29	-26	-30	-30	-28	-33
	12	**2.18**	-30	-31	-32	-33	-31	-34	-39	-32	-36	-41
	6	**1.09**	-18	-22	-19	-23	-28	-17	-29	-30	-18	-32
120	**8**	**1.46**	-30	-32	-31	-33	-34	-31	-37	-36	-33	-40
	12	**2.18**	-33	-34	-36	-37	-35	-38	-45	-37	-40	-47

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	**S (ft)**	**NCHRP**	**No**	**1S(L)**	**1S(R)**	**2S**	**1R(L)**	**1R(R)**	**2R**	**1SR(L)**	**1SR(R)**	**2SR**
	6	**1.20**	-13	-13	-13	-13	-12	-13	-12	-12	-13	-12
40	**8**	**1.48**	-6	-8	-6	-8	-6	-6	-6	-8	-6	-8
	12	**1.98**	-1	-1	-2	-2	-1	-1	-1	-1	-2	-2
	6	**1.08**	-8	-10	-8	-10	-8	-8	-8	-9	-8	-10
60	**8**	**1.32**	-3	-6	-4	-7	-3	-3	-3	-7	-4	-8
	12	**1.77**	0	-1	-2	-3	0	-3	-2	-1	-6	-7
	6	**1.01**	-8	-10	-9	-11	-11	-8	-11	-13	-9	-15
80	**8**	**1.23**	-6	-9	-8	-10	-8	-6	-8	-11	-8	-13
	12	**1.64**	-3	-4	-5	-6	-3	-8	-9	-5	-10	-13
	6	**0.95**	-5	-7	-6	-8	-12	-5	-12	-13	-6	-16
100	**8**	**1.16**	-6	-8	-8	-10	-10	-7	-11	-12	-9	-16
	12	**1.54**	-1	-3	-4	-6	-2	-7	-13	-4	-10	-17
	6	**0.91**	-1	-7	-3	-8	-14	0	-15	-16	-2	-18
120	**8**	**1.10**	-7	-10	-9	-11	-13	-9	-16	-15	-11	-20
	12	**1.47**	-1	-3	-4	-6	-3	-9	-19	-6	-11	-22

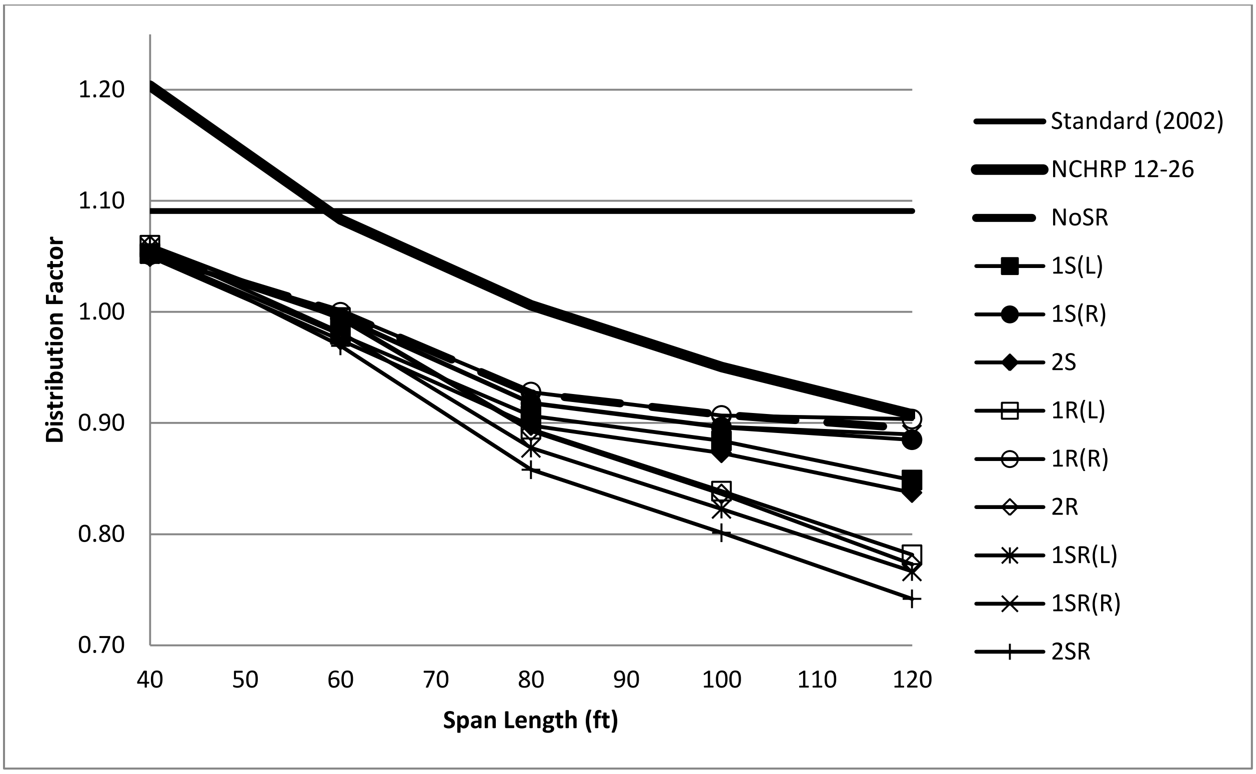


Figure 4.46. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6ft)

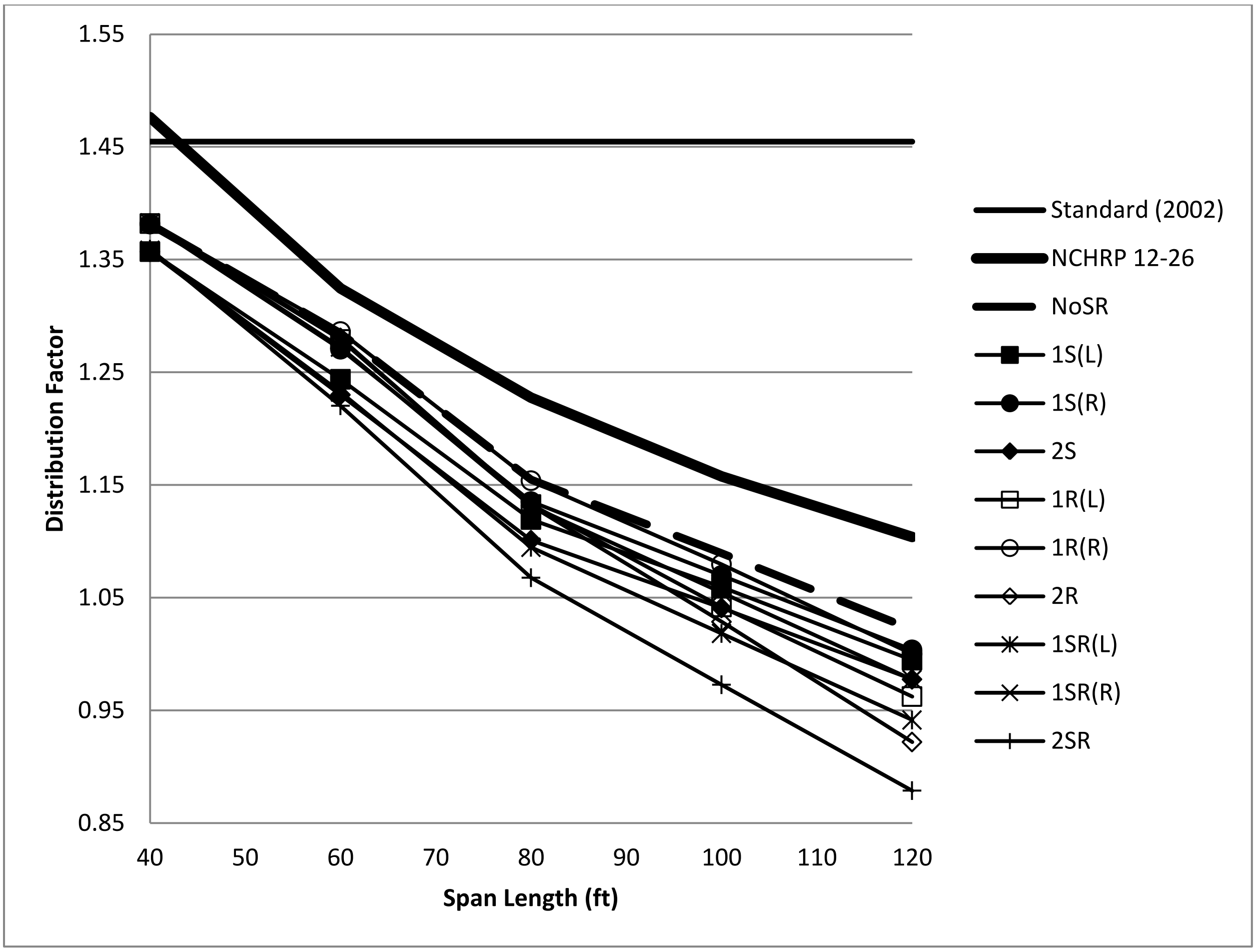


Figure 4.47. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8ft)

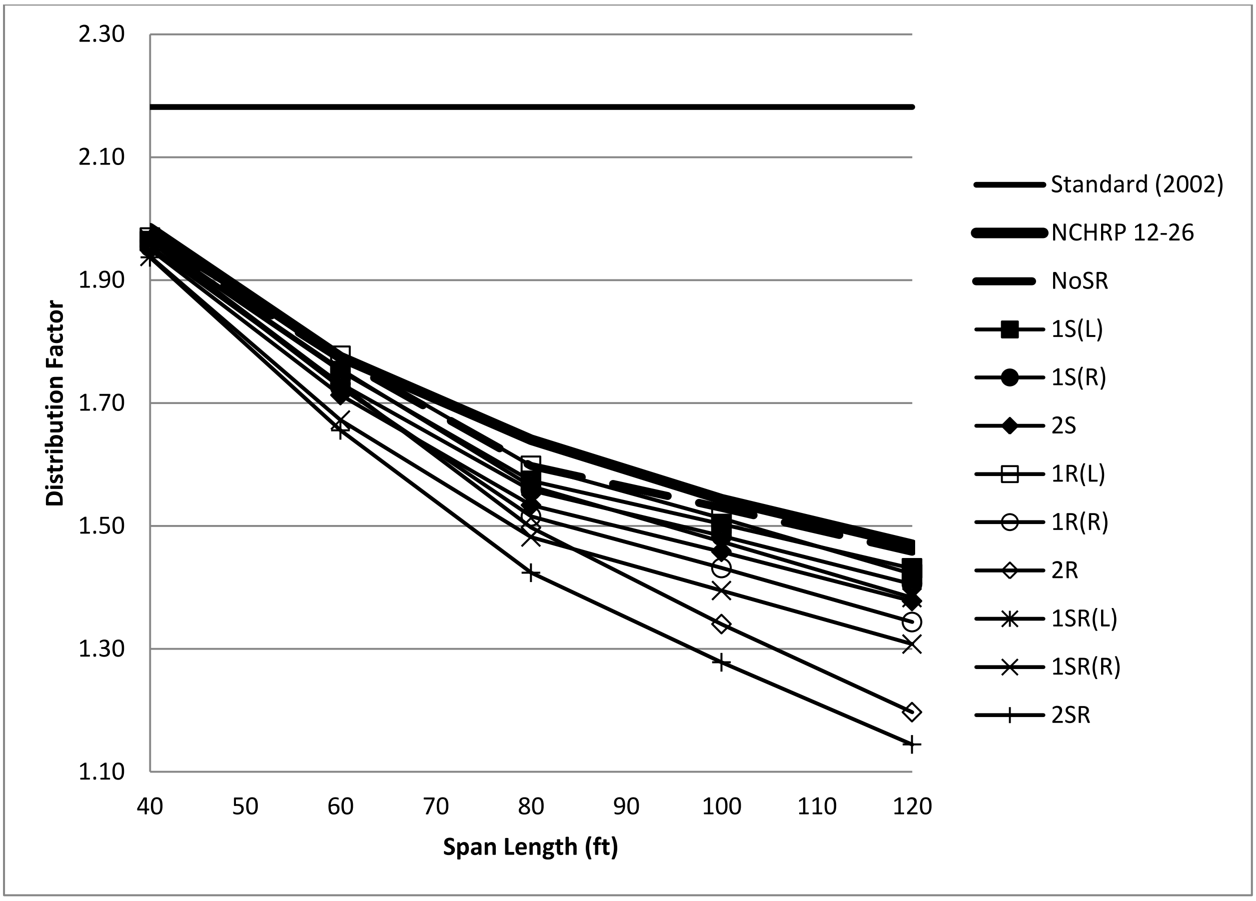


Figure 4.48. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.34 (considering steel beams only) as compared to Table 4.32 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.32 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: regardless of the span length, the FEA distribution factors are approximately equal to (1) for girder spacings up to 8 ft and about 8 % higher than (1) for girder spacings between 8 and 12 ft.
2. Sidewalk on one side (left or right): regardless of the span length, (1) is about 2 % higher than FEA distribution factors for girder spacings up to 8 ft while FEA distribution factors are about 5 % higher than (1) for girder spacings between 8 and 12 ft.
3. Sidewalk on both sides: for spans up to 60 ft, (1) is about 5 % higher than FEA distribution factors for a girder spacing up to 6ft and about 1 % higher for girder spacing between 6 and 8 ft while FEA distribution factors are about 7 % higher than (1) for girder spacings between 8 and 12 ft. For spans between 60 and 120 ft, (1) is about 3 % higher than FEA distribution factors for girder spacings up to 8ft while FEA distribution factors are about 2 % higher than (1) for girder spacing between 8 and 12 ft.
4. Railing on one side (left or right): for spans up to 60 ft, (1) is about 2 % higher than FEA distribution factors for girder spacings up to 6ft and (1) is

nearly equal to FEA results for a girder spacing between 6 and 8ft while FEA results are about 6 % higher than (1) for a girder spacing between 8 and 12 ft. However, (1) is approximately equal to the FEA results for spans between 60 and 120 ft.

5. Railing on both sides: for spans up to 60 ft, (1) is about 3 % higher than FEA distribution factors for girder spacings up to 6 ft, and (1) is approximately equal to the FEA distribution factors for girder spacings between 6 and 8ft while FEA distribution factors are about 8 % higher than (1) for girder spacings between 8 and 12 ft. Similarly, (1) is approximately equal to FEA results for spans between 60 and 80 ft. Otherwise, (1) is about 5 % higher than FEA distribution factors for spans between 80 and 100 ft and about 11 % higher for spans between 100 and 120 ft.
6. Sidewalk and railing on one side (left or right): for spans up to 60 ft, (1) is approximately equal to FEA distribution factors for girder spacings up to 8ft while FEA distribution factors are about 3 % higher than (1) for girder spacings between 8 and 12 ft. For spans between 60 and 120 ft, (1) is approximately equal to the FEA distribution factors.
7. Sidewalk and railing on both sides: for spans up to 40 ft, (1) is approximately equal to FEA distribution factors for girder spacings up to 8ft while FEA distribution factors are about 7 % higher than (1) for girder spacings between 8 and 12 ft. Similarly, (1) is approximately equal to FEA results for spans between 40 and 60 ft. Otherwise, (1) is about 5 % higher than FEA distribution factors for spans between 60 and 80 ft and about 10 %

higher for spans between 80 and 100 ft and about 15% higher for spans between 100 and 120 ft.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it's important to note that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a maximum of 2 % reduction in distribution factor is observed. Otherwise, the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 4 % when introducing sidewalks on both sides; 0 % for spans between 40 and 80 ft (12 and 24 m) and 0 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 6 ft and 2 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 6 and 12 ft when introducing railings on one side ; 1 % for spans between 40 and 60 ft (12 and 18 m) and 4 % for spans between 60 and 80 ft (18 m and 24 m) and 8 % for spans between 80 and 120 ft (24 and 36 m) when introducing railings on both sides; 1 % for spans between 40 and 80 ft (12 and 24 m) and 1 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing up to 6ft and 4 % for spans between 80 and 120 ft (24 and 36 m) with girder spacing between 6 and 12 ft for a combination of sidewalks and railings on one side; and 5 % for spans between 40 and 60 ft (12 and 18 m) and 8 % for spans between 60 and 80 ft (18 and 24 m) and 13 % for spans between 80 and 120 ft (24 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

4.4.3. Four-Lane Bridges

Tables A.25-A.27 in the appendix show a summary of the negative bending moments calculated in the concrete slab and in steel girders for a two-span 4-lane bridge with span length of 80 ft (24.4 m) and girder spacings of 6, 8, and 12 ft (1.83, 2.44, and 3.66 m) due to the various cases related to the presence of sidewalks and/or railings. Table A.25 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no sidewalk and/or railing on the bridge. However, when introducing a sidewalk on either side or on both sides of the bridge deck, the concrete slab and sidewalk contribute about 16 % to the total bending moment of the exterior girder. On introducing a railing or parapet on either side or on both sides of the bridge deck, the concrete slab and railing will contribute about 28 % to the total bending moment of the exterior girder. Moreover, introducing the combination of sidewalk and railing on either side or on both sides will raise the contribution percentage to about 41 %.

Unlike the two-span positive moment case, when looking for the negative moment in a 2-span 4-lane bridge; even though AASHTO trucks were placed 2 ft (0.61 m) from the left girder, the maximum bending moment will occur in one of the interior girders. And even when the sidewalks and/or railings were placed on the left side or on both sides, the maximum bending moment still occurred mostly in one of the interior girders. However, using Tables A.25-A.27 to identify the maximum bending moments at critical sections (usually occurring in the interior girder) and then to calculate the corresponding FEA distribution factors, will yield values higher or lower than the AASHTO (2002) and (1), depending on the geometry of the bridge. However, this observation has no significant effect on our approach to the analysis as we're already excluding the exterior girders which used to produce the highest total moment in previous bridge cases. In this context, although the maximum moment is mostly occurring in one of the interior girders even after placing sidewalks and/or railing on the heavily loaded edge; excluding the exterior girders moment would affect nothing but only make sure that if for any bridge case, the addition of sidewalks and railings increases the moment in the exterior girder such that it exceeds the maximum moment in any one of the interior girders; this exterior girder moment is excluded from the analysis as it is an overestimate of the maximum design bending moment. Furthermore, it's important to note that although the maximum moment was still observed in an interior girder, it's worth to be noted that its value was significantly reduced by the placing of sidewalks and/or railings, which is to be discussed further later on. It is worth mentioning that all maximum wheel load distributions of the 4-lane bridge cases have been reduced by 25 %, as permitted by the AASHTO Standard Specifications (2002) for 4-lane bridges, in order to account for the improbable situation of having all lanes loaded at the same time and at locations along the bridge deck producing the maximum bending moment in a bridge super-structure. The effective section of a concrete slab for the interior girders continues to contribute about 5 % to 9 % of the total bending moment regardless of the presence of sidewalks or railings on one or both sides.

These maximum bending moments and FEA distribution factors are summarized in Table 4.35 for the interior girders. The maximum FEA wheel load distribution factors were then compared with the AASHTO (2002) formula and (l) for the 150 4-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.36 for all the bridges.

Table 4.35. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	2341	2343	2342	2343	2354	2341	2355	2356	2342	2356
	8	1,874	3119	3119	3120	3119	3142	3119	3142	3128	3120	3128
	12	1,874	4668	4662	4658	4652	4687	4664	4683	4680	4659	4672
60	**6**	4,394	5432	5396	5424	5389	5476	5429	5473	5409	5427	5404
	8	4,394	7104	7066	7095	7027	7143	7103	7136	7107	7100	7072
	12	4,394	10408	10329	10307	10228	10495	10400	10486	10363	10310	10264
80	**6**	6,545	7619	7544	7578	7504	7600	7622	7602	7464	7588	7433
	8	6,545	9909	9728	9857	9677	9806	9918	9797	9685	9872	9567
	12	6,545	14160	13997	13979	13776	14186	14129	14155	13895	14005	13620
100	**6**	9,936	11315	11174	11211	11074	11040	11326	11052	10830	11215	10733
	8	9,936	14685	14382	14552	14252	14077	14707	14038	13843	14555	13536
	12	9,936	20862	20210	20697	20042	20019	20914	19808	19526	20712	18859
120	**6**	14,432	15918	15492	15788	15356	14946	15979	14924	14684	15827	14464
	8	14,432	20437	19945	20221	19721	18883	20512	18824	18559	20328	18102
	12	14,432	29233	28070	28915	27752	26538	29250	25821	25876	28827	24561

(b) Distribution factor = 0.75*DF = 0.75*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	1,874	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
	8	1,874	1.25	1.25	1.25	1.25	1.26	1.25	1.26	1.25	1.25	1.25
	12	1,874	1.87	1.87	1.86	1.86	1.88	1.87	1.87	1.87	1.86	1.87
60	**6**	4,394	0.93	0.92	0.93	0.92	0.93	0.93	0.93	0.92	0.93	0.92
	8	4,394	1.21	1.21	1.21	1.20	1.22	1.21	1.22	1.21	1.21	1.21
	12	4,394	1.78	1.76	1.76	1.75	1.79	1.78	1.79	1.77	1.76	1.75
80	**6**	6,545	0.87	0.86	0.87	0.86	0.87	0.87	0.87	0.86	0.87	0.85
	8	6,545	1.14	1.11	1.13	1.11	1.12	1.14	1.12	1.11	1.13	1.10
	12	6,545	1.62	1.60	1.60	1.58	1.63	1.62	1.62	1.59	1.60	1.56
100	**6**	9,936	0.85	0.84	0.85	0.84	0.83	0.85	0.83	0.82	0.85	0.81
	8	9,936	1.11	1.09	1.10	1.08	1.06	1.11	1.06	1.04	1.10	1.02
	12	9,936	1.57	1.53	1.56	1.51	1.51	1.58	1.50	1.47	1.56	1.42
120	**6**	14,432	0.83	0.81	0.82	0.80	0.78	0.83	0.78	0.76	0.82	0.75
	8	14,432	1.06	1.04	1.05	1.02	0.98	1.07	0.98	0.96	1.06	0.94
	12	14,432	1.52	1.46	1.50	1.44	1.38	1.52	1.34	1.34	1.50	1.28

Table 4.36. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1.09	-14	-14	-14	-14	-14	-14	-14	-14	-14	-13
	8	1.46	-15	-15	-14	-15	-14	-15	-14	-14	-14	-14
	12	2.18	-14	-14	-14	-15	-14	-14	-14	-14	-14	-14
60	6	1.09	-15	-15	-15	-16	-14	-15	-14	-15	-15	-15
	8	1.46	-17	-17	-17	-18	-16	-17	-17	-17	-17	-17
	12	2.18	-19	-19	-19	-20	-18	-19	-18	-19	-19	-20
80	6	1.09	-20	-21	-20	-21	-20	-20	-20	-22	-20	-22
	8	1.46	-22	-24	-23	-24	-23	-22	-23	-24	-23	-25
	12	2.18	-26	-26	-27	-28	-25	-26	-26	-27	-26	-28
100	6	1.09	-22	-23	-22	-23	-24	-22	-23	-25	-22	-26
	8	1.46	-24	-26	-25	-26	-27	-24	-27	-28	-25	-30
	12	2.18	-28	-30	-28	-31	-31	-28	-31	-32	-28	-35
120	6	1.09	-24	-26	-25	-27	-29	-24	-29	-30	-25	-31
	8	1.46	-27	-29	-28	-30	-33	-27	-33	-34	-28	-36
	12	2.18	-30	-33	-31	-34	-37	-30	-38	-38	-31	-41

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1.20	-22	-22	-22	-22	-22	-22	-22	-22	-22	-22
	8	1.48	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15
	12	1.98	-6	-6	-6	-6	-5	-6	-6	-6	-6	-6
60	6	1.08	-14	-15	-15	-15	-14	-14	-14	-15	-14	-15
	8	1.32	-8	-9	-9	-9	-8	-8	-8	-8	-9	-9
	12	1.77	0	-1	-1	-2	1	0	1	0	-1	-1
80	6	1.01	-13	-14	-14	-15	-13	-13	-13	-15	-14	-15
	8	1.23	-7	-9	-8	-10	-8	-7	-9	-10	-8	-11
	12	1.64	-1	-2	-2	-4	-1	-1	-1	-3	-2	-5
100	6	0.95	-10	-11	-11	-12	-12	-10	-12	-14	-11	-15
	8	1.16	-4	-6	-5	-7	-8	-4	-8	-10	-5	-12
	12	1.54	2	-1	1	-2	-2	2	-3	-5	1	-8
120	6	0.91	-9	-11	-10	-12	-14	-9	-15	-16	-9	-17
	8	1.10	-4	-6	-5	-7	-11	-3	-11	-13	-4	-15
	12	1.47	3	-1	2	-2	-6	3	-9	-8	2	-13

Figures 4.49-4.51 show the variation of all the distribution factors as a function of span length. AASHTO (2002) factors are shown to be the most conservative. To a lesser extent, (l) is also shown to be conservative, and it follows a similar trend to the FEA results of bridge models without sidewalks and railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 150 4-lane bridges, considering only the bending moments in all the steel girders at critical sections, is presented in Table 4.37. It should be noted that Table 4.37 reports the contribution of steel girders only; therefore, the maximum bending moments and distribution factors listed do not include the contributions of the concrete slab, sidewalk, and railing. Again, the FEA distribution factors were symbolically compared with the AASHTO (2002) formula and (1). A summary of the percentage decrease in distribution factors, when considering the maximum bending moments in the steel girders only, are shown in Table 4.38 for all the bridges. Figures 4.52-4.54 show a trend similar to Figures 4.49-4.51, respectively, of the wheel load distribution factors as a function of span length for the various bridge conditions.

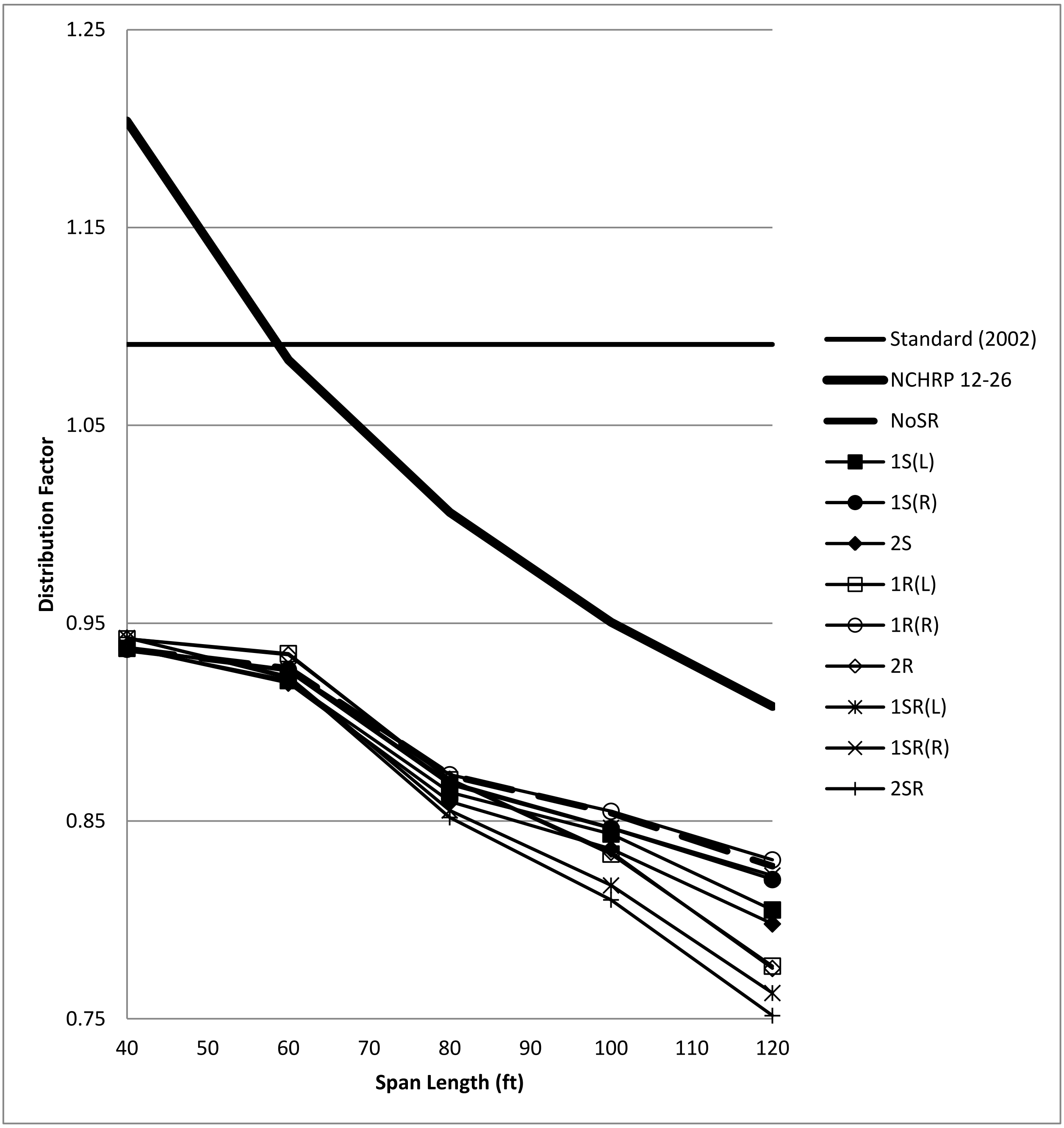


Figure 4.49. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

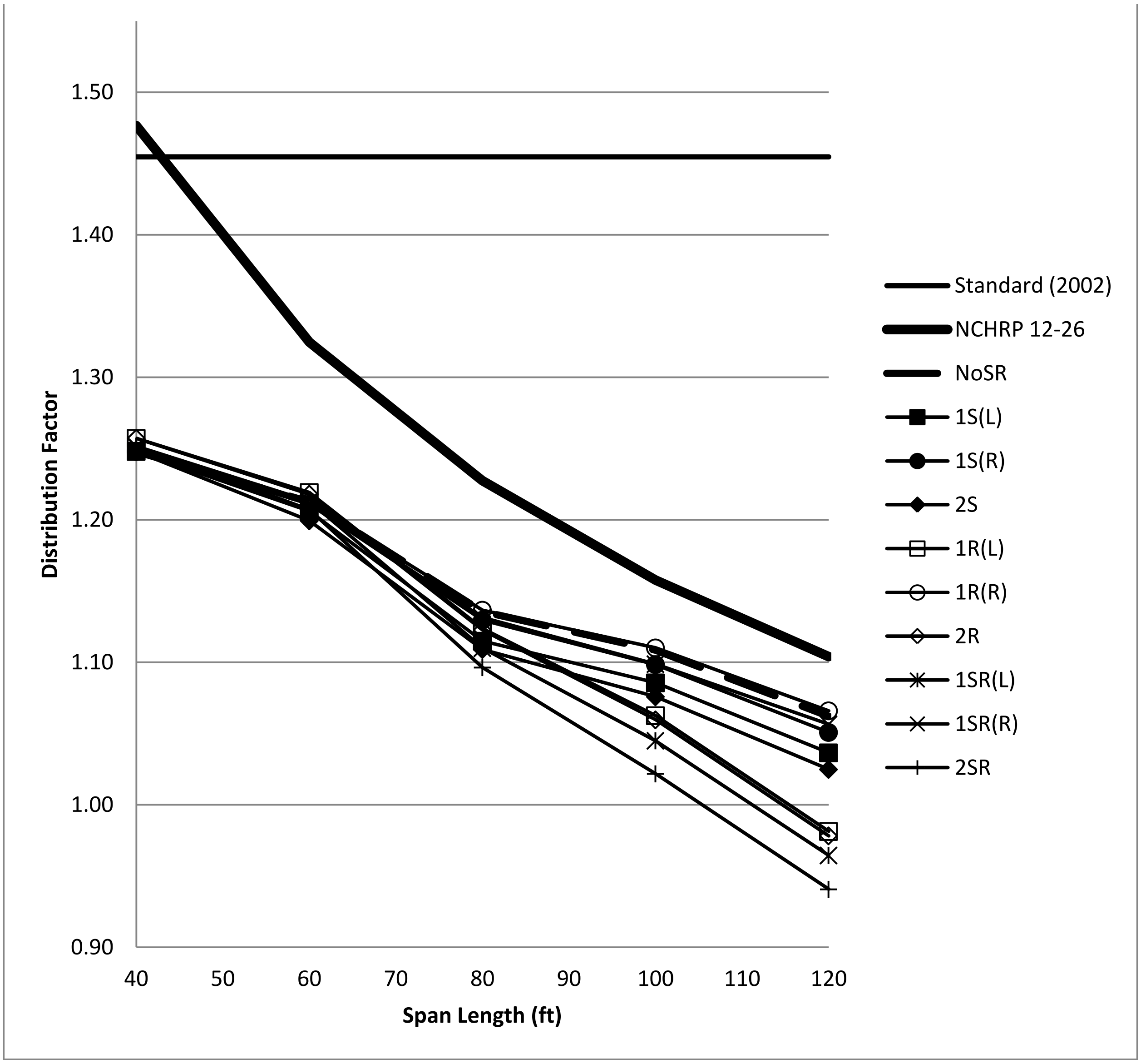


Figure 4.50. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

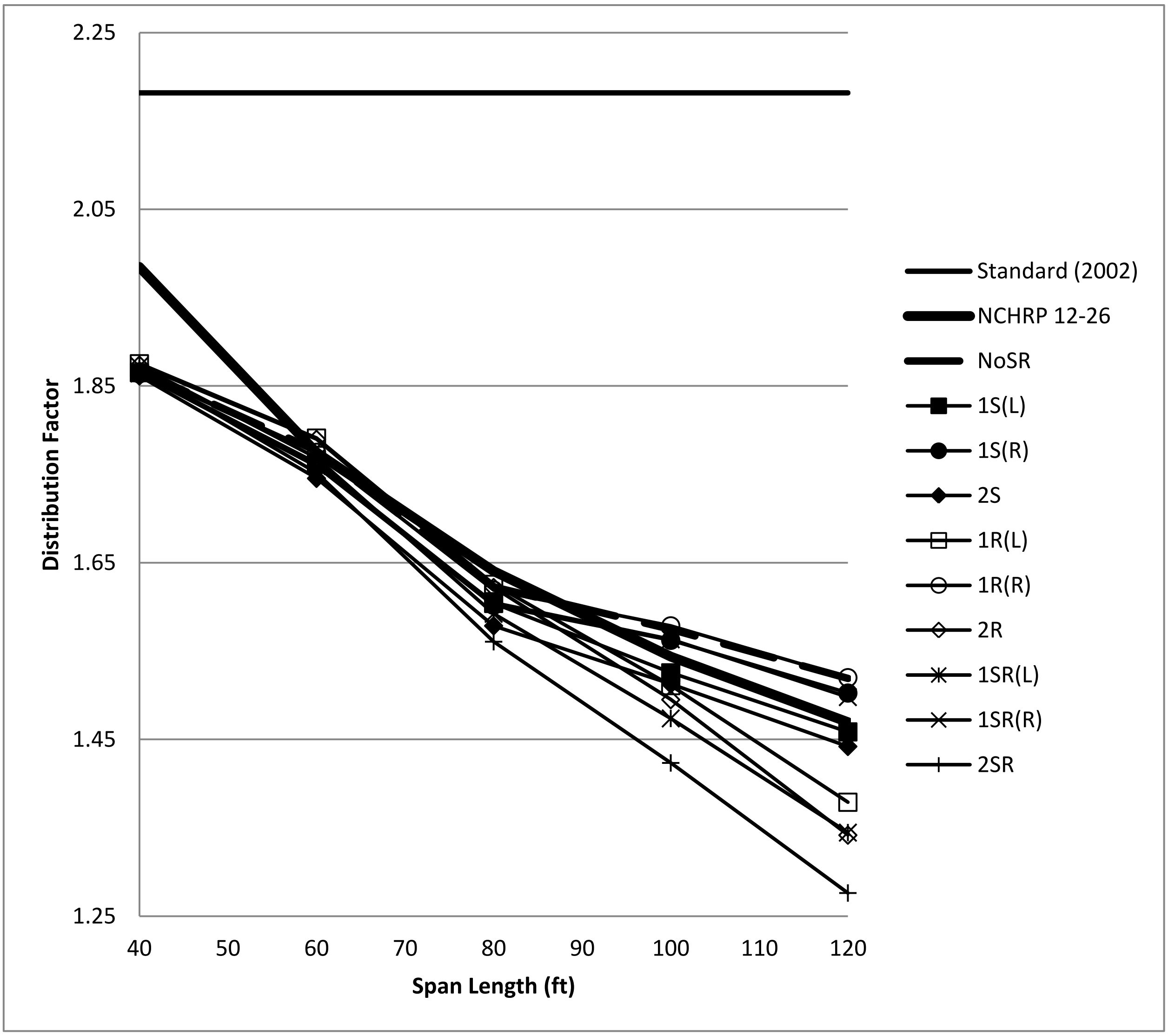


Figure 4.51. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.37. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only)

(a) Maximum bending moment = Mmax (kip-in)

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1,874	2198	2199	2199	2200	2210	2198	2210	2212	2199	2212
	8	1,874	2891	2892	2892	2892	2913	2891	2913	2900	2892	2900
	12	1,874	4235	4230	4227	4222	4252	4232	4249	4247	4228	4240
60	6	4,394	5135	5102	5128	5095	5177	5132	5174	5113	5130	5109
	8	4,394	6633	6597	6624	6560	6669	6631	6663	6636	6629	6603
	12	4,394	9507	9436	9415	9344	9586	9500	9579	9468	9418	9379
80	6	6,545	7210	7139	7171	7100	7191	7212	7194	7063	7180	7033
	8	6,545	9257	9089	9208	9042	9161	9266	9152	9048	9223	8936
	12	6,545	12931	12784	12761	12583	12955	12903	12927	12691	12786	12440
100	6	9,936	10760	10626	10661	10530	10498	10770	10510	10298	10665	10206
	8	9,936	13793	13508	13667	13387	13220	13813	13183	13001	13671	12711
	12	9,936	19148	18546	18996	18392	18381	19196	18187	17930	19010	17319
120	6	14,432	15159	14751	15035	14624	14230	15216	14208	13980	15072	13770
	8	14,432	19219	18756	19011	18545	17754	19286	17697	17448	19113	17018
	12	14,432	26854	25786	26562	25494	24384	26871	23723	23777	26481	22568

(b) Distribution factor = 0.75*DF = 0.75*Mmax/Mo

L (ft)	S (ft)	Mo (kip-in)	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	6	1,874	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.89	0.88	0.89
	8	1,874	1.16	1.16	1.16	1.16	1.17	1.16	1.17	1.16	1.16	1.16
	12	1,874	1.69	1.69	1.69	1.69	1.70	1.69	1.70	1.70	1.69	1.70
60	6	4,394	0.88	0.87	0.88	0.87	0.88	0.88	0.88	0.87	0.88	0.87
	8	4,394	1.13	1.13	1.13	1.12	1.14	1.13	1.14	1.13	1.13	1.13
	12	4,394	1.62	1.61	1.61	1.59	1.64	1.62	1.63	1.62	1.61	1.60
80	6	6,545	0.83	0.82	0.82	0.81	0.82	0.83	0.82	0.81	0.82	0.81
	8	6,545	1.06	1.04	1.06	1.04	1.05	1.06	1.05	1.04	1.06	1.02
	12	6,545	1.48	1.46	1.46	1.44	1.48	1.48	1.48	1.45	1.47	1.43
100	6	9,936	0.81	0.80	0.80	0.79	0.79	0.81	0.79	0.78	0.81	0.77
	8	9,936	1.04	1.02	1.03	1.01	1.00	1.04	1.00	0.98	1.03	0.96
	12	9,936	1.45	1.40	1.43	1.39	1.39	1.45	1.37	1.35	1.43	1.31
120	6	14,432	0.79	0.77	0.78	0.76	0.74	0.79	0.74	0.73	0.78	0.72
	8	14,432	1.00	0.97	0.99	0.96	0.92	1.00	0.92	0.91	0.99	0.88
	12	14,432	1.40	1.34	1.38	1.32	1.27	1.40	1.23	1.24	1.38	1.17

Table 4.38. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26

(a) Percent decrease in DF = [(FEA-AASHTO)/AASHTO] x 100

L (ft)	S (ft)	AASHTO	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.09**	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19
	8	**1.46**	-21	-21	-21	-21	-20	-21	-20	-20	-21	-20
	12	**2.18**	-22	-22	-22	-22	-22	-22	-22	-22	-22	-22
60	**6**	**1.09**	-20	-20	-20	-20	-19	-20	-19	-20	-20	-20
	8	**1.46**	-22	-23	-23	-23	-22	-22	-22	-22	-23	-23
	12	**2.18**	-26	-26	-26	-27	-25	-26	-25	-26	-26	-27
80	**6**	**1.09**	-24	-25	-25	-25	-24	-24	-24	-26	-25	-26
	8	**1.46**	-27	-29	-28	-29	-28	-27	-28	-29	-28	-30
	12	**2.18**	-32	-33	-33	-34	-32	-32	-32	-33	-33	-35
100	**6**	**1.09**	-25	-26	-26	-27	-27	-25	-27	-29	-26	-29
	8	**1.46**	-29	-30	-29	-31	-32	-29	-32	-33	-29	-34
	12	**2.18**	-34	-36	-34	-36	-36	-34	-37	-38	-34	-40
120	**6**	**1.09**	-28	-30	-28	-30	-32	-27	-32	-33	-28	-34
	8	**1.46**	-32	-33	-32	-34	-37	-31	-37	-38	-32	-39
	12	**2.18**	-36	-39	-37	-39	-42	-36	-43	-43	-37	-46

(b) Percent decrease in DF = [(FEA-NCHRP)/NCHRP] x 100

L (ft)	S (ft)	NCHRP	No	1S(L)	1S(R)	2S	1R(L)	1R(R)	2R	1SR(L)	1SR(R)	2SR
40	**6**	**1.20**	-27	-27	-27	-27	-27	-27	-27	-26	-27	-26
	8	**1.48**	-22	-22	-22	-22	-21	-22	-21	-21	-22	-21
	12	**1.98**	-15	-15	-15	-15	-14	-15	-14	-14	-15	-14
60	**6**	**1.08**	-19	-20	-19	-20	-18	-19	-18	-19	-19	-19
	8	**1.32**	-15	-15	-15	-15	-14	-15	-14	-14	-15	-15
	12	**1.77**	-9	-9	-9	-10	-8	-9	-8	-9	-9	-10
80	**6**	**1.01**	-18	-19	-18	-19	-18	-18	-18	-20	-18	-20
	8	**1.23**	-14	-15	-14	-16	-14	-13	-15	-16	-14	-17
	12	**1.64**	-10	-11	-11	-12	-9	-10	-10	-11	-11	-13
100	**6**	**0.95**	-15	-16	-15	-16	-17	-14	-17	-18	-15	-19
	8	**1.16**	-10	-12	-11	-13	-14	-10	-14	-15	-11	-17
	12	**1.54**	-6	-9	-7	-10	-10	-6	-11	-12	-7	-15
120	**6**	**0.91**	-13	-16	-14	-16	-19	-13	-19	-20	-14	-21
	8	**1.10**	-10	-12	-11	-13	-16	-9	-17	-18	-10	-20
	12	**1.47**	-5	-9	-6	-10	-14	-5	-16	-16	-6	-20

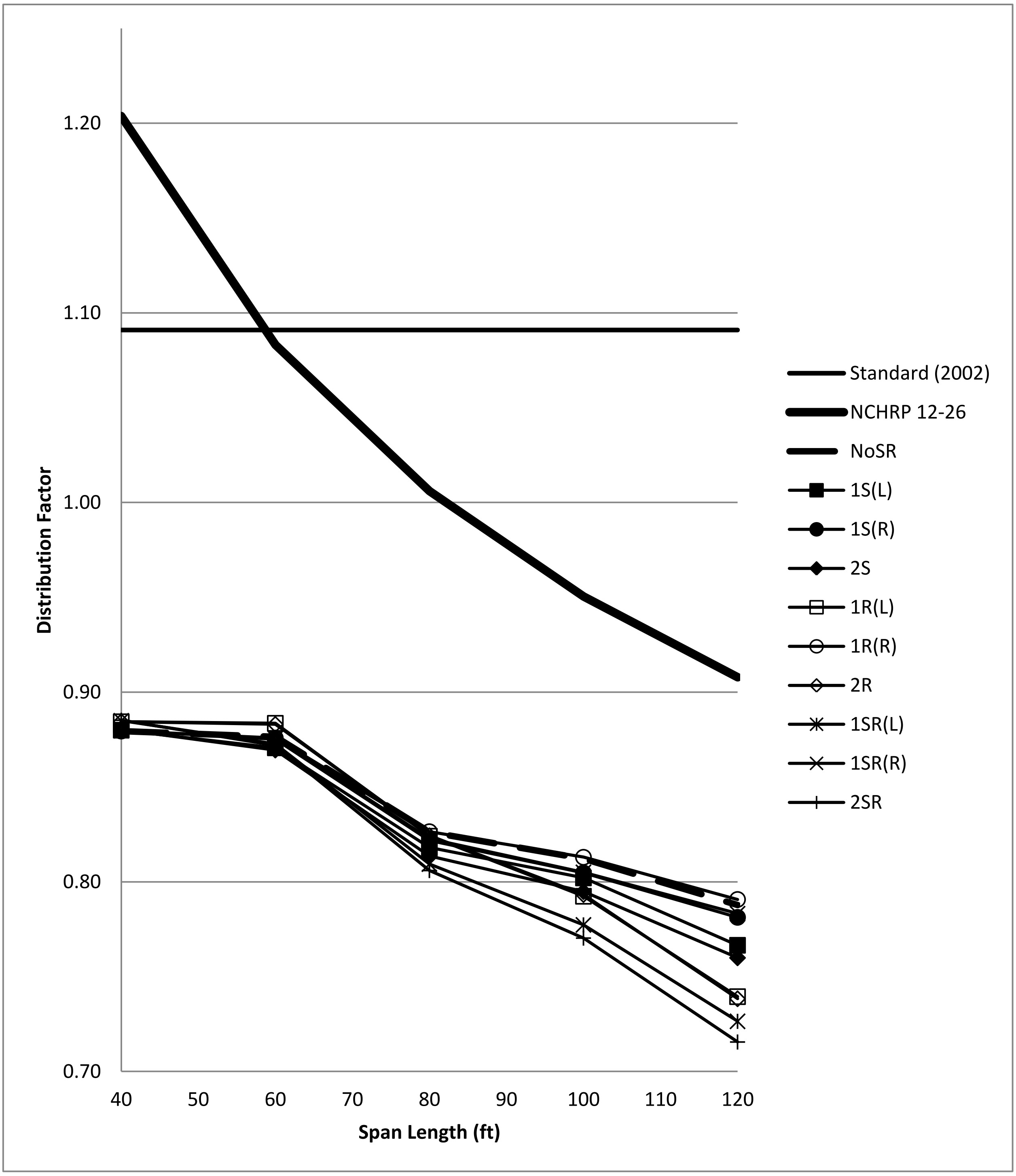


Figure 4.52. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6ft)

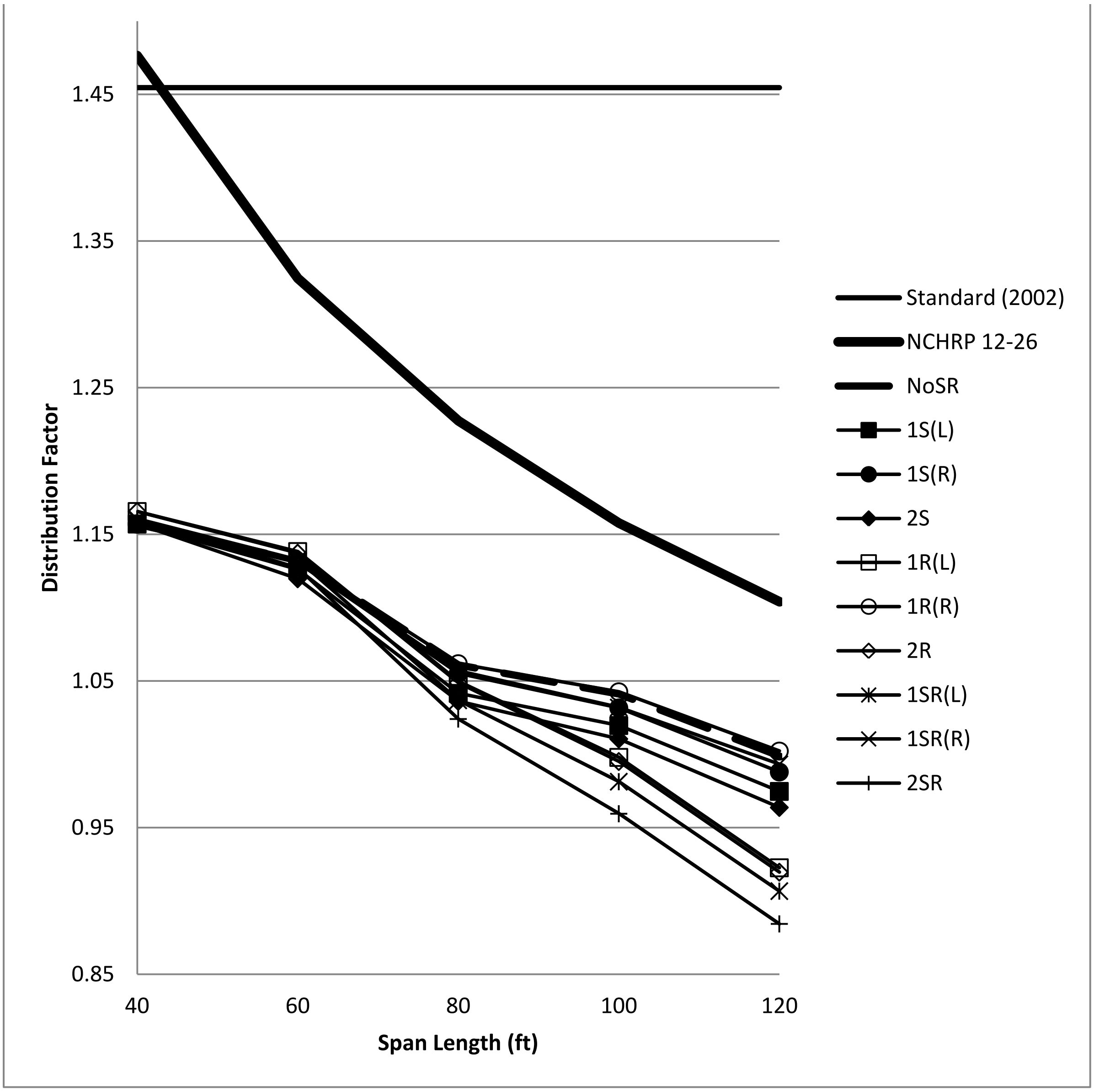


Figure 4.53. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8ft)

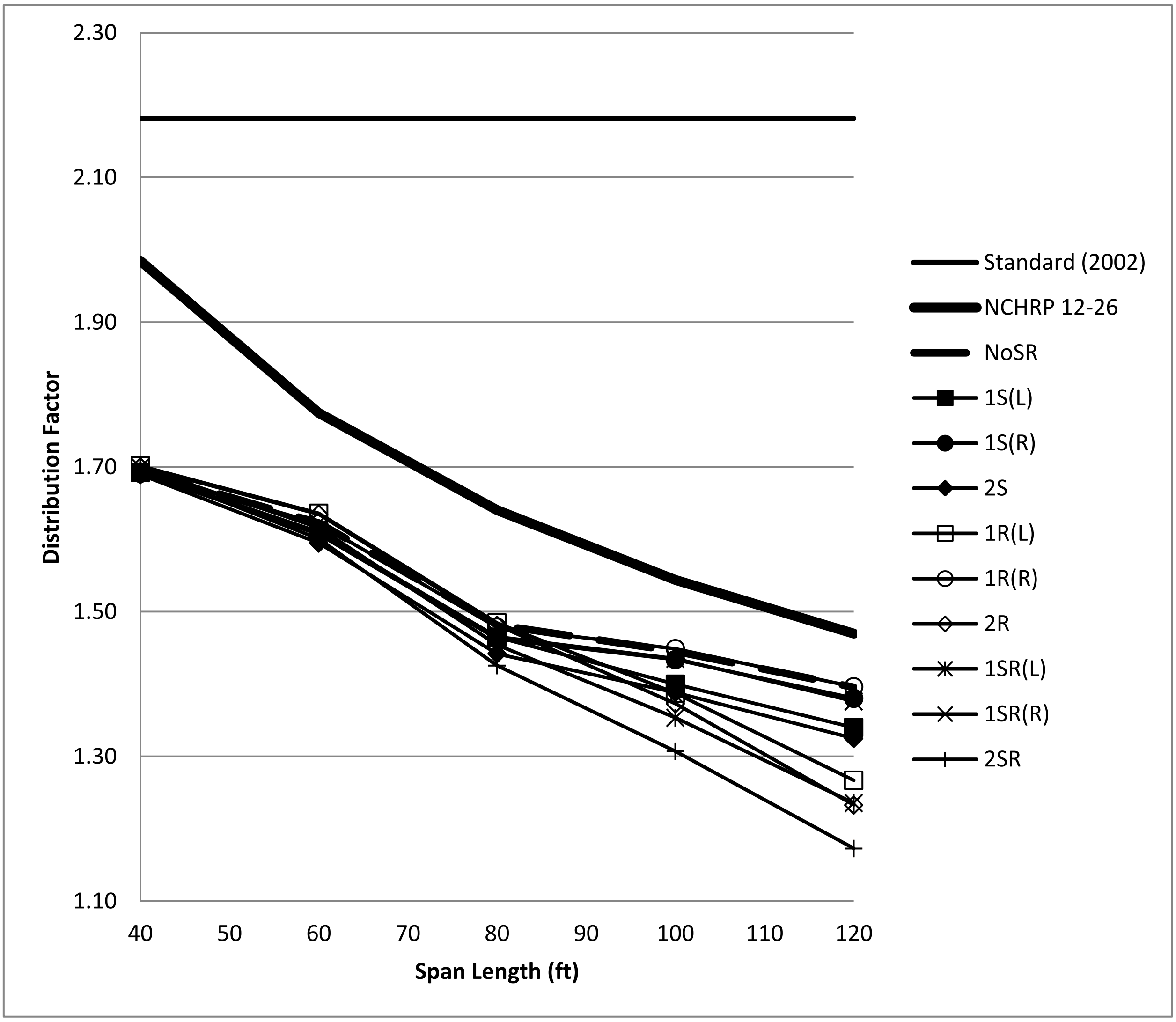


Figure 4.54. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12ft)

Here, a comparison of the FEA distribution factors with (1) will be investigated further since the AASHTO (2002) formula is excessively conservative. The overall percentage decrease of the FEA distribution factors as compared with (1) were generally higher in Table 4.38 (considering steel beams only) as compared to Table 4.36 (interior girders, steel + slab) due to the elimination of the concrete slab effect on maximum bending moments. Therefore, Table 4.36 is used to extract the following general conclusions for interior girders when introducing sidewalks and/or railings to a bridge deck:

1. No sidewalks or railings: for spans up to 40 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 8 ft and about 6 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 9 % higher than FEA distribution factors for girder spacings up to 6 ft and about 4 % higher for girder spacings between 6 and 8 ft while (1) is approximately equal to FEA values for girder spacings between 8 and 12 ft.
2. Sidewalk on one side (left or right): for spans up to 40 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 8 ft and about 6 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 10 % higher than FEA distribution factors for girder spacings up to 6 ft and about 5 % higher for girder spacings between 6 and 8 ft while (1) is approximately equal to FEA values for girder spacings between 8 and 12 ft.
3. Sidewalk on both sides: for spans up to 40 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 8 ft and about 6 %

higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 12 % higher than FEA distribution factors for girder spacings up to 6 ft and about 7 % higher for girder spacings between 6 and 8 ft and about 2 % higher for girder spacings between 8 and 12 ft.

4. Railing on one side (left or right): for spans up to 40 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 8 ft and about 5 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 9 % higher than FEA distribution factors for girder spacings up to 6 ft and about 4 % higher for girder spacings between 6 and 8 ft while (1) is approximately equal to FEA values for girder spacings between 8 and 12 ft.
5. Railing on both sides: for spans up to 40 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 8 ft and about 6 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 12 % higher than FEA distribution factors for girder spacings up to 6 ft and about 8 % higher for girder spacings between 6 and 8 ft and about 1 % higher for spans between 8 and 12 ft.
6. Sidewalk and railing on one side (left or right): for spans up to 40 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 8 ft and about 6 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 120 ft, (1) is about 10 % higher than FEA distribution factors for girder spacings up to 6 ft and about 5 % higher for girder spacings between 6 and 8 ft while (1) is approximately equal to the FEA values for girder spacings between 8 and 12 ft.

7. Sidewalk and railing on both sides: for spans up to 40 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 8 ft and about 6 % higher for girder spacings between 8 and 12 ft. For spans between 40 and 100 ft, (1) is about 15 % higher than FEA distribution factors for girder spacings up to 6 ft and about 9 % higher for girder spacings between 6 and 8 ft and about 1 % higher for spans between 8 and 12 ft. Similarly, (1) is about 15 % higher than FEA results for span lengths between 100 and 120 ft.

Considering the various bridge geometries for any specific sidewalk or railing encountered in the field, a conservative comparison of FEA distribution factors with and without these elements for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without sidewalks and/or railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing sidewalks and/or railings to the bridge deck. First of all, it's important to note that for spans up to 60 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor unless both sidewalks and railings are added simultaneously at both ends where a maximum of 1 % reduction in distribution factor is observed. Otherwise (for spans between 60 and 120 ft), the average reductions in FEA distribution factors when compared to the base case were 1 % when introducing sidewalks on one side; 2 % for spans between 60 and 80 ft (18 and 24 m) and 4 % for spans between 80 and 120 ft (24 and 36 m) when introducing sidewalks on both sides; 0 % when introducing railings on one side; 0 % for spans between 60 and 80 ft (18 and 24 m) and 4 % for spans between 80 and 100 ft (24 m and 30 m) and 8 % for spans between 100 and 120 ft (30 and 36 m) when introducing railings on both sides; 1 % for a combination of sidewalks and railings on one side; and 3 % for spans between 60 and 80 ft (18 and 24 m) and 8 % for spans between 80 and 100 ft (24 and 30 m) and 11 % for spans between 100 and 120 ft (30 and 36 m) for a combination of sidewalks and railings on both sides. The finite-element results show the effects of sidewalks and railings as a function of span length in a given bridge. However, the girder spacing did not have a significant impact on the reduction in the distribution factor. In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings in a bridge superstructure.

CHAPTER 5

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1. Introduction

In this thesis, the influence of sidewalks and railings on multi-span, multi-lane, steel girder bridges was investigated. Different combinations of sidewalks and/or railings on one or both edges of the bridge were considered, in addition to the variation of girder spacing and span length of different bridge cases. Generally, results obtained from the finite-element analysis were much smaller than the values predicted by AASHTO Standard Specifications (2002), and compared favorably with the values predicted by AASHTO Load and Resistance Factor Design (LRFD) design specifications (2010). However, the focus was also to assess the influence of sidewalks and railings cast integrally with the slab deck on the lateral load distribution in the bridge, which is mainly assessed by comparing bridge cases with sidewalks and/or railings with the reference case, namely the one-span and two-equal-spans bridge cases without sidewalks and railings.

Comparisons were therefore made between the FEA results and the reference case for each of the one-span and two-span bridges separately. One-span bridge cases were addressed first, and each of the two-equal-spans cases (positive moment and negative moment) were then addressed separately. In this chapter, results will be summarized and grouped together to come up with a general conclusion for each of the 2-lane, 3-lane and 4-lane bridges. For this purpose, this chapter will be divided into three main sections that summarize and generalize the average reductions in FEA distribution factor upon the introduction of sidewalks and railings on one or both of slab edges for the two-lane, three-lane and four-lane bridges respectively; and last the results of two-, three-, and four-lane bridges will be summed together in a section which summarizes the influence of sidewalks and railings on multi-span multi-lane steel girder bridges.

5.2. Conclusion of Two-Lane Bridges

As obtained in chapter 4 that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor in two-lane, one-span and two-equal-spans bridges (cases of both positive moment and negative moment).

For one-span bridges, the average reductions in FEA distribution factors when compared to the base case were 3 % when introducing sidewalks on one side; 7 % when introducing sidewalks on both sides; 2 % for spans between 40 and 120 ft with girder spacing up to 8 ft and 5 % for spans between 40 and 80 ft with girder spacing between 8 and 12 ft and 12 % for spans between 80 and 120 ft with girder spacing between 8 and 12 ft when introducing railings on one side; 12 % for spans between 40 and 60 ft and 20 % for spans between 60 and 80 ft and 25 % for spans between 80 and 120 ft when introducing railings on both sides; 3 % for spans between 40 and 80 ft with girder spacing up to 8ft and 11 % for spans between 40 and 80 ft with girder spacing between 8 and 12 ft and 8 % for spans between 80 and 120 ft with girder spacing up to 8 ft and 17 % for spans between 80 and 120 ft with girder spacing between 8 and 12 ft for a combination of sidewalks and railings on one side; and 19 % for spans between 40 and 60 ft and 25 % for spans between 60 and 80 ft and 30 % for spans between 80 and 120 ft for a combination of sidewalks and railings on both sides.

For two-equal-spans bridges (positive moment), the average reductions in FEA distribution factors when compared to the base case were 3 % when introducing sidewalks on one side; 7 % when introducing sidewalks on both sides; 1 % for spans between 40 and 120 ft with girder spacing up to 8ft and 4 % for spans between 40 and 80 ft with girder spacing between 8 and 12 ft and 10 % for spans between 80 and 120 ft with girder spacing between 8 and 12 ft when introducing railings on one side; 10 % for spans between 40 and 60 ft and 17 % for spans between 60 and 80 ft and 23 % for spans between 80 and 120 ft when introducing railings on both sides; 3 % for spans between 40 and 80 ft with girder spacing up to 8 ft and 9 % for spans between 40 and 80 ft with girder spacing between 8 and 12 ft and 6 % for spans between 80 and 120 ft with girder spacing up to 8 ft and 16 % for spans between 80 and 120 ft with girder spacing between 8 and 12 ft for a combination of sidewalks and railings on one side; and 14 % for spans between 40 and 60 ft and 23 % for spans between 60 and 80 ft and 29 % for spans between 80 and 120 ft for a combination of sidewalks and railings on both sides.

For two-equal-spans bridges (negative moment), the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 6 % when introducing sidewalks on both sides; 1 % for spans between 40 and 120 ft with girder spacing up to 8 ft and 3 % for spans between 40 and 80 ft with girder spacing between 8 and 12 ft and 8 % for spans between 80 and 120 ft with girder spacing between 8 and 12 ft when introducing railings on one side; 7 % for spans between 40 and 60 ft and 12 % for spans between 60 and 80 ft and 22 % for spans between 80 and 120 ft when introducing railings on both sides; 2 % for spans between 40 and 80 ft with girder spacing up to 8 ft and 6 % for spans between 40 and 80 ft with girder spacing between 8 and 12 ft and 4 % for spans between 80 and 120 ft with girder spacing up to 8 ft and 10 % for spans between 80 and 120 ft with girder spacing between 8 and 12 ft for a combination of sidewalks and railings on one side; and 10 % for spans between 40 and 60 ft and 17 % for spans between 60 and 80 ft and 25 % for spans between 80 and 120 ft for a combination of sidewalks and railings on both sides.

Table 5.1 shows the percentage decrease of the FEA distribution factors compared to the reference case (bridge without sidewalks and railings) for all three cases of 2-lane bridges, namely the one-span, two-equal-spans (positive moment) and two-equal-spans (negative moment) bridges. The labels in the leftmost column of the table represent the different combinations of sidewalks and/or railings on the bridge as illustrated below:

1S: presence of Sidewalk on one side

2 S: presence of Sidewalks on both sides

1 R: presence of Railing on one side

2 R: presence of Railings on both sides

1 SR: combination of Sidewalk and Railing on one side

2 SR: combination of Sidewalks and Railings on both sides

Looking at Table 5.1, one can simply note that the percentage reduction values for one-span positive moment and two-span positive moment are closer to each other than they are to the reduction values of two-span negative moment. Therefore, when calculating the average percentage reduction in the distribution factor that represents all cases of two-lane bridges (last column in the table), the following procedure was followed. The average of the values of one-span positive and two-span positive was calculated first, and then this value was averaged with the value of the two-span negative in order to calculate the final average percentage reduction in the distribution

Table 5.1. Percentage Reduction in Distribution Factor for Two-Lane Bridges

	Girder Spacing	Span Length	One-Span Positive Moment (%)	Two-Span Positive Moment (%)	Two-Span Negative Moment (%)	Average Percentage Reduction in DF (%)
1S	0 to 12 ft	40 to 120 ft	3	3	2	**2**
2S	0 to 12 ft	40 to 120 ft	7	7	6	**6**
1R	0 to 8 ft	40 to 120 ft	2	1	1	**1**
	8 to 12 ft	40 to 80 ft	5	4	3	**3**
		80 to 120 ft	12	10	8	**9**
2R	0 to 12 ft	40 to 60 ft	12	10	7	**9**
		60 to 80 ft	20	17	12	**15**
		80 to 120 ft	25	23	22	**23**
1SR	0 to 8 ft	40 to 80 ft	3	3	2	**2**
		80 to 120 ft	8	6	4	**5**
	8 to 12 ft	40 to 80 ft	11	9	6	**8**
		80 to 120 ft	17	16	10	**13**
2SR	0 to 12 ft	40 to 60 ft	19	14	10	**13**
		60 to 80 ft	25	23	17	**20**
		80 to 120 ft	30	29	25	**27**

factor for the two-lane bridges which is observed in the last column in the table (numbers in red).

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. Hence, looking at the values in the last column of Table 5.1, 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.

5.3. Conclusion of Three-Lane Bridges

As obtained in chapter 4 that for spans up to 40 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor in three-lane, one-span and two-equal-spans bridges (cases of both positive moment and negative moment).

For one-span bridges, the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 5 % when introducing sidewalks on both sides; 0 % for spans between 40 and 80 ft and 0 % for spans between 80 and 120 ft with girder spacing up to 6 ft and 3 % for spans between 80 and 120 ft with girder spacing between 6 and 12 ft when introducing railings on one side; 3 % for spans between 40 and 60 ft and 8 % for spans between 60 and 80 ft and 16 % for spans between 80 and 120 ft when introducing railings on both sides; 2 % for spans between 40 and 80 ft and 2 % for spans between 80 and 120 ft with girder spacing up to 6ft and 6 % for spans between 80 and 120 ft with girder spacing between 6 and 12 ft for a combination of sidewalks and railings on one side; and 8 % for spans between 40 and 60 ft and 13 % for spans between 60 and 80 ft and 20 % for spans between 80 and 120 ft for a combination of sidewalks and railings on both sides.

For two-equal-spans bridges (positive moment), the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 5 % when introducing sidewalks on both sides; 0 % for spans between 40 and 80 ft and 0 % for spans between 80 and 120 ft with girder spacing up to 6 ft and 3 % for spans between 80 and 120 ft with girder spacing between 6 and 12 ft when introducing railings on one side; 3 % for spans between 40 and 60 ft and 7 % for spans between 60 and 80 ft and 13% for spans between 80 and 120 ft when introducing railings on both sides; 2 % for spans between 40 and 80 ft and 2 % for spans between 80 and 120 ft with girder spacing up to 6ft and 6 % for spans between 80 and 120 ft with girder spacing between 6 and 12 ft for a combination of sidewalks and railings on one side; and 7 % for spans between 40 and 60 ft and 11 % for spans between 60 and 80 ft and 17 % for spans between 80 and 120 ft for a combination of sidewalks and railings on both sides.

For two-equal-spans bridges (negative moment), the average reductions in FEA distribution factors when compared to the base case were 2 % when introducing sidewalks on one side; 4 % when introducing sidewalks on both sides; 0 % for spans between 40 and 80 ft and 0 % for spans between 80 and 120 ft with girder spacing up to 6 ft and 2 % for spans between 80 and 120 ft with girder spacing between 6 and 12 ft when introducing railings on one side ; 1 % for spans between 40 and 60 ft and 4 % for spans between 60 and 80 ft and 8 % for spans between 80 and 120 ft when introducing railings on both sides; 1 % for spans between 40 and 80 ft and 1 % for spans between 80 and 120 ft with girder spacing up to 6ft and 4 % for spans between 80 and 120 ft with girder spacing between 6 and 12 ft for a combination of sidewalks and railings on one side; and 5 % for spans between 40 and 60 ft and 8 % for spans between 60 and 80 ft and 13 % for spans between 80 and 120 ft for a combination of sidewalks and railings on both sides.

Table 5.2 shows the percentage decrease of the FEA distribution factors compared to the reference case (bridge without sidewalks and railings) for all three cases of 3-lane bridges, namely the one-span, two-equal-spans (positive moment) and two-equal-spans (negative moment) bridges.

Looking at Table 5.2, one can simply note that the percentage reduction values for one-span positive moment and two-span positive moment are closer to each other than they are to the reduction values of two-span negative moment. Therefore, when calculating the average percentage reduction in the distribution factor that represents all cases of three-lane bridges (last column in the table), the following procedure was followed. The average of the values of one-span positive and two-span positive was calculated first, and then this value was averaged with the value of the two-span negative in order to calculate the final average percentage reduction in the distribution factor for the three-lane bridges which is observed in the last column in the table (numbers in red).

Table 5.2. Percentage Reduction in Distribution Factor for Three-Lane Bridges

	Girder Spacing	Span Length	One-Span Positive Moment	Two-Span Positive Moment	Two-Span Negative Moment	Average Percentage Reduction in DF
1S	0 to 12 ft	40 to 120 ft	2	2	2	**2**
2S	0 to 12 ft	40 to 120 ft	5	5	4	**4**
1R	0 to 6 ft	40 to 120 ft	0	0	0	**0**
	6 to 12 ft	40 to 80 ft	0	0	0	**0**
		80 to 120 ft	3	3	2	**2**
2R	0 to 12 ft	40 to 60 ft	3	3	1	**2**
		60 to 80 ft	8	7	4	**5**
		80 to 120 ft	16	13	8	**11**
1SR	0 to 6 ft	40 to 120 ft	2	2	1	**1**
	6 to 12 ft	40 to 80 ft	2	2	1	**1**
		80 to 120 ft	6	6	4	**5**
2SR	0 to 12 ft	40 to 60 ft	8	7	5	**6**
		60 to 80 ft	13	11	8	**10**
		80 to 120 ft	20	17	13	**15**

Since the percentage reduction in distribution factor implies an increase in the load-carrying capacity due to the presence of sidewalks and/or railings, one can notice a remarkable impact of adding sidewalks and railings on three-lane bridges, where a 15 % increase in load-carrying capacity can be reached. Looking at Tables 5.1 and 5.2, one can simply note that the effect of sidewalks and railing was more pronounced in two-lane bridges compared to three-lane bridges. This is logical as for two-lane bridges, due to the lateral positioning of trucks on the bridge's cross-section, the maximum moment occurs mostly in the exterior girder or the one next to it, and hence adding a sidewalk and/or a railing would assist the exterior girder in resisting the load and hence would decrease the distribution factor significantly, implying a significant increase in the bridge's load-carrying capacity. However, for three-lane bridges, the maximum moment occurs mostly in one of the interior girders and hence the effect of the sidewalks and railings is less pronounced as they're located on the edge.

5.4. Conclusion of Four-Lane Bridges

As obtained in chapter 4 that for spans up to 60 ft, the addition of sidewalks or railings has a negligible effect on the distribution factor in four-lane, one-span and two-equal-spans bridges (cases of both positive moment and negative moment).

For one-span bridges, the average reductions in FEA distribution factors when compared to the base case were 1 % when introducing sidewalks on one side; 3 % for spans between 60 and 80 ft and 5 % for spans between 80 and 120 ft when introducing sidewalks on both sides; 0 % when introducing railings on one side; 5 % for spans between 60 and 80 ft and 10 % for spans between 80 and 100 ft and 16 % for spans between 100 and 120 ft when introducing railings on both sides; 1% for a combination of sidewalks and railings on one side; and 8 % for spans between 60 and 80 ft and 14 % for spans between 80 and 100 ft and 20 % for spans between 100 and 120 ft for a combination of sidewalks and railings on both sides.

For two-equal-spans bridges (positive moment), the average reductions in FEA distribution factors when compared to the base case were 1 % when introducing sidewalks on one side; 3 % for spans between 60 and 80 ft and 5 % for spans between 80 and 120 ft when introducing sidewalks on both sides; 0 % when introducing railings on one side; 3 % for spans between 60 and 80 ft and 8 % for spans between 80 and 100 ft and 13 % for spans between 100 and 120 ft when introducing railings on both sides; 1 % for a combination of sidewalks and railings on one side; and 6 % for spans between 60 and 80 ft and 12 % for spans between 80 and 100 ft and 18 % for spans between 100 and 120 ft for a combination of sidewalks and railings on both sides.

For two-equal-spans bridges (negative moment), the average reductions in FEA distribution factors when compared to the base case were 1 % when introducing sidewalks on one side; 2 % for spans between 60 and 80 ft and 4 % for spans between 80 and 120 ft when introducing sidewalks on both sides; 0 % when introducing railings on one side; 0 % for spans between 60 and 80 ft and 4 % for spans between 80 and 100 ft and 8 % for spans between 100 and 120 ft when introducing railings on both sides; 1 % for a combination of sidewalks and railings on one side; and 3 % for spans between 60 and 80 ft and 8 % for spans between 80 and 100 ft and 11 % for spans between 100 and 120 ft for a combination of sidewalks and railings on both sides.

Table 5.3 shows the percentage decrease of the FEA distribution factors compared to the reference case (bridge without sidewalks and railings) for all three cases of 4-lane bridges, namely the one-span, two-equal-spans (positive moment) and two-equal-spans (negative moment) bridges.

Looking at Table 5.3, one can simply note that the percentage reduction values for one-span positive moment and two-span positive moment are closer to each other than they are to the reduction values of two-span negative moment. Therefore, when calculating the average percentage reduction in the distribution factor that represents all cases of four-lane bridges (last column in the table), the following procedure was followed. The average of the values of one-span positive and two-span positive was calculated first, and then this value was averaged with the value of the two-span negative in order to calculate the final average percentage reduction in the distribution factor for the four-lane bridges which is observed in the last column in the table (numbers in red).

Observing the results in Table 5.3 is really impressive as we reached 15 % reduction in the distribution factor in four-lane bridges. The main focus is that in four-lane bridges, the maximum moment always occurs in one of the interior girders, and still a significant reduction in the distribution factor is observed, proving that although the sidewalks and railings are added at the shoulders (edges of the bridge), they still have their impact on redistributing the lateral load in the bridge superstructure and thus they still increase the load-carrying capacity of the bridge.

Table 5.3. Percentage Reduction in Distribution Factor for Four-Lane Bridges

	Girder Spacing	Span Length	One-Span Positive Moment	Two-Span Positive Moment	Two-Span Negative Moment	Average Percentage Reduction in DF
1S	0 to 12 ft	60 to 120 ft	1	1	1	**1**
2S	0 to 12 ft	60 to 80 ft	3	3	2	**2**
		80 to 120 ft	5	5	4	**4**
1R	0 to 12 ft	60 to 120 ft	0	0	0	**0**
2R	0 to 12 ft	60 to 80 ft	5	3	0	**2**
		80 to 100 ft	10	8	4	**6**
		100 to 120 ft	16	13	8	**11**
1SR	0 to 12 ft	60 to 120 ft	1	1	1	**1**
2SR	0 to 12 ft	60 to 80 ft	8	6	3	**5**
		80 to 100 ft	14	12	8	**10**
		100 to 120 ft	20	18	11	**15**

5.5. Conclusion and Recommendations

The last step consists of summing the results obtained in the previous sections for two-, three-, and four-lane bridges to obtain a general assessment of the increase in the load-carrying capacity for all steel girder bridges analyzed upon the addition of sidewalks and/or railings on either or both of slab edges. Therefore, the results obtained earlier in Tables 5.1, 5.2, and 5.3 are summarized in Table 5.4 which produces the ranges of percentage decrease in distribution factor – the increase in load-carrying capacity – of one-span and two-span bridges (both positive and negative moment cases) for all different combinations of girder spacing, span length, bridge width, and S/R additions on either or both of slab edges.

Table 5.4 shows the efficiency of different combinations of sidewalks and/or railings, and one can simply observe that some of the S/R cases are inefficient and not worth to be considered, mainly the addition of one sidewalk on one side of the bridge (case 1S) which shows almost no reduction for all bridge cases; compared to the most efficient case, which is logically obtained upon the addition of sidewalks and railings on both of slab edges (case 2SR) as reflected in the tabulated results. One point to note also from Table 5.4 that the addition of railings is more effective compared to the addition of sidewalks; this is observed when comparing the 1S case to the 1R case and comparing the 2S case to the 2R case. In this context, although the railing is not supported at both of its ends, it extends above the slab deck and acts like an inverted beam, contributing further to the stiffness of the bridge deck than the simple thickening of the bridge deck upon the addition of sidewalks. Furthermore, Tables 5.1-5.3 show that more reduction in the distribution factor is observed for one-span bridges than for two-equal-spans bridges (positive moment), which in turn showed greater reduction values than two-equal-spans bridges (negative moment). In addition, more reduction is observed for larger girder spacing, longer spans, and for a less number of lanes. As explained earlier, the latter sounds logical since sidewalks and railings are added at the slab edges (shoulders) and thus their influence on interior girders decreases as the number of lanes (bridge width) increases. Last, the efficiency of reduction in wheel load distribution factor (or increase in load-carrying capacity) for the different parameters investigated can be described by the following order:

- Number of spans : One-Span > Two-Span Positive > Two-Span Negative
- Number of lanes : Two-Lane > Three-Lane > Four-Lane
- Girder spacing : 12ft > 8ft > 6ft
- Span length : 120ft > 100ft > 80ft > 60ft > 40ft

Before concluding, one should keep in mind that the results obtained in this research are based on analytical modeling of bridges, as the values are obtained from the finite-element analysis of different bridges. However, these FEA-based results have set the ground for future experimental testing of the load-carrying capacity of bridges with sidewalks and railings. Further, the percentages observed in tables 5.1-5.4 are impressive and prove to be a strong argument proposing to include the effect of Sidewalks and Railings in AASHTO Specifications, namely the LRFD Design Specifications, which include several parameters that are believed to influence the load distribution in steel girder bridges. This step would require further testing of steel girder bridges, and would require some kind of experimental testing, such as field load testing of existing bridges to be able to formulate the effect of sidewalks and railings on the wheel load distribution in steel girder bridges.

Table 5.4. Summary of Reduction with S/R for All Bridges

% Decrease	Girder Spacing	Span Length	Two-Lane	Three-Lane	Four-Lane
NoSR	6-12 ft	40-120 ft	0	0	0
1S	6-12 ft	40-120 ft	-	-	-
1R	6-12 ft	40-120 ft	0-10	-	-
1SR	6-12 ft	40-120 ft	0-15	0-5	-
2S	6-12 ft	40-120 ft	5-10	5	0-5
2R	6-12 ft	40-120 ft	10-25	0-15	0-15
2SR	6-12 ft	40-120 ft	10-30	5-20	5-20

To conclude, the results shown in Tables 5.1-5.4 prove that "Sidewalks and Railings", when cast integrally with the bridge deck, become a part of the concrete section that resists the loadings on the bridge and thus add to the stiffness of the bridge superstructure, leading to as much as 25 % increase in the load-carrying capacity of the bridge superstructure in some cases. Hence, since most of the bridges are cast with sidewalks and railings, it would be recommended that these latter be casted integrally with the bridge deck in order to increase the moment capacity of the bridge.

APPENDIX 1

TABULATED RESULTS OF THE BENDING MOMENTS OF DIFFERENT BRIDGE COMPONENTS OBTAINED FROM THE FINITE ELEMENT ANALYSIS

Table A.1. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 2 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Girder 5 (7)**	**Total (8)**
No SR	Girder	514.8	503.3	472.9	405.4	311.5	2208.0
	Slab	31.9	32.6	23.8	16.4	17.0	121.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	546.7	535.9	496.7	421.9	328.5	2329.6
1S(L)	Girder	473.1	477.1	460.9	403.2	317.6	2132.0
	Slab	108.5	32.5	23.9	16.7	16.1	197.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	581.6	509.6	484.8	420.0	333.7	2329.6
1S(R)	Girder	506.0	492.6	459.4	392.1	313.7	2163.9
	Slab	31.4	31.9	22.3	13.9	66.3	165.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	537.4	524.5	481.7	406.0	380.0	2329.6
2S	Girder	465.4	467.6	448.1	389.7	316.9	2087.7
	Slab	106.9	31.9	22.6	14.3	66.2	241.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	572.3	499.5	470.8	404.0	383.1	2329.6
1R(L)	Girder	348.3	407.9	429.0	398.8	336.2	1920.1
	Slab	26.6	32.6	24.6	17.6	16.7	118.0
	Railing	291.5	0.0	0.0	0.0	0.0	291.5
	Total	666.3	440.6	453.6	416.3	352.9	2329.6
1R(R)	Girder	526.6	498.2	447.6	353.9	227.9	2054.1
	Slab	32.7	32.9	23.6	14.9	14.7	118.7
	Railing	0.0	0.0	0.0	0.0	156.7	156.7
	Total	559.3	531.1	471.1	368.8	399.4	2329.6
2R	Girder	353.6	398.1	398.6	339.0	238.0	1727.3
	Slab	27.4	33.1	24.5	16.0	14.2	115.2
	Railing	305.5	0.0	0.0	0.0	181.6	487.1
	Total	686.5	431.3	423.1	355.0	433.8	2329.6
1SR(L)	Girder	342.9	390.7	413.3	387.4	328.3	1862.5
	Slab	78.5	31.2	23.9	17.2	15.0	165.8
	Railing	301.4	0.0	0.0	0.0	0.0	301.4
	Total	722.7	421.9	437.2	404.6	343.3	2329.6
1SR(R)	Girder	512.1	482.2	429.1	338.4	234.7	1996.5
	Slab	31.9	31.8	21.5	11.4	49.4	146.0
	Railing	0.0	0.0	0.0	0.0	187.1	187.1
	Total	544.0	514.0	450.6	349.9	471.2	2329.6
2SR	Girder	338.0	371.4	370.9	316.9	235.8	1633.1
	Slab	78.9	31.1	22.3	13.0	48.7	194.1
	Railing	303.4	0.0	0.0	0.0	199.1	502.5
	Total	720.3	402.6	393.3	329.9	483.6	2329.6

Table A.2. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 2 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Total (7)**
No SR	Girder	622.9	614.6	532.7	404.2	2174.3
	Slab	39.5	50.6	39.0	26.1	155.3
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	662.4	665.2	571.7	430.3	2329.6
1S(L)	Girder	569.6	584.5	522.4	408.3	2084.8
	Slab	130.7	50.0	39.1	25.0	244.8
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	700.4	634.5	561.4	433.3	2329.6
1S(R)	Girder	611.1	597.7	510.1	399.5	2118.4
	Slab	38.9	49.2	35.6	87.5	211.2
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	650.0	646.9	545.7	487.0	2329.6
2S	Girder	559.2	569.2	500.5	400.3	2029.3
	Slab	128.7	48.8	36.0	86.9	300.4
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	687.9	618.0	536.4	487.3	2329.6
1R(L)	Girder	408.7	509.3	500.2	426.4	1844.7
	Slab	32.1	49.9	40.0	25.8	147.8
	Railing	337.1	0.0	0.0	0.0	337.1
	Total	777.9	559.2	540.2	452.2	2329.5
1R(R)	Girder	634.0	593.4	469.5	285.3	1982.2
	Slab	40.4	50.6	36.9	21.9	149.8
	Railing	0.0	0.0	0.0	197.6	197.6
	Total	674.4	644.0	506.4	504.8	2329.6
2R	Girder	411.2	482.1	428.7	290.6	1612.6
	Slab	32.9	50.1	38.0	21.3	142.2
	Railing	352.2	0.0	0.0	222.6	574.8
	Total	796.3	532.2	466.6	534.5	2329.6
1SR(L)	Girder	400.7	484.0	480.6	413.1	1778.5
	Slab	92.0	47.4	38.9	23.5	201.8
	Railing	349.3	0.0	0.0	0.0	349.3
	Total	842	531.5	519.5	436.6	2329.6
1SR(R)	Girder	613.7	569.2	443.1	289.8	1915.8
	Slab	39.2	48.4	32.3	62.9	182.8
	Railing	0.0	0.0	0.0	231.0	231.0
	Total	652.9	617.6	475.4	583.7	2329.6
2SR	Girder	390.0	443.9	393.0	282.9	1509.9
	Slab	91.6	46.5	33.6	61.0	232.7
	Railing	348.3	0.0	0.0	238.8	587.1
	Total	829.9	490.4	426.6	582.8	2329.6

Table A.3. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 2 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Total (6)
No SR	Girder	795.0	768.0	560.1	2123.1
	Slab	71.1	90.6	44.9	206.5
	Railing	0.0	0.0	0.0	0.0
	Total	866.1	858.6	605.0	2329.6
1S(L)	Girder	723.1	733.2	557.3	2013.5
	Slab	184.3	89.0	42.9	316.1
	Railing	0.0	0.0	0.0	0.0
	Total	907.4	822.1	600.1	2329.6
1S(R)	Girder	777.0	732.6	539.2	2048.8
	Slab	69.7	85.5	125.8	280.9
	Railing	0.0	0.0	0.0	0.0
	Total	846.7	818.0	664.9	2329.6
2S	Girder	707.1	700.7	533.1	1940.9
	Slab	180.8	84.4	123.5	388.7
	Railing	0.0	0.0	0.0	0.0
	Total	887.9	785.1	656.6	2329.6
1R(L)	Girder	503.9	661.1	568.3	1733.4
	Slab	58.9	88.6	43.5	190.9
	Railing	405.3	0.0	0.0	405.3
	Total	968.1	749.7	611.8	2329.6
1R(R)	Girder	797.8	695.3	378.0	1871.1
	Slab	71.4	86.8	36.9	195.1
	Railing	0.0	0.0	263.4	263.4
	Total	869.2	782.2	678.2	2329.6
2R	Girder	497.3	581.9	369.9	1449.0
	Slab	59.2	85.0	35.2	179.4
	Railing	417.9	0.0	283.3	701.2
	Total	915.2	666.9	653.2	2329.6
1SR(L)	Girder	489.0	622.3	544.4	1655.7
	Slab	129.1	84.5	39.8	253.4
	Railing	420.5	0.0	0.0	420.5
	Total	1038.6	706.8	584.2	2329.6
1SR(R)	Girder	767.0	652.3	374.5	1793.9
	Slab	69.1	79.8	87.4	236.2
	Railing	0.0	0.0	299.5	299.5
	Total	836.1	732.1	761.4	2329.6
2SR	Girder	465.4	522.5	350.2	1338.1
	Slab	126.4	76.7	82.0	285.1
	Railing	411.3	0.0	295.2	706.5
	Total	1003.1	599.2	727.4	2329.6

Table A.4. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 3 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Girder 6 (8)	Girder 7 (9)	Total (10)
No SR	Girder	548.2	564.3	564.6	533.0	464.6	369.1	260.4	3304.2
	Slab	33.5	35.9	30.2	35.1	29.0	17.6	9.0	190.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	581.7	600.2	594.7	568.1	493.6	386.7	269.4	3494.4
1S(L)	Girder	514.8	538.1	549.1	525.1	461.8	370.0	264.2	3223.1
	Slab	115.8	35.4	30.1	35.2	27.8	17.7	9.2	271.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	630.6	573.5	579.2	560.3	489.6	387.8	273.4	3494.4
1S(R)	Girder	545.1	559.1	557.2	523.2	452.1	359.4	272.5	3268.6
	Slab	33.4	35.6	29.6	34.2	27.9	15.1	50.1	225.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	578.5	594.7	586.8	557.4	480.0	374.5	322.6	3494.4
2S	Girder	511.3	533.2	542.1	515.6	449.5	360.2	275.3	3187.2
	Slab	115.2	35.2	29.6	34.3	26.7	15.3	50.8	307.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	626.6	568.3	571.7	550.0	476.3	375.5	326.1	3494.4
1R(L)	Girder	381.7	466.2	513.3	511.2	462.2	379.9	281.0	2995.6
	Slab	28.4	36.3	31.4	36.5	28.6	18.6	9.9	189.6
	Railing	309.2	0.0	0.0	0.0	0.0	0.0	0.0	309.2
	Total	719.2	502.6	544.8	547.7	490.8	398.4	291.0	3494.5
1R(R)	Girder	556.1	568.0	562.7	523.0	442.2	329.3	199.2	3180.5
	Slab	34.0	36.3	30.6	35.5	29.1	16.8	6.5	188.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	125.0	125.0
	Total	590.1	604.3	593.4	558.5	471.4	346.1	330.7	3494.4
2R	Girder	386.8	468.4	510.4	499.8	437.4	335.3	211.5	2849.7
	Slab	28.8	36.8	32.0	37.0	28.8	17.8	7.2	188.5
	Railing	315.8	0.0	0.0	0.0	0.0	0.0	140.5	456.3
	Total	731.4	505.2	542.5	536.8	466.2	353.2	359.2	3494.4
1SR(L)	Girder	378.9	446.6	494.8	498.1	453.7	375.1	279.7	2926.9
	Slab	84.8	34.4	30.5	36.0	26.7	18.5	9.9	240.7
	Railing	326.8	0.0	0.0	0.0	0.0	0.0	0.0	326.8
	Total	790.5	481.0	525.2	534.1	480.4	393.6	289.6	3494.5
1SR(R)	Girder	552.5	561.4	552.9	509.6	425.0	314.9	208.4	3124.6
	Slab	33.8	36.0	30.0	34.3	27.7	13.7	35.4	210.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	158.9	158.9
	Total	586.4	597.3	582.9	543.9	452.7	328.6	402.7	3494.5
2SR	Girder	380.3	443.2	483.3	474.5	412.7	316.5	217.5	2728.0
	Slab	85.6	34.7	30.5	35.4	25.6	14.8	38.0	264.7
	Railing	330.5	0.0	0.0	0.0	0.0	0.0	171.3	501.8
	Total	796.4	477.9	513.8	509.9	438.3	331.3	426.8	3494.5

Table A.5. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 3 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Girder 6 (8)	Total (9)
No SR	Girder	464.2	591.4	658.5	636.8	532.7	370.5	3254.0
	Slab	25.8	54.2	55.8	48.4	38.4	15.6	238.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	490.0	645.6	714.3	685.2	571.1	386.0	3492.2
1S(L)	Girder	462.6	565.8	638.8	624.3	525.8	368.7	3185.9
	Slab	22.5	51.1	54.4	85.6	38.1	15.5	267.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	485.1	616.9	693.2	709.8	563.9	384.1	3453.1
1S(R)	Girder	460.2	582.4	644.4	617.2	512.7	383.5	3200.2
	Slab	56.7	53.6	54.6	46.7	34.2	77.3	323.1
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	516.9	636.0	699.0	663.8	546.8	460.8	3523.3
2S	Girder	457.6	557.3	625.6	605.5	506.3	380.6	3132.8
	Slab	53.3	50.7	53.3	83.5	34.0	76.8	351.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	510.9	608.0	678.9	689.0	540.3	457.3	3484.4
1R(L)	Girder	318.2	506.0	618.0	624.0	536.4	385.7	2988.4
	Slab	23.9	53.2	56.5	47.2	39.3	16.2	236.3
	Railing	269.4	0.0	0.0	0.0	0.0	0.0	269.4
	Total	611.5	559.2	674.5	671.2	575.8	401.9	3494.1
1R(R)	Girder	475.8	593.9	648.0	604.4	465.8	258.3	3046.2
	Slab	24.5	55.0	56.5	48.7	36.7	11.6	233.0
	Railing	0.0	0.0	0.0	0.0	0.0	211.2	211.2
	Total	500.3	648.9	704.5	653.1	502.5	481.1	3490.4
2R	Girder	325.3	506.1	605.9	589.8	466.3	267.4	2761.0
	Slab	22.5	54.0	57.3	47.5	37.7	12.2	231.2
	Railing	277.9	0.0	0.0	0.0	0.0	222.1	500.0
	Total	625.7	560.1	663.3	637.3	504.0	501.7	3492.2
1SR(L)	Girder	331.4	480.3	593.6	606.5	525.3	380.7	2917.8
	Slab	19.8	48.8	54.4	72.8	38.8	16.0	250.5
	Railing	297.2	0.0	0.0	0.0	0.0	0.0	297.2
	Total	648.4	529.1	648.0	679.3	564.0	396.7	3465.5
1SR(R)	Girder	470.0	581.8	629.6	579.9	442.5	276.4	2980.1
	Slab	46.2	54.2	54.9	46.4	31.4	55.1	288.2
	Railing	0.0	0.0	0.0	0.0	0.0	244.1	244.1
	Total	516.2	636.0	684.5	626.3	473.9	575.6	3512.4
2SR	Girder	332.1	470.8	566.3	550.8	434.2	279.3	2633.5
	Slab	40.7	49.2	53.8	71.2	32.2	56.2	303.3
	Railing	300.0	0.0	0.0	0.0	0.0	249.2	549.2
	Total	672.8	520.0	620.1	622.0	466.4	584.7	3486

Table A.6. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 3 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Total (7)**
No SR	Girder	633.6	865.6	907.2	756.1	3162.5
	Slab	60.8	107.3	99.5	64.3	331.9
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	694.4	973.0	1006.7	820.4	3494.4
1S(L)	Girder	610.8	827.0	889.7	750.8	3078.3
	Slab	152.0	103.7	96.5	64.0	416.1
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	762.7	930.7	986.2	814.8	3494.4
1S(R)	Girder	631.6	849.3	865.9	713.2	3060.0
	Slab	60.7	106.2	95.7	171.9	434.5
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	692.2	955.4	961.6	885.2	3494.4
2S	Girder	607.4	811.9	849.7	707.2	2976.1
	Slab	151.7	102.7	93.0	171.0	518.4
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	759.1	914.6	942.7	878.1	3494.4
1R(L)	Girder	430.9	778.0	885.7	773.7	2868.3
	Slab	51.7	106.1	98.6	65.7	322.1
	Railing	304.1	0.0	0.0	0.0	304.1
	Total	786.7	884.1	984.2	839.5	3494.4
1R(R)	Girder	655.9	840.0	800.7	502.1	2798.7
	Slab	62.6	108.4	97.3	52.3	320.5
	Railing	0.0	0.0	0.0	375.2	375.2
	Total	718.5	948.5	898.0	929.5	3494.5
2R	Girder	440.1	747.1	774.5	509.4	2471.1
	Slab	53.1	107.3	96.4	53.4	310.3
	Railing	322.7	0.0	0.0	390.4	713.1
	Total	815.9	854.4	870.9	953.2	3494.4
1SR(L)	Girder	427.1	732.8	860.6	761.7	2782.2
	Slab	108.8	100.5	94.3	65.0	368.6
	Railing	343.7	0.0	0.0	0.0	343.7
	Total	879.6	833.3	954.9	826.7	3494.5
1SR(R)	Girder	644.7	815.0	754.4	493.6	2707.7
	Slab	61.8	105.8	91.2	119.2	378.0
	Railing	0.0	0.0	0.0	408.7	408.7
	Total	706.5	920.7	845.6	1021.6	3494.4
2SR	Girder	426.3	683.5	709.9	491.0	2310.7
	Slab	110.9	99.9	87.0	120.7	418.5
	Railing	351.5	0.0	0.0	413.7	765.2
	Total	888.7	783.4	796.9	1025.4	3494.4

Table A.7. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 4 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Girder 5 (7)**	**Girder 6 (8)**	**Girder 7 (9)**	**Girder 8 (10)**	**Girder 9 (11)**	**Total (12)**
No SR	Girder	543.1	582.3	609.2	609.5	578.1	530.5	436.9	314.4	199.3	4403.0
	Slab	33.0	36.2	31.7	39.1	33.6	35.8	27.8	12.5	6.4	256.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	576.1	618.5	640.8	648.5	611.7	566.3	464.7	326.9	205.7	4659.2
1S(L)	Girder	517.7	557.4	592.8	600.1	573.2	528.5	436.7	315.5	201.3	4323.3
	Slab	114.8	35.4	31.4	39.1	32.2	36.0	27.9	12.6	6.5	336.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	632.6	592.8	624.2	639.2	605.5	564.5	464.7	328.1	207.8	4659.2
1S(R)	Girder	543.0	580.5	605.4	603.7	570.3	520.8	425.4	308.2	220.0	4377.3
	Slab	33.0	36.2	31.5	38.7	33.3	34.9	26.2	9.8	38.3	282.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	576.0	616.6	637.0	642.4	603.5	555.7	451.6	318.0	258.3	4659.2
2S	Girder	517.1	555.6	589.2	594.4	565.6	519.0	425.4	309.2	221.6	4297.1
	Slab	114.8	35.3	31.3	38.8	31.9	35.1	26.3	10.0	38.7	362.1
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	632.0	590.9	620.5	633.2	597.5	554.1	451.7	319.1	260.3	4659.2
1R(L)	Girder	384.6	488.5	559.8	587.4	572.5	533.7	444.0	322.9	208.5	4101.8
	Slab	28.0	36.7	33.0	40.6	33.4	36.9	28.5	13.0	6.7	256.9
	Railing	300.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	300.5
	Total	713.1	525.2	592.8	628.0	605.9	570.6	472.5	335.9	215.2	4659.2
1R(R)	Girder	545.6	584.5	610.7	609.7	575.5	522.9	421.1	287.4	160.6	4318.2
	Slab	33.2	36.4	31.9	39.4	33.9	36.1	27.8	11.9	4.6	255.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	85.8	85.8
	Total	578.8	620.9	642.7	649.1	609.5	559.1	448.9	299.3	251	4659.3
2R	Girder	386.4	490.3	561.2	587.6	569.8	525.8	427.3	294.3	166.9	4009.6
	Slab	28.2	36.9	33.3	41.0	33.7	37.2	28.6	12.3	4.8	256.0
	Railing	302.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	91.4	393.6
	Total	716.8	527.2	594.6	628.6	603.5	563.0	455.9	306.6	263.1	4659.2
1SR(L)	Girder	384.6	467.8	539.5	572.8	562.5	527.4	440.7	322.1	209.8	4027.1
	Slab	84.4	34.5	31.9	40.1	31.3	36.8	28.6	13.0	6.8	307.4
	Railing	324.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	324.8
	Total	793.8	502.3	571.4	612.8	593.7	564.2	469.3	335.2	216.6	4659.3
1SR(R)	Girder	546.1	582.9	606.8	603.0	565.9	510.1	404.8	274.5	172.0	4266.1
	Slab	33.2	36.4	31.8	39.1	33.5	35.1	26.0	8.8	27.3	271.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	121.9	121.9
	Total	579.4	619.3	638.6	642.1	599.4	545.2	430.8	283.3	321.2	4659.2
2SR	Girder	386.1	467.8	537.0	566.3	550.3	506.5	407.5	280.0	178.3	3879.9
	Slab	84.9	34.7	32.1	40.1	31.2	36.2	26.8	9.3	28.7	324.0
	Railing	326.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	128.5	455.3
	Total	797.9	502.5	569.1	606.5	581.5	542.7	434.3	289.3	335.5	4659.3

Table A.8. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 4 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Girder 5 (7)**	**Girder 6 (8)**	**Girder 7 (9)**	**Total (10)**
No SR	Girder	680.7	753.3	767.5	735.4	645.9	471.1	282.1	4336.1
	Slab	42.3	59.0	64.2	56.5	52.8	34.8	13.6	323.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	723.0	812.3	831.7	791.9	698.7	505.9	295.7	4659.2
1S(L)	Girder	644.4	719.7	748.7	726.6	642.8	471.5	284.9	4238.6
	Slab	143.3	57.2	63.8	54.6	53.0	35.0	13.8	420.7
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	787.7	777.0	812.6	781.1	695.7	506.4	298.7	4659.3
1S(R)	Girder	680.1	749.6	760.1	724.4	630.3	454.5	300.7	4299.7
	Slab	42.3	58.8	63.8	55.8	51.1	31.3	56.5	359.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	722.4	808.4	823.9	780.2	681.4	485.8	357.2	4659.2
2S	Girder	643.2	716.1	741.7	715.8	627.3	454.8	302.8	4201.7
	Slab	143.1	57.1	63.4	54.0	51.3	31.5	57.1	457.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	786.3	773.3	805.1	769.8	678.6	486.3	359.9	4659.3
1R(L)	Girder	467.6	642.4	718.7	721.0	648.9	481.6	296.4	3976.6
	Slab	35.0	58.9	66.0	56.3	54.5	35.9	14.4	320.9
	Railing	361.8	0.0	0.0	0.0	0.0	0.0	0.0	361.8
	Total	864.4	701.3	784.7	777.3	703.3	517.5	310.8	4659.3
1R(R)	Girder	685.4	756.4	767.6	729.2	627.2	431.8	217.0	4214.7
	Slab	42.6	59.4	64.8	57.0	53.1	34.0	10.2	321.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	123.5	123.5
	Total	728.1	815.8	832.4	786.2	680.3	465.7	350.7	4659.2
2R	Girder	470.7	644.8	718.5	714.4	629.0	439.7	225.9	3843.0
	Slab	35.2	59.4	66.6	56.8	54.8	35.1	10.8	318.8
	Railing	365.0	0.0	0.0	0.0	0.0	0.0	132.4	497.5
	Total	870.9	704.2	785.1	771.2	683.9	474.7	369.2	4659.2
1SR(L)	Girder	464.8	609.9	693.5	705.4	640.0	478.2	297.3	3889.1
	Slab	102.2	55.2	64.5	53.3	54.2	35.9	14.5	379.7
	Railing	390.4	0.0	0.0	0.0	0.0	0.0	0.0	390.4
	Total	957.4	665.1	758.0	758.7	694.2	514.1	311.8	4659.2
1SR(R)	Girder	685.3	752.3	758.7	714.9	606.1	407.5	227.0	4151.8
	Slab	42.6	59.2	64.3	56.2	51.1	29.8	39.1	342.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	165.1	165.1
	Total	728.0	811.5	823.0	771.1	657.2	437.3	431.2	4659.2
2SR	Girder	466.7	608.3	684.7	684.7	599.1	411.6	235.2	3690.4
	Slab	103.0	55.6	64.6	53.2	52.6	30.9	41.2	401.1
	Railing	393.5	0.0	0.0	0.0	0.0	0.0	174.3	567.8
	Total	963.2	663.8	749.4	737.9	651.7	442.5	450.7	4659.3

Table A.9. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (One-Span, 4 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 5 (7)**	**Girder 4 (6)**	**Total (8)**
No SR	Girder	914.4	1047.5	1013.2	793.6	441.7	4210.3
	Slab	79.8	123.0	119.2	96.3	30.7	448.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	994.1	1170.4	1132.4	889.9	472.4	4659.3
1S(L)	Girder	856.9	1000.4	994.4	788.5	444.9	4085.0
	Slab	211.0	119.8	116.1	96.4	30.9	574.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	1067.9	1120.2	1110.4	884.9	475.8	4659.3
1S(R)	Girder	912.3	1037.9	994.6	760.0	450.3	4155.1
	Slab	79.6	122.2	117.5	90.9	93.9	504.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	991.9	1160.1	1112.1	850.9	544.2	4659.3
2S	Girder	854.1	991.4	976.3	755.3	452.2	4029.2
	Slab	210.6	119.2	114.5	91.2	94.6	630.1
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	1064.7	1110.6	1090.8	846.4	546.8	4659.3
1R(L)	Girder	605.5	919.7	975.9	797.0	463.1	3761.2
	Slab	66.9	122.1	118.6	98.8	32.2	438.6
	Railing	459.5	0.0	0.0	0.0	0.0	459.5
	Total	1131.9	1041.7	1094.6	895.8	495.3	4659.3
1R(R)	Girder	922.9	1047.8	995.5	736.6	320.9	4023.7
	Slab	80.4	124.0	119.9	94.7	23.9	442.9
	Railing	0.0	0.0	0.0	0.0	192.7	192.7
	Total	1003.4	1171.8	1115.4	831.2	537.5	4659.3
2R	Girder	610.4	918.6	956.8	736.4	332.7	3555.0
	Slab	67.5	123.1	119.4	97.2	25.1	432.4
	Railing	465.5	0.0	0.0	0.0	206.4	671.9
	Total	1143.4	1041.7	1076.2	833.6	564.3	4659.3
1SR(L)	Girder	593.8	867.5	948.1	784.3	461.9	3655.5
	Slab	149.4	116.0	113.6	98.1	32.2	509.3
	Railing	494.4	0.0	0.0	0.0	0.0	494.4
	Total	1237.6	983.5	1061.7	882.4	494.1	4659.3
1SR(R)	Girder	920.6	1035.5	971.1	693.7	324.5	3945.3
	Slab	80.3	123.0	117.8	88.0	63.4	472.5
	Railing	0.0	0.0	0.0	0.0	241.4	241.4
	Total	1000.8	1158.5	1088.9	781.7	629.3	4659.2
2SR	Girder	595.5	855.5	905.7	681.9	333.1	3371.6
	Slab	150.5	116.3	112.5	90.1	66.5	536.0
	Railing	498.5	0.0	0.0	0.0	253.1	751.7
	Total	1244.5	971.8	1018.2	772.0	652.7	4659.2

Table A.10. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 2 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Total (8)
No SR	Girder	411.5	410.4	388.7	326.8	236.5	1773.9
	Slab	23.6	26.8	25.0	18.0	10.9	104.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	435.1	437.2	413.7	344.8	247.3	1878.2
1S(L)	Girder	379.0	389.0	379.0	325.3	241.5	1713.7
	Slab	84.0	26.1	25.0	18.2	11.2	164.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	463.0	415.1	404.0	343.5	252.6	1878.2
1S(R)	Girder	404.1	401.9	378.5	316.3	240.1	1740.8
	Slab	23.3	26.3	24.1	16.3	47.1	137.1
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	427.4	428.2	402.6	332.6	287.2	1877.9
2S	Girder	372.6	381.4	369.3	314.5	242.8	1680.6
	Slab	82.7	25.7	24.2	16.6	48.1	197.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	455.3	407.1	393.5	331.1	291.0	1878.0
1R(L)	Girder	277.9	338.8	358.3	325.0	257.6	1557.7
	Slab	19.2	26.5	25.8	19.0	12.2	102.7
	Railing	218.6	0.0	0.0	0.0	0.0	218.6
	Total	515.8	365.4	384.1	344.0	269.7	1879.0
1R(R)	Girder	420.6	408.2	372.3	290.8	174.5	1666.5
	Slab	24.2	27.2	25.2	17.6	8.3	102.6
	Railing	0.0	0.0	0.0	0.0	109.5	109.5
	Total	444.8	435.4	397.6	308.4	292.4	1878.6
2R	Girder	282.7	333.5	338.1	282.8	184.1	1421.1
	Slab	19.8	27.1	26.2	18.7	9.4	101.1
	Railing	228.7	0.0	0.0	0.0	128.5	357.2
	Total	531.1	360.5	364.2	301.5	322.0	1879.5
1SR(L)	Girder	272.9	322.4	345.5	316.4	252.0	1509.2
	Slab	59.3	24.6	24.9	18.6	11.9	139.4
	Railing	230.5	0.0	0.0	0.0	0.0	230.5
	Total	562.6	347.0	370.5	335.0	263.9	1879.0
1SR(R)	Girder	409.6	396.0	358.2	277.1	179.5	1620.3
	Slab	23.7	26.5	24.0	15.3	32.8	122.3
	Railing	0.0	0.0	0.0	0.0	135.9	135.9
	Total	433.3	422.5	382.2	292.4	348.2	1878.6
2SR	Girder	269.9	309.2	315.1	263.4	182.5	1340.1
	Slab	59.6	24.9	24.5	16.5	35.2	160.7
	Railing	232.2	0.0	0.0	0.0	146.5	378.7
	Total	561.8	334.1	339.6	279.9	364.1	1879.5

Table A.11. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 2 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Total (7)
No SR	Girder	498.1	504.8	432.8	308.9	1744.5
	Slab	33.8	42.2	40.1	17.9	134.1
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	531.8	547.0	472.9	326.9	1878.6
1S(L)	Girder	456.4	479.9	424.8	312.8	1673.8
	Slab	105.3	41.1	40.2	18.3	204.8
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	561.7	521.0	464.9	331.0	1878.6
1S(R)	Girder	488.1	491.7	415.3	307.4	1702.5
	Slab	33.2	41.2	37.7	63.7	175.8
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	521.4	532.9	453.0	371.1	1878.3
2S	Girder	447.7	468.0	407.6	308.7	1632.1
	Slab	103.6	40.3	38.0	64.5	246.3
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	551.3	508.3	445.6	373.2	1878.4
1R(L)	Girder	327.2	427.0	412.6	329.9	1496.8
	Slab	27.4	41.6	41.2	19.5	129.7
	Railing	253.0	0.0	0.0	0.0	253.0
	Total	607.7	468.6	453.7	349.5	1879.4
1R(R)	Girder	507.8	492.0	388.9	220.3	1609.0
	Slab	34.5	42.6	39.5	14.1	130.7
	Railing	0.0	0.0	0.0	139.5	139.5
	Total	542.3	534.6	428.3	373.8	1879.1
2R	Girder	330.7	409.6	361.9	227.4	1329.7
	Slab	28.1	42.1	40.6	15.4	126.2
	Railing	264.5	0.0	0.0	159.6	424.1
	Total	623.3	451.7	402.5	402.4	1880.0
1SR(L)	Girder	319.3	404.2	397.3	320.3	1441.1
	Slab	73.1	38.8	40.1	19.0	171.0
	Railing	267.4	0.0	0.0	0.0	267.4
	Total	659.7	443.0	437.4	339.3	1879.5
1SR(R)	Girder	492.0	473.5	367.1	223.0	1555.5
	Slab	33.7	41.1	36.2	43.6	154.5
	Railing	0.0	0.0	0.0	169.1	169.1
	Total	525.6	514.5	403.2	435.7	1879.1
2SR	Girder	312.6	375.8	331.3	221.0	1240.7
	Slab	73.1	38.7	37.1	45.7	194.6
	Railing	267.4	0.0	0.0	177.4	444.8
	Total	653.1	414.4	368.4	444.2	1880.1

Table A.12. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 2 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Total (6)
No SR	Girder	636.2	632.4	431.9	1700.4
	Slab	56.3	83.7	38.9	179.0
	Railing	0.0	0.0	0.0	0.0
	Total	692.5	716.1	470.8	1879.4
1S(L)	Girder	579.4	603.9	430.8	1614.0
	Slab	144.7	81.7	38.9	265.4
	Railing	0.0	0.0	0.0	0.0
	Total	724.1	685.6	469.7	1879.4
1S(R)	Girder	620.9	605.0	418.4	1644.2
	Slab	55.3	80.1	99.6	235.0
	Railing	0.0	0.0	0.0	0.0
	Total	676.1	685.1	518.0	1879.2
2S	Girder	566.2	578.3	414.7	1559.2
	Slab	141.9	78.5	99.6	320.0
	Railing	0.0	0.0	0.0	0.0
	Total	708.0	656.9	514.3	1879.2
1R(L)	Girder	405.7	556.2	445.3	1407.2
	Slab	46.1	82.2	40.4	168.8
	Railing	304.3	0.0	0.0	304.3
	Total	756.2	638.5	485.7	1880.3
1R(R)	Girder	642.1	583.0	295.0	1520.1
	Slab	57.1	82.4	32.1	171.5
	Railing	0.0	0.0	188.5	188.5
	Total	699.2	665.4	515.6	1880.1
2R	Girder	403.7	500.6	293.6	1197.9
	Slab	46.8	81.0	33.3	161.1
	Railing	315.4	0.0	206.6	522.0
	Total	765.9	581.6	533.5	1881.0
1SR(L)	Girder	390.7	523.6	427.3	1341.5
	Slab	99.3	77.9	39.1	216.3
	Railing	322.7	0.0	0.0	322.7
	Total	812.7	601.4	466.3	1880.5
1SR(R)	Girder	617.2	548.7	290.9	1456.7
	Slab	55.3	77.3	68.7	201.3
	Railing	0.0	0.0	222.2	222.2
	Total	672.5	626.0	581.8	1880.2
2SR	Girder	375.2	448.3	276.9	1100.4
	Slab	98.0	73.7	69.2	240.9
	Railing	317.1	0.0	222.7	539.8
	Total	790.4	522.0	568.8	1881.1

Table A.13. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 3 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Girder 6 (8)	Girder 7 (9)	Total (10)
No SR	Girder	428.0	455.1	466.1	443.8	382.5	292.0	187.5	2655.0
	Slab	24.0	28.2	28.6	28.5	27.4	17.9	8.0	162.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	452.0	483.3	494.7	472.3	409.9	309.9	195.5	2817.6
1S(L)	Girder	403.6	434.0	454.0	437.8	380.5	292.6	190.1	2592.6
	Slab	87.4	27.0	28.3	28.5	27.5	18.0	8.1	225.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	491.0	461.0	482.3	466.4	408.0	310.6	198.2	2817.6
1S(R)	Girder	425.6	450.7	459.7	435.8	373.3	284.7	200.2	2630.0
	Slab	24.0	28.1	28.2	27.8	26.2	15.9	37.1	187.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	449.6	478.8	487.9	463.6	399.5	300.6	237.3	2817.3
2S	Girder	400.9	429.8	447.9	430.1	371.4	285.2	202.1	2567.4
	Slab	87.0	26.9	28.0	27.9	26.4	16.0	37.6	249.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	487.9	456.7	475.9	458.0	397.8	301.2	239.7	2817.3
1R(L)	Girder	298.4	383.8	431.9	431.8	383.5	300.5	201.2	2431.1
	Slab	19.7	28.1	29.6	29.8	28.4	18.7	8.7	163.1
	Railing	224.6	0.0	0.0	0.0	0.0	0.0	0.0	224.6
	Total	542.6	411.9	461.5	461.5	412.0	319.2	209.9	2818.7
1R(R)	Girder	432.6	457.5	465.5	438.4	369.1	266.4	146.7	2576.1
	Slab	24.3	28.5	28.9	28.8	27.6	17.6	6.1	161.7
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	80.1	80.1
	Total	456.9	486.0	494.4	467.2	396.7	284.0	232.9	2818.0
2R	Girder	301.4	385.4	430.8	425.6	368.6	272.1	155.1	2339.0
	Slab	20.0	28.4	30.0	30.2	28.7	18.4	6.6	162.3
	Railing	228.3	0.0	0.0	0.0	0.0	0.0	89.6	317.9
	Total	549.7	413.8	460.8	455.7	397.3	290.4	251.3	2819.1
1SR(L)	Girder	295.7	365.4	416.9	421.5	376.9	297.1	200.6	2374.1
	Slab	62.3	25.7	28.5	29.2	28.2	18.6	8.7	201.3
	Railing	243.4	0.0	0.0	0.0	0.0	0.0	0.0	243.4
	Total	601.4	391.1	445.4	450.7	405.1	315.7	209.3	2818.7
1SR(R)	Girder	430.2	452.5	457.6	427.8	356.0	254.0	153.9	2532.1
	Slab	24.2	28.3	28.5	28.1	26.2	15.1	26.1	176.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	109.3	109.3
	Total	454.5	480.8	486.1	455.9	382.3	269.2	289.4	2818.0
2SR	Girder	296.1	362.6	408.7	405.6	349.5	256.2	160.3	2239.0
	Slab	62.8	25.9	28.6	29.0	27.2	16.0	28.0	217.5
	Railing	245.3	0.0	0.0	0.0	0.0	0.0	117.4	362.7
	Total	604.3	388.5	437.3	434.6	376.7	272.2	305.7	2819.2

Table A.14. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 3 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Girder 5 (7)**	**Girder 6 (8)**	**Total (9)**
No SR	Girder	344.9	480.3	554.0	535.0	431.0	267.6	2612.8
	Slab	16.7	44.1	49.4	52.2	32.6	11.8	206.7
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	361.5	524.4	603.4	587.3	463.5	279.4	2819.5
1S(L)	Girder	349.9	458.8	537.5	524.2	425.3	266.5	2562.3
	Slab	73.0	40.3	47.8	51.6	32.3	11.7	256.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	422.9	499.1	585.3	575.8	457.6	278.2	2819.1
1S(R)	Girder	342.4	473.2	542.1	518.8	413.9	283.4	2573.9
	Slab	16.6	43.7	48.5	50.4	28.7	57.2	245.1
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	359.0	517.0	590.6	569.2	442.6	340.6	2819.0
2S	Girder	346.6	452.2	526.3	508.7	408.6	281.3	2523.7
	Slab	72.4	40.1	47.1	49.8	28.6	56.8	294.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	419.0	492.2	573.4	558.5	437.2	338.2	2818.5
1R(L)	Girder	237.0	420.8	528.6	528.6	434.5	277.3	2426.7
	Slab	13.0	42.9	50.0	53.1	33.3	12.2	204.4
	Railing	189.2	0.0	0.0	0.0	0.0	0.0	189.2
	Total	439.2	463.8	578.5	581.7	467.7	289.4	2820.4
1R(R)	Girder	352.0	482.6	548.5	515.1	385.8	188.0	2472.0
	Slab	17.0	44.6	50.0	52.6	31.6	8.9	204.6
	Railing	0.0	0.0	0.0	0.0	0.0	143.5	143.5
	Total	369.0	527.2	598.5	567.7	417.3	340.4	2820.1
2R	Girder	241.3	421.7	522.3	507.7	387.3	193.8	2274.2
	Slab	13.2	43.5	50.6	53.5	32.3	9.2	202.3
	Railing	194.3	0.0	0.0	0.0	0.0	150.1	344.5
	Total	448.9	465.2	572.9	561.2	419.6	353.2	2821.0
1SR(L)	Girder	249.4	397.0	507.9	513.9	425.9	274.5	2368.8
	Slab	52.2	38.1	47.8	52.1	32.9	12.1	235.1
	Railing	216.0	0.0	0.0	0.0	0.0	0.0	216.0
	Total	517.6	435.1	555.7	566.0	458.8	286.6	2819.9
1SR(R)	Girder	348.8	473.6	533.5	494.8	364.4	203.6	2418.8
	Slab	16.8	44.2	48.9	50.3	26.9	40.6	227.7
	Railing	0.0	0.0	0.0	0.0	0.0	173.1	173.1
	Total	365.6	517.7	582.4	545.1	391.3	417.4	2819.6
2SR	Girder	249.9	390.3	488.6	474.6	358.8	205.6	2167.8
	Slab	52.6	38.4	47.6	50.5	27.5	41.3	257.9
	Railing	217.8	0.0	0.0	0.0	0.0	176.5	394.3
	Total	520.2	428.8	536.2	525.1	386.3	423.5	2820.0

Table A.15. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 3 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Total (7)
No SR	Girder	478.9	718.9	753.8	582.2	2533.8
	Slab	42.1	93.6	97.0	53.7	286.4
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	521.1	812.4	850.8	635.9	2820.2
1S(L)	Girder	466.3	689.0	739.3	577.3	2471.9
	Slab	109.5	89.4	95.7	53.4	348.1
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	575.9	778.4	835.0	630.6	2820.0
1S(R)	Girder	476.8	705.6	721.2	553.0	2456.7
	Slab	42.0	92.7	93.3	135.3	363.4
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	518.9	798.3	814.5	688.4	2820.0
2S	Girder	463.3	676.7	707.7	547.6	2395.2
	Slab	109.2	88.8	92.2	134.4	424.7
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	572.5	765.5	799.9	682.0	2819.8
1R(L)	Girder	331.5	660.9	742.4	594.9	2329.7
	Slab	34.9	92.5	98.1	54.7	280.3
	Railing	211.2	0.0	0.0	0.0	211.2
	Total	577.6	753.4	840.5	649.6	2821.2
1R(R)	Girder	495.6	705.4	681.7	392.0	2274.7
	Slab	43.5	95.1	96.0	44.9	279.5
	Railing	0.0	0.0	0.0	267.3	267.3
	Total	539.1	800.5	777.7	704.1	2821.4
2R	Girder	339.3	643.9	667.3	398.0	2048.5
	Slab	36.0	94.2	97.2	45.7	273.1
	Railing	223.6	0.0	0.0	277.2	500.8
	Total	599.0	738.1	764.5	720.9	2822.5
1SR(L)	Girder	327.2	624.4	721.9	585.7	2259.1
	Slab	76.0	86.8	96.1	54.1	313.0
	Railing	249.3	0.0	0.0	0.0	249.3
	Total	652.5	711.2	818.0	639.8	2821.4
1SR(R)	Girder	487.2	685.1	643.5	383.3	2199.1
	Slab	43.0	93.2	90.2	94.5	320.9
	Railing	0.0	0.0	0.0	301.7	301.7
	Total	530.2	778.3	733.7	779.5	2821.7
2SR	Girder	327.3	591.4	613.0	381.4	1913.0
	Slab	77.6	87.2	90.1	95.5	350.4
	Railing	254.8	0.0	0.0	304.7	559.5
	Total	659.6	678.6	703.1	781.7	2823.0

Table A.16. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 4 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Girder 5 (7)**	**Girder 6 (8)**	**Girder 7 (9)**	**Girder 8 (10)**	**Girder 9 (11)**	**Total (12)**
No SR	Girder	419.3	464.1	497.0	503.8	479.7	440.7	356.2	242.3	135.8	3538.9
	Slab	23.4	27.9	28.7	29.9	31.8	31.9	25.3	13.1	5.3	217.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	442.7	492.0	525.7	533.7	511.5	472.6	381.5	255.4	141.1	3756.2
1S(L)	Girder	401.1	444.2	484.6	496.7	476.1	439.3	356.1	243.0	137.0	3478.2
	Slab	85.7	26.4	28.3	29.9	31.8	32.0	25.3	13.2	5.4	278.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	486.8	470.6	512.8	526.6	507.9	471.3	381.5	256.2	142.4	3756.2
1S(R)	Girder	419.4	462.8	494.0	498.9	473.0	432.8	348.1	238.0	155.0	3522.1
	Slab	23.4	27.9	28.6	29.7	31.3	31.1	24.0	11.0	26.7	233.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	442.9	490.7	522.7	528.6	504.3	464.0	372.1	249.0	181.7	3755.9
2S	Girder	400.8	442.9	481.7	492.0	469.5	431.5	348.1	238.7	156.0	3461.1
	Slab	85.7	26.4	28.2	29.7	31.4	31.3	24.1	11.1	26.9	294.7
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	486.5	469.3	509.9	521.6	500.9	462.8	372.1	249.8	183.0	3755.9
1R(L)	Girder	297.1	396.5	464.4	491.7	478.8	444.7	361.7	247.7	140.8	3323.3
	Slab	19.3	27.8	29.8	31.2	32.9	32.7	25.8	13.4	5.5	218.5
	Railing	215.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	215.6
	Total	531.9	424.3	494.2	522.9	511.7	477.4	387.5	261.2	146.3	3757.3
1R(R)	Girder	420.6	465.2	497.9	503.9	478.4	436.7	347.5	226.7	113.8	3490.7
	Slab	23.5	28.0	28.8	30.1	32.0	32.1	25.3	12.8	4.0	216.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	49.2	49.2
	Total	444.1	493.2	526.7	534.0	510.4	468.8	372.8	239.5	167.0	3756.5
2R	Girder	298.0	397.5	465.2	491.8	477.4	440.5	352.5	231.3	117.3	3271.5
	Slab	19.3	27.9	29.9	31.4	33.1	32.9	25.8	13.1	4.2	217.6
	Railing	216.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	52.0	268.5
	Total	533.7	425.4	495.1	523.2	510.5	473.4	378.3	244.4	173.5	3757.6
1SR(L)	Girder	296.5	377.7	448.6	480.4	471.1	440.0	359.4	247.3	141.8	3262.8
	Slab	61.4	25.1	28.5	30.6	32.6	32.7	25.8	13.5	5.6	255.8
	Railing	238.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	238.9
	Total	596.7	402.8	477.1	511.1	503.7	472.7	385.2	260.8	147.4	3757.4
1SR(R)	Girder	421.1	464.2	494.9	498.7	470.7	426.7	335.6	216.2	122.7	3450.8
	Slab	23.5	28.0	28.8	29.9	31.6	31.3	23.9	10.4	19.0	226.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	79.2	79.2
	Total	444.6	492.2	523.7	528.7	502.3	458.1	359.5	226.6	220.8	3756.5
2SR	Girder	297.2	377.4	446.4	475.5	462.2	425.9	338.2	219.8	126.4	3169.0
	Slab	61.6	25.2	28.6	30.7	32.5	32.1	24.5	10.8	19.8	265.8
	Railing	240.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	82.8	322.9
	Total	598.9	402.6	475.1	506.2	494.6	458.0	362.7	230.6	229.1	3757.7

Table A.17. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 4 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Girder 6 (8)	Girder 7 (9)	Total (10)
No SR	Girder	524.7	608.0	633.2	612.5	535.6	373.0	195.5	3482.5
	Slab	34.5	46.4	54.4	54.6	45.8	29.4	9.4	274.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	559.3	654.4	687.6	667.0	581.4	402.4	205.0	3757.1
1S(L)	Girder	498.7	581.4	619.1	605.8	533.2	373.2	197.3	3408.7
	Slab	110.7	44.2	53.8	54.5	45.9	29.5	9.6	348.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	609.4	625.6	672.9	660.3	579.2	402.7	206.9	3757.0
1S(R)	Girder	524.4	604.9	626.9	602.9	523.8	361.4	214.1	3458.3
	Slab	34.5	46.3	54.1	53.8	44.3	26.4	39.0	298.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	558.9	651.2	680.9	656.7	568.1	387.8	253.1	3756.7
2S	Girder	497.8	578.4	612.9	596.4	521.6	361.5	215.4	3384.2
	Slab	110.7	44.1	53.6	53.8	44.4	26.5	39.4	372.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	608.5	622.6	666.5	650.2	566.0	388.1	254.8	3756.7
1R(L)	Girder	362.0	529.9	603.0	606.6	540.0	380.5	203.8	3225.7
	Slab	28.4	46.1	56.0	56.2	47.0	30.2	9.9	273.8
	Railing	258.9	0.0	0.0	0.0	0.0	0.0	0.0	258.9
	Total	649.2	576.0	658.9	662.9	587.0	410.6	213.7	3758.4
1R(R)	Girder	527.2	609.7	633.5	609.3	525.2	349.5	155.6	3410.1
	Slab	34.7	46.6	54.7	54.9	46.0	28.9	7.2	273.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	74.4	74.4
	Total	561.9	656.4	688.2	664.2	571.2	378.4	237.2	3757.5
2R	Girder	363.6	531.3	603.1	603.3	529.0	355.5	161.0	3146.9
	Slab	28.5	46.4	56.3	56.6	47.3	29.6	7.6	272.2
	Railing	260.6	0.0	0.0	0.0	0.0	0.0	79.2	339.7
	Total	652.7	577.7	659.4	659.9	576.2	385.2	247.7	3758.8
1SR(L)	Girder	358.4	502.0	583.6	594.4	533.3	378.1	204.7	3154.6
	Slab	77.8	42.2	54.4	55.6	46.9	30.2	10.0	317.0
	Railing	287.0	0.0	0.0	0.0	0.0	0.0	0.0	287.0
	Total	723.3	544.2	638.0	650.0	580.2	408.3	214.6	3758.6
1SR(R)	Girder	527.3	606.6	626.4	597.7	509.3	330.7	163.4	3361.5
	Slab	34.7	46.6	54.4	54.1	44.3	25.5	26.8	286.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	109.5	109.5
	Total	562.0	653.2	680.8	651.9	553.7	356.2	299.8	3757.5
2SR	Girder	359.3	500.3	576.9	579.8	506.5	334.0	168.4	3025.3
	Slab	78.2	42.4	54.5	55.3	45.5	26.2	28.1	330.2
	Railing	288.7	0.0	0.0	0.0	0.0	0.0	114.9	403.5
	Total	726.2	542.7	631.4	635.1	552.0	360.3	311.4	3759.1

Table A.18. Calculated Positive Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Positive, 4 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 5 (7)	Girder 4 (6)	Total (8)
No SR	Girder	704.8	861.1	848.0	648.2	313.2	3375.3
	Slab	60.6	103.9	112.8	83.6	22.7	383.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	765.3	965.0	960.7	731.8	335.9	3758.8
1S(L)	Girder	663.5	824.4	833.5	644.2	315.2	3280.8
	Slab	159.1	100.1	112.2	83.6	22.9	478.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	822.6	924.5	945.7	727.8	338.2	3758.8
1S(R)	Girder	703.0	852.6	832.2	624.0	326.1	3337.9
	Slab	60.5	103.3	111.0	78.8	67.0	420.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	763.5	955.9	943.2	702.8	393.1	3758.5
2S	Girder	661.1	816.3	818.2	620.2	327.3	3243.1
	Slab	158.8	99.6	110.5	79.0	67.5	515.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	820.0	915.9	928.7	699.2	394.8	3758.5
1R(L)	Girder	471.4	774.4	828.3	653.6	326.6	3054.3
	Slab	50.1	103.1	115.1	85.5	23.8	377.5
	Railing	328.7	0.0	0.0	0.0	0.0	328.7
	Total	850.1	877.5	943.4	739.0	350.3	3760.4
1R(R)	Girder	709.6	861.9	838.5	613.5	234.6	3258.2
	Slab	60.9	104.5	113.4	82.7	18.2	379.6
	Railing	0.0	0.0	0.0	0.0	121.7	121.7
	Total	770.5	966.4	951.9	696.2	374.5	3759.5
2R	Girder	474.3	774.5	818.2	616.8	242.4	2926.2
	Slab	50.4	103.8	115.8	84.6	19.0	373.5
	Railing	331.9	0.0	0.0	0.0	129.6	461.5
	Total	856.6	878.2	934.0	701.4	391.0	3761.2
1SR(L)	Girder	459.6	731.9	806.1	643.7	326.1	2967.4
	Slab	110.1	96.8	113.2	85.0	23.8	428.9
	Railing	364.6	0.0	0.0	0.0	0.0	364.6
	Total	934.3	828.7	919.3	728.7	349.9	3760.8
1SR(R)	Girder	708.0	851.8	818.6	581.0	237.2	3196.5
	Slab	60.9	103.9	111.3	77.0	45.4	398.5
	Railing	0.0	0.0	0.0	0.0	164.7	164.7
	Total	768.8	955.6	929.9	658.1	447.3	3759.7
2SR	Girder	460.1	722.7	777.0	574.9	242.9	2777.6
	Slab	110.7	97.0	112.0	78.7	47.4	445.7
	Railing	366.6	0.0	0.0	0.0	171.9	538.5
	Total	937.5	819.7	889.0	653.5	462.1	3761.8

Table A.19. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 2 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Total (8)
No SR	Girder	448.3	494.5	469.7	377.7	215.2	2005.3
	Slab	26.8	28.1	26.7	21.5	11.6	114.7
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	475.1	522.6	496.4	399.2	226.8	2120.1
1S(L)	Girder	412.3	470.5	459.1	376.6	221.4	1939.9
	Slab	94.1	26.5	26.1	21.4	12.0	180.1
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	506.5	497.0	485.2	398.0	233.4	2120.0
1S(R)	Girder	444.8	486.6	458.4	377.7	210.5	1978.1
	Slab	26.6	27.7	26.1	21.9	40.9	143.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	471.4	514.3	484.5	399.6	251.4	2121.3
2S	Girder	409.2	462.9	448.3	375.7	215.7	1911.8
	Slab	93.7	26.0	25.5	21.7	42.5	209.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	502.9	488.9	473.8	397.4	258.2	2121.1
1R(L)	Girder	351.8	416.6	448.7	383.0	240.8	1840.9
	Slab	24.9	23.5	25.5	21.8	13.3	108.9
	Railing	166.1	0.0	0.0	0.0	0.0	166.1
	Total	542.9	440.1	474.2	404.7	254.1	2116.0
1R(R)	Girder	458.4	495.1	459.0	347.7	181.7	1941.9
	Slab	27.4	28.1	26.1	19.8	10.6	112.1
	Railing	0.0	0.0	0.0	0.0	64.6	64.6
	Total	485.8	523.3	485.1	367.6	256.9	2118.6
2R	Girder	360.3	414.6	435.5	346.4	200.2	1756.9
	Slab	25.7	23.3	24.8	19.7	12.3	105.8
	Railing	172.9	0.0	0.0	0.0	78.7	251.6
	Total	558.9	437.9	460.2	366.1	291.2	2114.3
1SR(L)	Girder	318.9	405.3	434.0	375.0	239.4	1772.7
	Slab	69.4	23.0	24.7	21.3	13.2	151.6
	Railing	192.3	0.0	0.0	0.0	0.0	192.3
	Total	580.6	428.3	458.7	396.3	252.7	2116.6
1SR(R)	Girder	453.7	485.1	444.8	346.4	169.1	1899.0
	Slab	27.2	27.6	25.4	20.3	30.4	130.9
	Railing	0.0	0.0	0.0	0.0	89.2	89.2
	Total	480.9	512.7	470.2	366.7	288.7	2119.1
2SR	Girder	323.2	393.8	408.3	335.8	183.9	1644.9
	Slab	71.2	22.2	23.3	19.5	35.1	171.3
	Railing	197.0	0.0	0.0	0.0	102.1	299.1
	Total	591.4	416.1	431.6	355.3	321.0	2115.3

Table A.20. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 2 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Total (7)
No SR	Girder	545.8	618.7	528.6	289.7	1982.7
	Slab	36.1	43.7	37.4	18.0	135.2
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	581.8	662.4	565.9	307.8	2117.9
1S(L)	Girder	497.3	591.0	520.4	296.3	1905.0
	Slab	116.1	41.5	36.7	18.5	212.9
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	613.5	632.5	557.2	314.8	2117.9
1S(R)	Girder	541.0	604.5	521.2	279.0	1945.6
	Slab	35.8	42.7	37.4	57.6	173.5
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	576.8	647.2	558.6	336.6	2119.1
2S	Girder	493.0	577.4	512.5	284.1	1867.0
	Slab	115.5	40.5	36.6	59.4	252.0
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	608.5	617.8	549.2	343.5	2119.0
1R(L)	Girder	416.6	537.0	521.2	319.6	1794.4
	Slab	32.8	37.4	36.9	20.1	127.3
	Railing	191.7	0.0	0.0	0.0	191.7
	Total	641.1	574.4	558.2	339.7	2113.4
1R(R)	Girder	558.8	612.7	491.2	236.4	1899.1
	Slab	37.0	43.3	34.6	16.2	131.1
	Railing	0.0	0.0	0.0	85.8	85.8
	Total	595.8	656.1	525.8	338.3	2116.0
2R	Girder	426.6	527.0	476.4	256.6	1686.6
	Slab	33.9	36.7	33.6	18.3	122.5
	Railing	200.2	0.0	0.0	102.0	302.2
	Total	660.8	563.6	510.0	376.9	2111.3
1SR(L)	Girder	375.0	520.0	504.2	315.7	1714.9
	Slab	83.6	36.6	35.7	20.0	175.8
	Railing	223.3	0.0	0.0	0.0	223.3
	Total	681.8	556.7	539.8	335.7	2114.0
1SR(R)	Girder	551.4	594.4	482.8	217.1	1845.7
	Slab	36.6	42.0	34.8	41.6	155.0
	Railing	0.0	0.0	0.0	115.7	115.7
	Total	587.9	636.4	517.6	374.4	2116.4
2SR	Girder	378.7	493.7	451.3	231.5	1555.2
	Slab	85.6	34.6	32.4	46.7	199.4
	Railing	228.4	0.0	0.0	129.2	357.7
	Total	692.8	528.3	483.7	407.5	2112.3

Table A.21. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 2 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Total (6)
No SR	Girder	705.2	816.6	424.6	1946.4
	Slab	55.3	77.7	33.3	166.3
	Railing	0.0	0.0	0.0	0.0
	Total	760.5	894.3	458.0	2112.7
1S(L)	Girder	635.1	785.4	429.1	1849.6
	Slab	155.2	74.5	33.6	263.3
	Railing	0.0	0.0	0.0	0.0
	Total	790.3	859.9	462.7	2112.9
1S(R)	Girder	696.1	796.9	399.6	1892.6
	Slab	54.7	76.3	90.4	221.4
	Railing	0.0	0.0	0.0	0.0
	Total	750.8	873.2	489.9	2114.0
2S	Girder	627.5	765.8	402.3	1795.6
	Slab	153.7	73.0	91.7	318.4
	Railing	0.0	0.0	0.0	0.0
	Total	781.2	838.8	494.0	2114.0
1R(L)	Girder	522.4	742.7	457.2	1722.3
	Slab	49.3	70.0	35.5	154.8
	Railing	230.4	0.0	0.0	230.4
	Total	802.1	812.7	492.6	2107.5
1R(R)	Girder	721.1	775.1	332.7	1828.9
	Slab	56.3	73.5	29.6	159.4
	Railing	0.0	0.0	121.6	121.6
	Total	777.5	848.6	483.9	2110.0
2R	Girder	532.4	692.2	351.3	1576.0
	Slab	50.4	64.8	31.6	146.9
	Railing	241.3	0.0	140.7	382.0
	Total	824.2	757.0	523.6	2104.9
1SR(L)	Girder	464.3	715.3	445.9	1625.6
	Slab	109.1	67.8	34.8	211.7
	Railing	271.0	0.0	0.0	271.0
	Total	844.4	783.1	480.7	2108.3
1SR(R)	Girder	705.8	753.2	299.6	1758.6
	Slab	55.4	72.2	63.7	191.3
	Railing	0.0	0.0	160.4	160.4
	Total	761.2	825.5	523.8	2110.4
2SR	Girder	464.1	646.4	308.1	1418.7
	Slab	110.6	61.4	68.3	240.3
	Railing	275.0	0.0	172.2	447.2
	Total	849.8	707.8	548.6	2106.2

Table A.22. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 3 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Girder 6 (8)	Girder 7 (9)	Total (10)
No SR	Girder	439.0	531.6	561.6	543.6	465.1	328.4	139.8	3009.1
	Slab	25.8	30.2	31.9	31.0	26.4	18.6	6.8	170.7
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	464.8	561.7	593.5	574.6	491.5	347.1	146.6	3179.8
1S(L)	Girder	408.9	514.0	549.3	538.5	463.9	329.4	142.2	2946.3
	Slab	90.7	29.1	31.2	30.7	26.3	18.7	7.0	233.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	499.5	543.1	580.5	569.1	490.3	348.1	149.2	3179.9
1S(R)	Girder	439.2	529.0	556.1	535.0	454.6	336.3	143.8	2994.0
	Slab	25.8	30.0	31.6	30.5	25.9	19.6	23.9	187.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	465.0	559.1	587.7	565.5	480.5	355.9	167.7	3181.4
2S	Girder	408.8	511.2	544.0	530.0	453.6	337.0	146.0	2930.6
	Slab	90.8	28.9	30.9	30.2	25.8	19.7	24.4	250.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	499.7	540.1	574.9	560.2	479.4	356.6	170.4	3181.4
1R(L)	Girder	352.1	459.9	540.3	542.0	471.5	337.0	148.6	2851.5
	Slab	24.1	25.9	30.7	30.9	26.7	19.1	7.3	164.8
	Railing	158.3	0.0	0.0	0.0	0.0	0.0	0.0	158.3
	Total	534.5	485.8	571.1	572.9	498.2	356.1	155.9	3174.6
1R(R)	Girder	441.3	533.4	562.3	541.6	457.8	313.9	126.2	2976.5
	Slab	25.9	30.3	32.0	30.8	26.0	17.9	6.1	169.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	33.3	33.3
	Total	467.2	563.6	594.3	572.4	483.8	331.8	165.6	3178.7
2R	Girder	354.0	461.4	541.0	539.9	463.7	320.8	133.1	2814.0
	Slab	24.3	26.0	30.8	30.7	26.3	18.3	6.6	162.9
	Railing	159.4	0.0	0.0	0.0	0.0	0.0	37.1	196.5
	Total	537.7	487.4	571.8	570.7	490.0	339.0	176.8	3173.4
1SR(L)	Girder	320.8	451.3	523.6	531.8	466.3	335.6	150.5	2779.9
	Slab	67.5	25.7	29.8	30.3	26.5	19.0	7.5	206.3
	Railing	188.8	0.0	0.0	0.0	0.0	0.0	0.0	188.8
	Total	577.1	477.0	553.5	562.1	492.8	354.7	158.0	3175.0
1SR(R)	Girder	442.3	531.1	556.5	531.7	444.7	316.3	119.7	2942.3
	Slab	26.0	30.1	31.7	30.3	25.4	18.6	18.1	180.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	56.5	56.5
	Total	468.3	561.3	588.2	562.0	470.0	335.0	194.4	3179.0
2SR	Girder	322.9	449.9	518.6	519.9	445.3	320.9	127.0	2704.6
	Slab	68.1	25.6	29.6	29.7	25.4	18.9	19.9	217.1
	Railing	190.6	0.0	0.0	0.0	0.0	0.0	61.7	252.3
	Total	581.7	475.5	548.2	549.6	470.7	339.8	208.6	3174.0

Table A.23. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 3 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Girder 6 (8)	Total (9)
No SR	Girder	304.1	575.6	699.8	676.6	503.6	210.7	2970.4
	Slab	16.2	40.6	49.3	47.6	35.5	11.3	200.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	320.4	616.2	749.1	724.2	539.1	222.1	3171.0
1S(L)	Girder	299.1	570.4	678.4	666.0	499.6	212.1	2925.6
	Slab	65.4	40.9	47.7	46.9	35.2	11.4	247.6
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	364.6	611.2	726.1	712.9	534.9	223.5	3173.1
1S(R)	Girder	304.9	570.5	688.0	655.4	504.9	217.1	2940.8
	Slab	16.3	40.2	48.5	46.1	36.4	45.2	232.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	321.2	610.7	736.5	701.5	541.3	262.3	3173.5
2S	Girder	299.3	565.0	667.2	645.4	500.7	217.8	2895.4
	Slab	65.6	40.5	47.0	45.4	36.1	45.5	280.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	364.9	605.5	714.2	690.9	536.7	263.3	3175.5
1R(L)	Girder	228.2	529.1	686.4	676.9	508.5	216.8	2845.9
	Slab	14.0	37.3	48.4	47.6	35.8	11.6	194.9
	Railing	126.8	0.0	0.0	0.0	0.0	0.0	126.8
	Total	369.1	566.4	734.8	724.5	544.3	228.4	3167.6
1R(R)	Girder	308.3	578.7	699.2	666.2	472.8	163.8	2889.1
	Slab	16.4	40.8	49.3	46.9	33.4	9.7	196.5
	Railing	0.0	0.0	0.0	0.0	0.0	83.2	83.2
	Total	324.8	619.5	748.5	713.1	506.3	256.6	3168.8
2R	Girder	231.2	531.5	685.7	666.2	476.7	168.0	2759.4
	Slab	14.2	37.5	48.4	46.9	33.7	9.9	190.6
	Railing	129.0	0.0	0.0	0.0	0.0	86.2	215.3
	Total	374.5	568.9	734.1	713.2	510.3	264.2	3165.2
1SR(L)	Girder	217.3	524.1	660.5	663.4	503.2	218.3	2786.7
	Slab	49.3	38.0	46.6	46.8	35.4	11.7	227.8
	Railing	155.5	0.0	0.0	0.0	0.0	0.0	155.5
	Total	422.1	562.0	707.1	710.2	538.6	230.0	3170.0
1SR(R)	Girder	309.6	573.1	685.7	641.5	471.3	161.3	2842.4
	Slab	16.5	40.4	48.4	45.2	34.3	34.2	219.0
	Railing	0.0	0.0	0.0	0.0	0.0	109.7	109.7
	Total	326.1	613.4	734.0	686.7	505.7	305.2	3171.1
2SR	Girder	219.9	520.3	647.1	628.8	469.2	165.2	2650.5
	Slab	50.2	37.7	45.7	44.3	34.1	35.4	247.5
	Railing	158.2	0.0	0.0	0.0	0.0	113.6	271.8
	Total	428.4	558.0	692.7	673.1	503.3	314.2	3169.8

Table A.24. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 3 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Total (7)
No SR	Girder	445.0	917.0	968.4	577.2	2907.6
	Slab	34.7	87.1	92.1	45.0	258.9
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	479.7	1004.1	1060.4	622.3	3166.5
1S(L)	Girder	422.2	898.6	953.5	577.8	2852.1
	Slab	94.0	85.9	90.6	45.0	315.6
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	516.2	984.5	1044.1	622.8	3167.7
1S(R)	Girder	447.4	904.3	944.1	536.6	2832.3
	Slab	34.9	85.8	90.1	124.1	334.9
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	482.2	990.1	1034.2	660.7	3167.2
2S	Girder	423.7	886.1	929.5	536.5	2775.9
	Slab	94.7	84.6	88.6	124.4	392.4
	Railing	0.0	0.0	0.0	0.0	0.0
	Total	518.4	970.8	1018.2	660.9	3168.2
1R(L)	Girder	354.2	873.1	968.3	588.5	2784.1
	Slab	31.0	82.7	92.0	45.9	251.6
	Railing	126.7	0.0	0.0	0.0	126.7
	Total	511.9	955.8	1060.3	634.4	3162.5
1R(R)	Girder	460.4	918.5	909.3	447.8	2736.0
	Slab	35.9	87.2	86.0	40.4	249.4
	Railing	0.0	0.0	0.0	175.4	175.4
	Total	496.3	1005.7	995.3	663.6	3160.9
2R	Girder	364.9	872.4	907.6	455.9	2600.8
	Slab	32.1	82.6	85.8	41.2	241.7
	Railing	133.5	0.0	0.0	180.6	314.1
	Total	530.5	955.0	993.4	677.6	3156.5
1SR(L)	Girder	319.9	851.7	948.2	588.3	2708.1
	Slab	66.9	81.6	90.0	45.8	284.3
	Railing	169.9	0.0	0.0	0.0	169.9
	Total	556.7	933.3	1038.2	634.1	3162.3
1SR(R)	Girder	460.7	898.1	884.3	402.0	2645.1
	Slab	35.9	85.2	84.5	88.1	293.6
	Railing	0.0	0.0	0.0	222.1	222.1
	Total	496.6	983.3	968.8	712.2	3160.8
2SR	Girder	328.6	830.8	862.9	408.0	2430.3
	Slab	69.6	79.4	82.3	90.2	321.5
	Railing	177.2	0.0	0.0	227.4	404.6
	Total	575.4	910.2	945.2	725.6	3156.4

Table A.25. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 4 lanes, Span L=80 ft, Spacing S = 6 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 4 (6)**	**Girder 5 (7)**	**Girder 6 (8)**	**Girder 7 (9)**	**Girder 8 (10)**	**Girder 9 (11)**	**Total (12)**
No SR	Girder	420.0	530.1	581.8	600.8	586.4	534.5	422.3	263.2	76.3	4015.4
	Slab	24.5	30.1	33.0	34.1	33.3	30.3	23.9	15.0	3.0	227.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	444.5	560.2	614.8	634.9	619.7	564.9	446.2	278.2	79.4	4242.8
1S(L)	Girder	393.0	516.1	569.4	594.9	584.3	534.4	422.9	264.0	77.2	3956.1
	Slab	86.0	29.3	32.3	33.8	33.2	30.3	24.0	15.0	3.1	286.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	479.0	545.3	601.7	628.7	617.5	564.7	446.8	279.0	80.3	4243.0
1S(R)	Girder	420.8	529.9	580.4	597.6	580.4	526.0	413.4	275.6	85.7	4009.8
	Slab	24.6	30.1	33.0	34.0	33.0	29.9	23.5	16.3	10.3	234.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	445.4	560.0	613.3	631.5	613.5	555.9	436.9	291.8	96.0	4244.4
2S	Girder	393.7	515.7	568.0	591.7	578.4	525.9	414.0	276.3	86.5	3950.2
	Slab	86.2	29.2	32.2	33.6	32.9	29.9	23.5	16.3	10.5	294.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	479.9	544.9	600.3	625.3	611.3	555.8	437.5	292.6	96.9	4244.6
1R(L)	Girder	339.2	463.8	562.1	599.3	591.8	540.9	427.3	266.4	77.3	3868.2
	Slab	22.9	26.1	31.9	34.0	33.6	30.7	24.2	15.1	3.1	221.7
	Railing	147.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	147.7
	Total	509.8	489.9	594.0	633.3	625.4	571.6	451.5	281.5	80.4	4237.5
1R(R)	Girder	420.2	530.4	582.2	601.0	585.9	532.7	418.5	260.4	76.5	4007.7
	Slab	24.5	30.1	33.0	34.1	33.3	30.2	23.7	14.9	2.5	226.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.1	8.1
	Total	444.7	560.5	615.2	635.1	619.2	562.9	442.2	275.3	87.1	4242.4
2R	Girder	339.4	464.1	562.5	599.5	591.3	539.0	423.5	263.5	77.5	3860.3
	Slab	22.9	26.1	31.9	34.0	33.6	30.6	24.0	15.1	2.5	220.8
	Railing	147.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.2	156.0
	Total	510.1	490.2	594.4	633.5	624.9	569.6	447.5	278.6	88.2	4237.1
1SR(L)	Girder	309.7	456.7	545.2	588.6	586.2	538.7	426.9	267.0	78.7	3797.6
	Slab	64.0	26.1	31.0	33.5	33.3	30.6	24.2	15.2	3.2	261.0
	Railing	179.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	179.3
	Total	553.0	482.8	576.2	622.0	619.5	569.2	451.1	282.1	81.8	4237.8
1SR(R)	Girder	421.3	530.6	581.3	598.3	580.4	523.7	407.2	265.0	74.8	3982.6
	Slab	24.6	30.1	33.0	34.0	33.0	29.8	23.2	15.8	8.0	231.4
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.4	28.4
	Total	445.9	560.7	614.3	632.3	613.4	553.5	430.4	280.8	111.2	4242.5
2SR	Girder	310.5	456.9	544.6	586.1	580.2	527.8	411.6	268.5	76.6	3762.7
	Slab	64.3	26.1	31.0	33.3	33.0	30.0	23.4	16.0	8.3	265.3
	Railing	179.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	29.5	209.4
	Total	554.7	483.0	575.6	619.4	613.2	557.8	435.1	284.5	114.3	4237.4

Table A.26. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 4 lanes, Span L=80 ft, Spacing S = 8 ft)

Case (1)	Zone (2)	Girder 1 (3)	Girder 2 (4)	Girder 3 (5)	Girder 4 (6)	Girder 5 (7)	Girder 6 (8)	Girder 7 (9)	Total (10)
No SR	Girder	524.1	709.9	771.4	758.9	650.8	424.7	124.6	3964.5
	Slab	34.0	50.1	54.3	53.5	45.7	30.0	6.7	274.5
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	558.2	760.1	825.7	812.4	696.5	454.7	131.4	4239.0
1S(L)	Girder	487.3	690.6	757.4	754.6	650.3	425.7	126.1	3892.0
	Slab	109.1	48.9	53.3	53.2	45.7	30.1	6.8	347.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	596.5	739.6	810.7	807.8	696.0	455.7	133.0	4239.2
1S(R)	Girder	525.2	708.8	767.4	750.5	636.5	430.7	133.7	3952.8
	Slab	34.1	50.1	54.0	52.9	44.7	31.3	20.7	287.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	559.3	758.9	821.4	803.5	681.2	462.1	154.4	4240.7
2S	Girder	488.1	689.3	753.5	746.2	636.1	431.6	135.1	3879.9
	Slab	109.4	48.8	53.0	52.7	44.7	31.4	21.0	361.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Total	597.5	738.2	806.5	798.9	680.7	463.0	156.0	4240.8
1R(L)	Girder	413.2	637.7	759.4	763.4	658.7	430.7	127.6	3790.7
	Slab	31.2	44.6	53.5	53.8	46.2	30.4	6.9	266.6
	Railing	175.3	0.0	0.0	0.0	0.0	0.0	0.0	175.3
	Total	619.7	682.3	812.9	817.2	704.9	461.2	134.5	4232.6
1R(R)	Girder	524.8	710.8	772.1	758.2	646.1	415.4	116.5	3943.9
	Slab	34.1	50.2	54.4	53.4	45.4	29.5	5.8	272.8
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	21.4	21.4
	Total	558.8	761.0	826.5	811.7	691.5	444.8	143.8	4238.1
2R	Girder	413.7	638.5	760.1	762.7	653.9	421.1	119.0	3769.1
	Slab	31.2	44.6	53.6	53.7	45.9	29.9	6.0	264.9
	Railing	175.5	0.0	0.0	0.0	0.0	0.0	22.2	197.7
	Total	620.5	683.2	813.7	816.4	699.8	450.9	147.2	4231.7
1SR(L)	Girder	374.7	623.6	737.9	754.0	655.4	430.8	129.6	3706.0
	Slab	79.4	44.3	52.0	53.1	46.0	30.4	7.0	312.3
	Railing	214.4	0.0	0.0	0.0	0.0	0.0	0.0	214.4
	Total	668.5	667.9	789.9	807.1	701.5	461.2	136.6	4232.7
1SR(R)	Girder	526.2	710.2	768.6	749.7	629.9	414.0	111.0	3909.5
	Slab	34.2	50.1	54.1	52.9	44.3	30.3	15.2	281.1
	Railing	0.0	0.0	0.0	0.0	0.0	0.0	47.4	47.4
	Total	560.4	760.4	822.7	802.5	674.1	444.3	173.5	4238.0
2SR	Girder	376.0	623.4	735.1	744.7	634.2	419.3	114.5	3647.2
	Slab	79.7	44.3	51.8	52.5	44.6	30.6	15.9	319.5
	Railing	215.4	0.0	0.0	0.0	0.0	0.0	49.5	264.9
	Total	671.1	667.8	786.8	797.2	678.8	449.9	179.9	4231.6

Table A.27. Calculated Negative Bending Moments (kip-ft) at Critical Section Based on FEA Results (Two-Span Negative, 4 lanes, Span L=80 ft, Spacing S = 12 ft)

Case (1)	**Zone (2)**	**Girder 1 (3)**	**Girder 2 (4)**	**Girder 3 (5)**	**Girder 5 (7)**	**Girder 4 (6)**	**Total (8)**
No SR	Girder	707.7	1067.7	1077.6	796.1	228.1	3877.2
	Slab	55.4	101.8	102.4	75.6	18.0	353.3
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	763.1	1169.6	1180.0	871.7	246.1	4230.5
1S(L)	Girder	651.3	1039.5	1065.3	795.7	230.8	3782.6
	Slab	153.7	99.4	101.1	75.6	18.2	448.0
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	804.9	1138.9	1166.4	871.3	249.0	4230.6
1S(R)	Girder	708.8	1063.4	1060.5	788.4	232.0	3853.2
	Slab	55.5	101.5	100.7	75.8	45.3	378.9
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	764.3	1164.9	1161.3	864.2	277.4	4232.1
2S	Girder	652.0	1035.1	1048.5	788.0	234.3	3757.9
	Slab	154.0	99.0	99.5	75.8	46.0	474.2
	Railing	0.0	0.0	0.0	0.0	0.0	0.0
	Total	805.9	1134.1	1148.0	863.8	280.3	4232.1
1R(L)	Girder	542.4	996.0	1079.6	805.8	235.5	3659.3
	Slab	49.8	94.4	102.5	76.5	18.5	341.7
	Railing	221.3	0.0	0.0	0.0	0.0	221.3
	Total	813.6	1090.4	1182.1	882.3	254.0	4222.3
1R(R)	Girder	709.7	1069.6	1075.3	777.7	197.8	3830.1
	Slab	55.6	102.0	102.2	73.9	15.9	349.6
	Railing	0.0	0.0	0.0	0.0	48.9	48.9
	Total	765.3	1171.6	1177.4	851.6	262.6	4228.6
2R	Girder	544.0	997.7	1077.3	786.6	203.3	3608.9
	Slab	50.0	94.5	102.3	74.7	16.4	337.9
	Railing	222.1	0.0	0.0	0.0	51.4	273.5
	Total	816.0	1092.3	1179.6	861.3	271.1	4220.3
1SR(L)	Girder	485.6	968.2	1057.6	802.1	238.2	3551.7
	Slab	109.1	92.6	100.4	76.1	18.7	397.0
	Railing	273.3	0.0	0.0	0.0	0.0	273.3
	Total	868.0	1060.8	1158.0	878.3	257.0	4222.0
1SR(R)	Girder	711.5	1065.5	1056.6	763.5	182.3	3779.5
	Slab	55.7	101.6	100.3	73.6	32.5	363.7
	Railing	0.0	0.0	0.0	0.0	84.7	84.7
	Total	767.3	1167.1	1156.9	837.1	299.5	4227.9
2SR	Girder	487.8	965.4	1036.7	768.1	188.7	3446.7
	Slab	109.8	92.3	98.3	73.9	34.1	408.4
	Railing	275.1	0.0	0.0	0.0	89.0	364.1
	Total	872.7	1057.7	1135.0	842.0	311.8	4219.2

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