AMERICAN UNIVERSITY OF BEIRUT

INFLUENCE OF STIFFNESS OF RAILINGS ON ONE-SPAN MULTI-LANE STEEL GIRDER BRIDGES

by
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AN ABSTRACT OF THE THESIS OF

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The conventional analysis and design of highway bridges ignore the contribution of railings in a bridge deck when calculating the flexural strength of superstructures. In fact, the presence of 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 stiffness of 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 simply supported, two-lane, three-lane, and four-lane steel girder bridges. The finite element program SAP2000 (2013) 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 stiffnesses of railings on either or both edges of the slab are considered. Bridges without railings served as reference cases. The wheel load distribution factor for the reference cases and for cases with 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). Increasing the stiffness of railings has shown to increase the load-carrying capacity when 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 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. AASHTO LRFD wheel load distribution formulae were based on NCHR Project 12-26, which was introduced by Zokaie et al. (1991)

NCHR project (Development of a comprehensive Bridge Specifications and Commentary) was initiated in 1988 with the objective of developing a comprehensive new design code that could eventually replace the AASHTO Standard Specifications for Highway Bridge. The new code is based on a probability based approach. Structural performance is measured in terms of the reliability (or probability of failure). So, the major tool in the development of new code is the reliability analysis procedure. The code provisions are formulated so that the structure designed using the code has a consistent and uniform safety level.

The current AASHTO procedures (Standard Specifications or LRFD) do not consider the influence of 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).

“Previously published research investigated the effect of parapets or railings on distribution factors in steel and prestressed girder bridges. Mabsout et al. (1997) reported the results of parametric study that investigated the influence of sidewalks and railings on wheel load distribution in steel girder highway bridges. Typical one-span, two-lane, simply-supported, composite steel girder bridges were selected to investigate the influence of various parameters such as: span length, girder spacing, raised sidewalks, and the addition of railings on live load distribution. The presence of sidewalks and railings was shown to increase the stiffness of the superstructure and improve the load-carrying capacity of steel bridges by as much as 30%. Chung et al. (2006) conducted a study to investigate the influence of secondary elements and deck cracking on the lateral load distribution of steel girder bridges. It was found that the presence of secondary elements such as lateral bracing and parapets produces load distribution factors up to 40% lower than the AASHTO LRFD values. Conner and Huo (2006) investigated the effect of parapets and bridge aspect ratio on live-load moment distribution bridge girders. The finite element method was used to investigate 34 two-span continuous bridges with different skew angles and overhang lengths. The presence of parapets was shown to reduce distribution factors by as much as 36 and 13% for exterior and interior girders, respectively. Akinci et al. (2008) tested the parapet strength and contribution to live-load response for super-load passages. The results of this study showed that girder distribution factors (GDFs) can be decreased by as much as 30%, depending on the stiffness of the girders and the transverse truck position if the parapets are included in the analysis. Roddenberry et al. (2011) examined the effect of secondary elements on load distribution in prestressed bridge girders. This research showed that including the effect of barriers changes the wheel load distribution and bending moments in girders."

An extensive study by Fawaz et al. (2015) 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 modelled 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. Abou Nooh et al. (2015) followed on this work by studying the influence of the railing stiffness of concrete slab bridges.

Furthermore, a study by Nuwayhid et al. (2015) 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), and most importantly Nuwayhid et al. (2015) form the basis of the current research which addresses the influence of stiffness of railings on simple-span multi-lane steel girder highway bridges.

1.4. Research Objectives

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 the stiffness of railings on the wheel load distribution and the load-carrying capacity of steel girder bridges. Typical one-span, 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 stiffnesses of railings on either or both edges of the slab were considered. The case of one-span bridges with no railings served as reference bridges. AASHTO design trucks (HS20) were assumed, longitudinally and transversally, positioned using influence lines in order to produce the maximum 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 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 stiffness of 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 stiffness of railings on wheel load distribution in multi-span 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 (2013) was selected for the analysis. The finite element method was used to investigate the effect of span length, girder spacing, railing stiffness, on simply supported one-span, 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. 5 Stiffness factors were used for railings (0.5, 1, 2, 3, 4).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 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 bridges without railings were considered as the reference cases. Railings with different stiffness factors were then placed integrally at either or both of the slab edges. The maximum longitudinal bending moments were computed and corresponding wheel load distribution factors on the girders were reported and compared for the reference bridges and the bridges with 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 railings of different stiffnesses to influence the lateral load distribution in steel girder bridges.

2.2. Effect of Railings on Steel Girder Bridges

As mentioned earlier, both AASHTO procedures (Standard Specifications and LRFD) do not consider the influence of raised railings that are built integrally with the bridge deck. In this context, previous research has shown 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 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 increasing stiffness of railings on wheel load distribution in simply-supported, one-span, multi-lane steel girder bridges. Bridge cases were modeled using three-dimensional (3D) finite element analysis subject to static wheel loading.

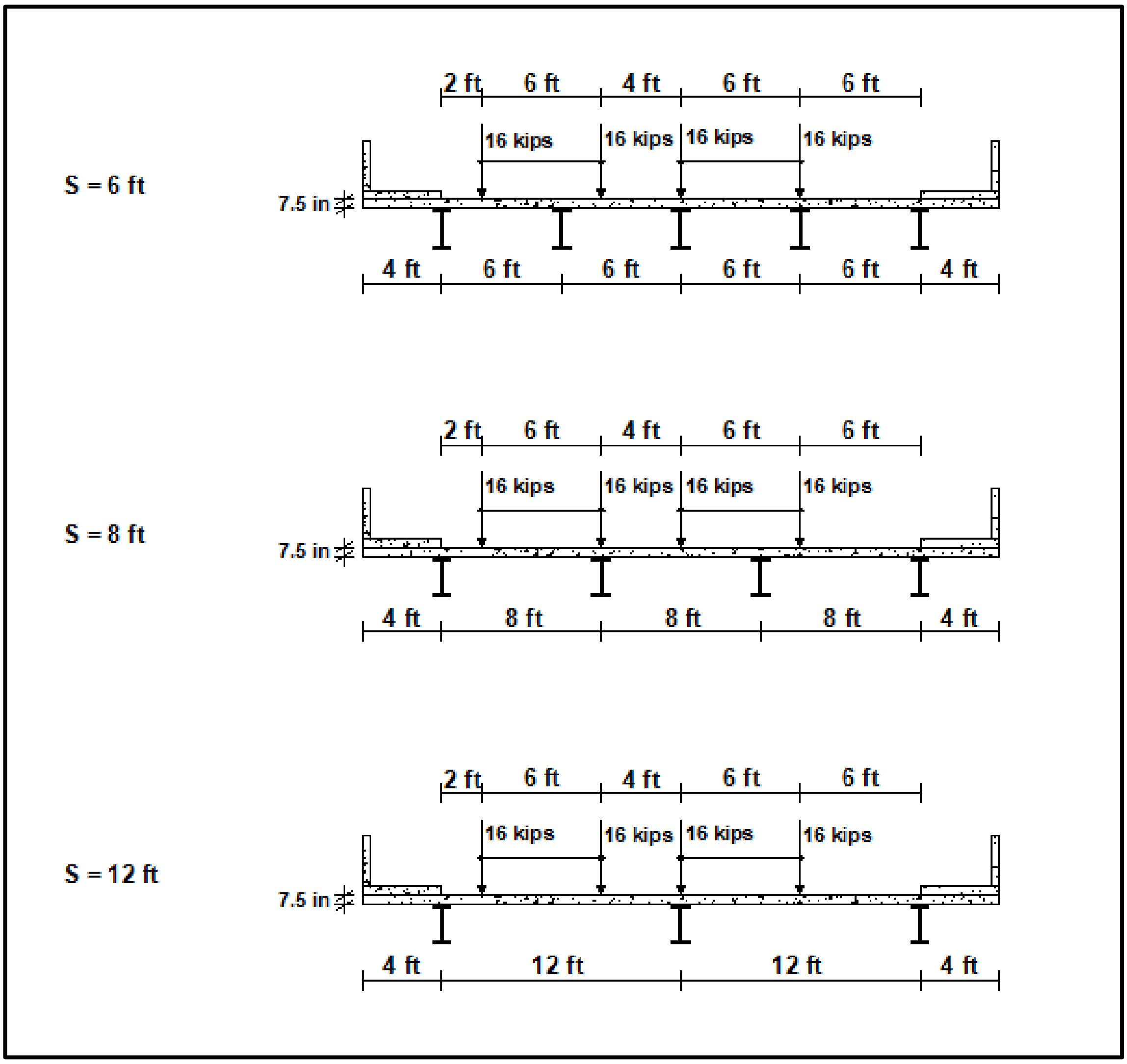


Figure 2.1. Typical Bridge Cross-section - Case of Two Lanes with Sidewalks and Railings on Both Sides

The 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. Railings of several stiffnesses 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

The bridges described below follow the same geometry , properties and loading researched earlier by Nuwayhid et al. (2015)

3.2.1. Bridge Geometry and Properties

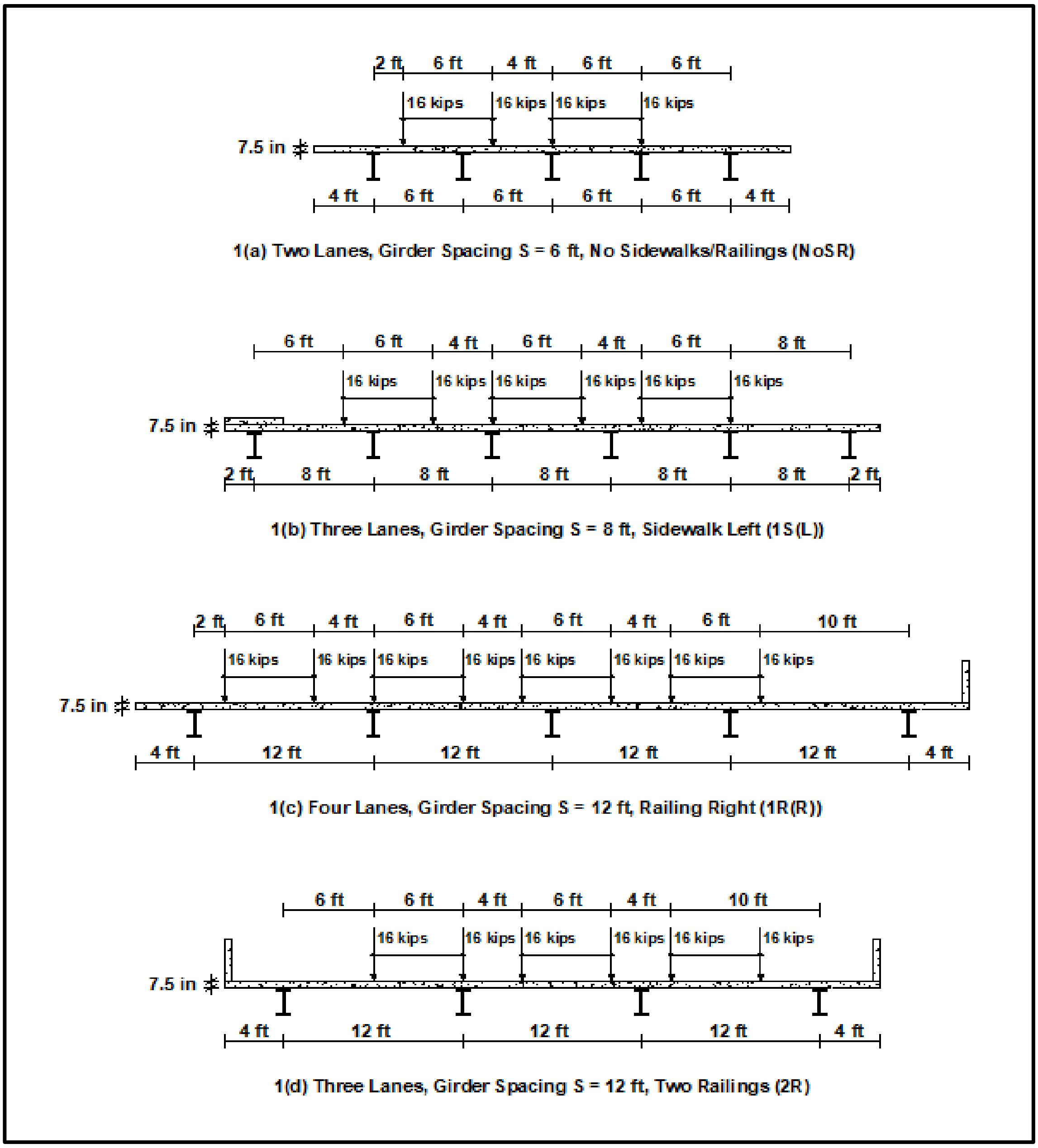
Typical one-span, simply supported, two-, three-, and four-lane steel girder bridges were selected for this study. The longitudinal axis of the bridges was assumed to be at right angles to the supports. The bridge deck consists of a 7.5 in (19.1 cm) reinforced concrete slab supported by W36X160 structural 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), 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 account for the existence of 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 addition to be investigated and that forms the basis of this research is the railings of several stiffnesses which can be present on either or both sides of the bridge deck. 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. Then, we started varying the stiffness of railings to study their effect on load distribution. Four other sizes of railings were considered having a stiffness of half , double, triple and quadruple the stiffness of the typical railing as shown in Table 3.1.

	Size	
	width	height
Typical railing (Stiffness x1)	8 in	30 in
Stiffness x0.5	8 in	24 in
Stiffness x2	8 in	38 in
Stiffness x3	8 in	43 in
Stiffness x4	8 in	48 in

Table 3.1: Approximate sizes of the several railings

It was assumed that the 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 railings (thereafter referred to as the “NoR” 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) as per Nuwayhid et al. (2015) (Though Sidewalks were excluded from our study). Furthermore, Table 3.2 summarizes the variation of parameters among the bridges studied which sum up to a total of 720 bridges studied for a given span to investigate moments and deflections.



1(a) Two Lanes, Girder Spacing S = 6 ft, No Sidewalks/Railings (NoSR)

1(b) Three Lanes, Girder Spacing S = 8 ft, Sidewalk Left (1S(L))

1(c) Four Lanes, Girder Spacing S = 12 ft, Railing Right (1R(R))

1(d) Three Lanes, Girder Spacing S = 12 ft, Two Railings (2R)

Figure 3.1. Typical Bridge Cross-Sections, with and without Sidewalks and/or Railings (Nuwayhid et al. 2015)

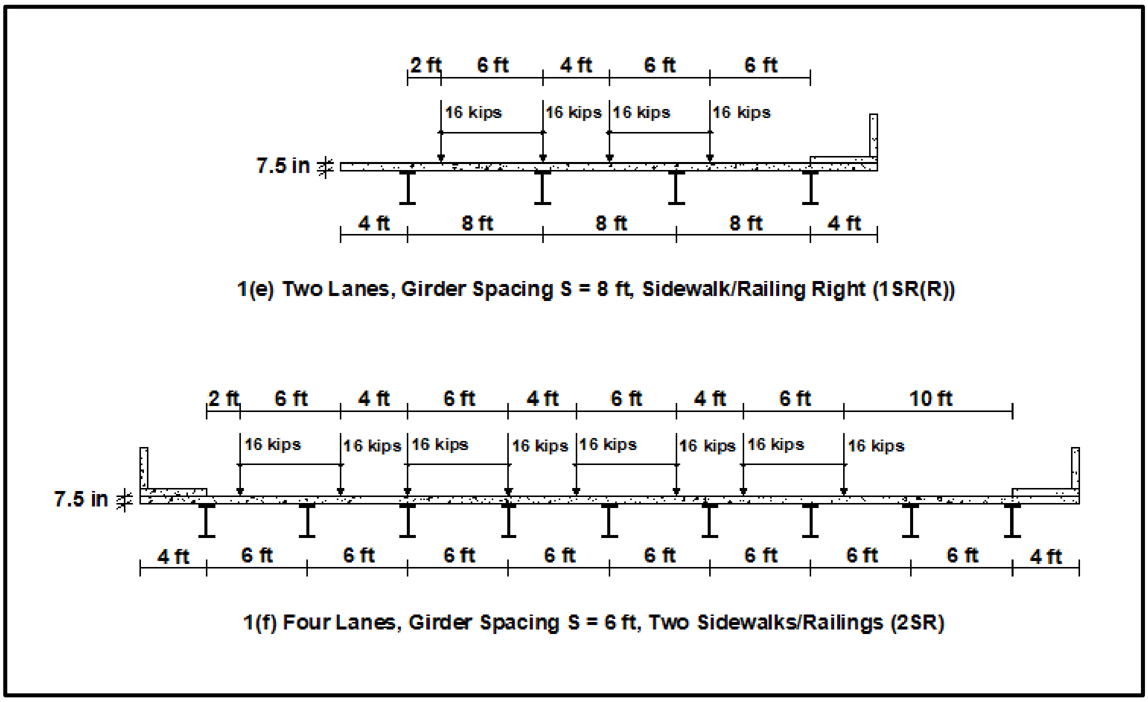


Figure 3.1 (Continued)

Table 3.2. Variable Parameters Investigated and Geometric Characteristics of the Modeled Bridges

Number of Lanes	Span Length L (ft)	Girder Spacing S (ft)	Different Combinations of Railings	Stiffness factor of railings	Total Number of Bridges
2	40 60 80 100 120	6 8 12	NoR 1R(L) 1R(R) 2R	0.5 1 2 3 4	240
3	40 60 80 100 120	6 8 12	NoR 1R(L) 1R(R) 2R	0.5 1 2 3 4	240
4	40 60 80 100 120	6 8 12	NoR 1R(L) 1R(R) 2R	0.5 1 2 3 4	240
Total					720

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 $29x10^6$ 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. For our case "one-span" bridges, the maximum moment was located according to Barre’s theorem, which states

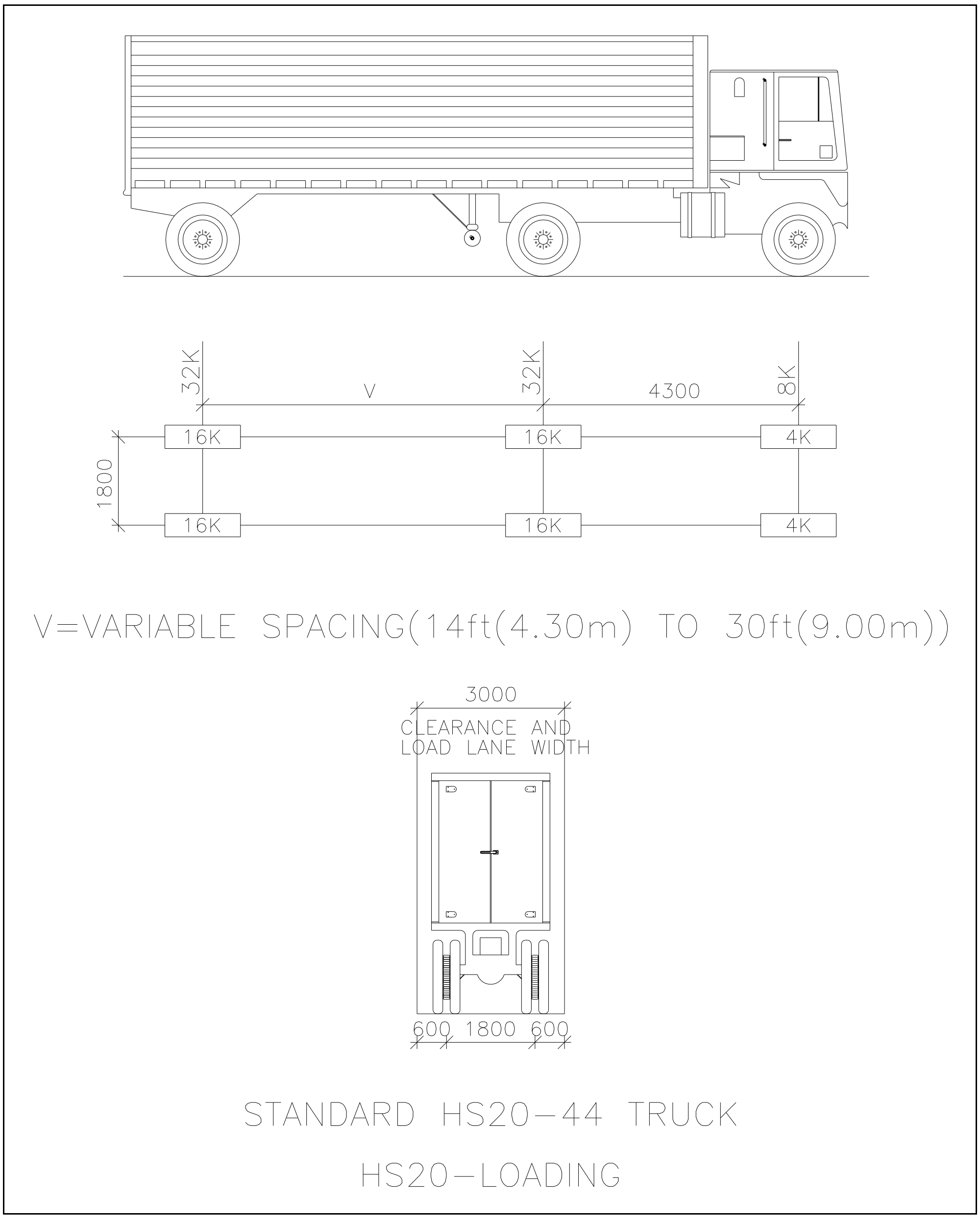


Figure 3.2. Longitudinal and Transversal Sections of AASHTO HS20 Design Truck

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. Figure 3.4 presents the longitudinal positioning of trucks on one-span bridges which was determined based on influence lines in order to produce the maximum moment.

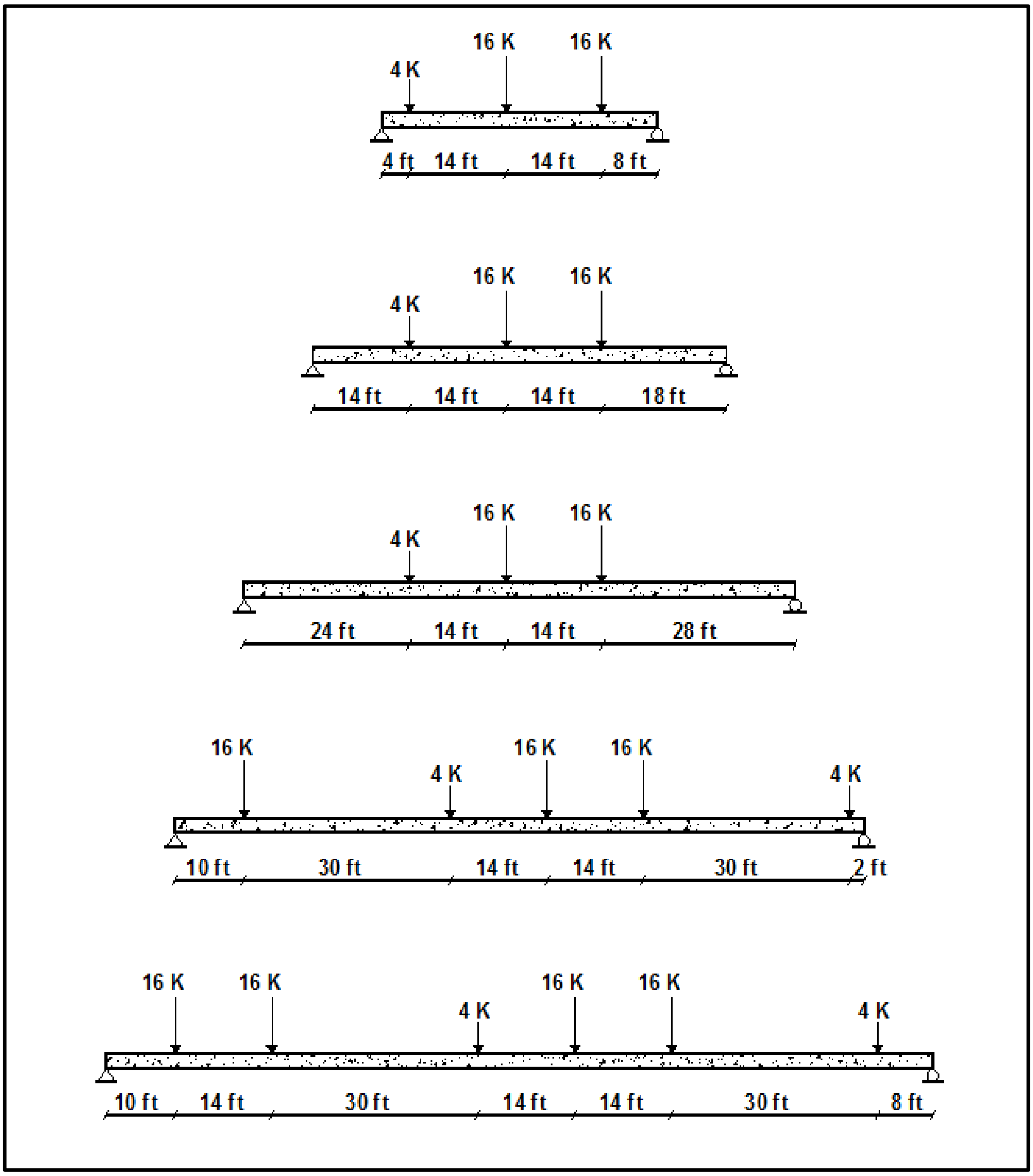


Figure 3.3. Longitudinal Beam Section of a Single-Span Bridge and Critical Position of HS20 Trucks for the Five Different Span Lengths

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 3.4 through 3.6 as per Nuwayhid et al. (2015). In fact, we didn't include sidewalks in our study as Nuwayhid et al. (2015) did, yet; the loading positions aren't affected by the presence or absence of sidewalks.

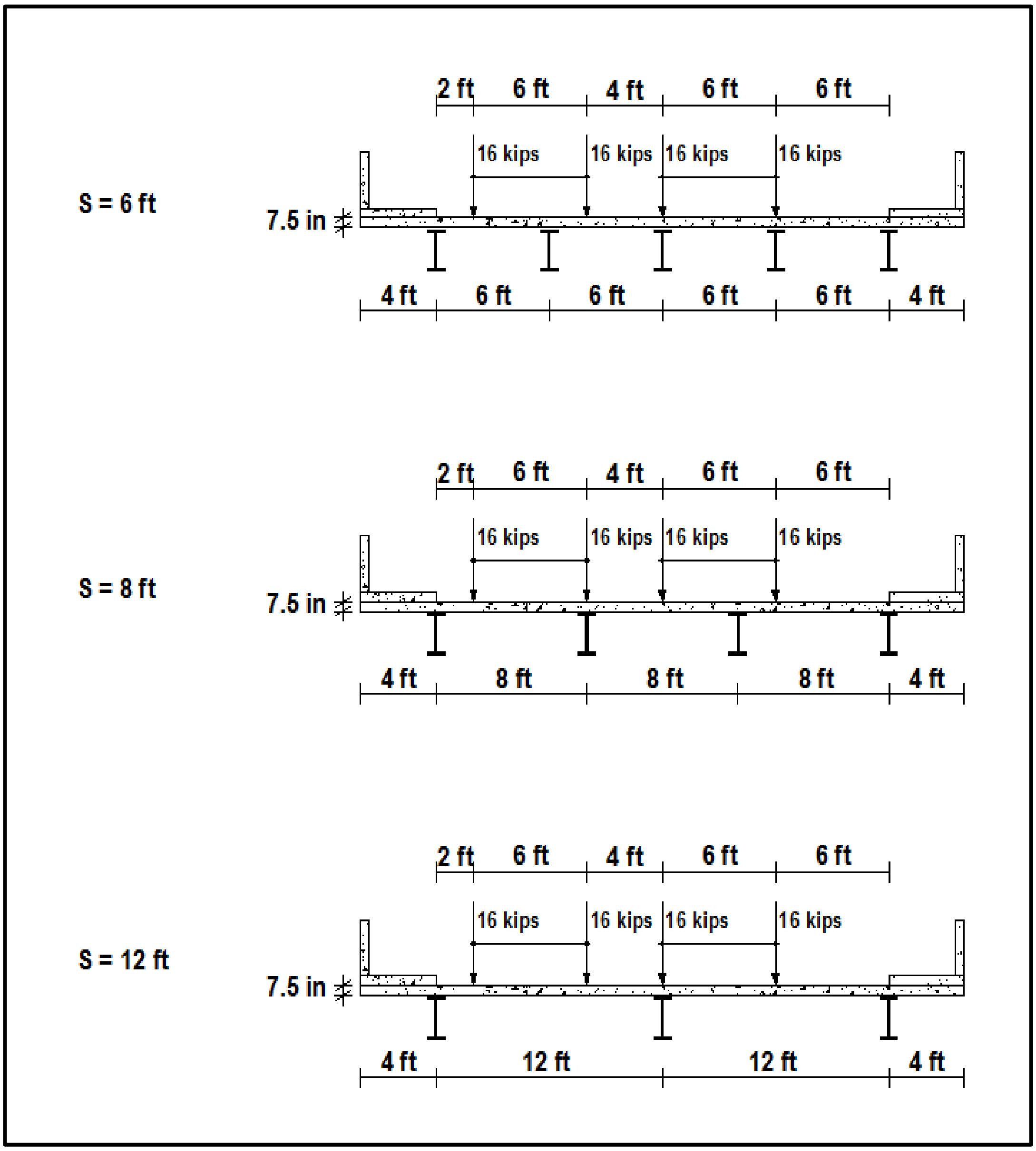


Figure 3.4. Typical Cross-Section of Two-Lane Bridges (Case 2SR) and The Critical Transverse Position of HS20 Trucks for the different Girder Spacing Considered (Nuwayhid et al. 2015)

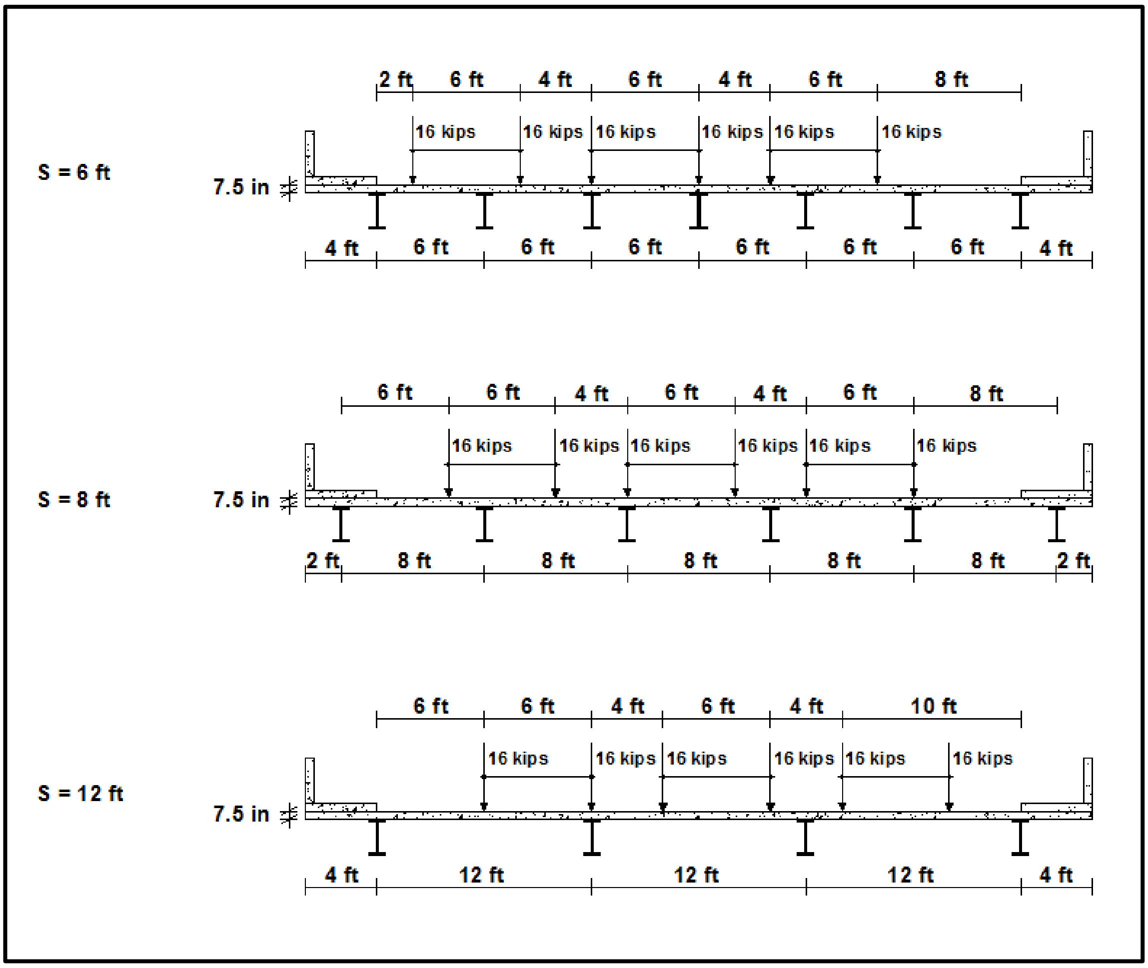


Figure 3.5. Typical Cross-Section of Three-Lane Bridges (Case 2SR) and The Critical Transverse Position of HS20 Trucks for the different Girder Spacing Considered (Nuwayhid et al. 2015)

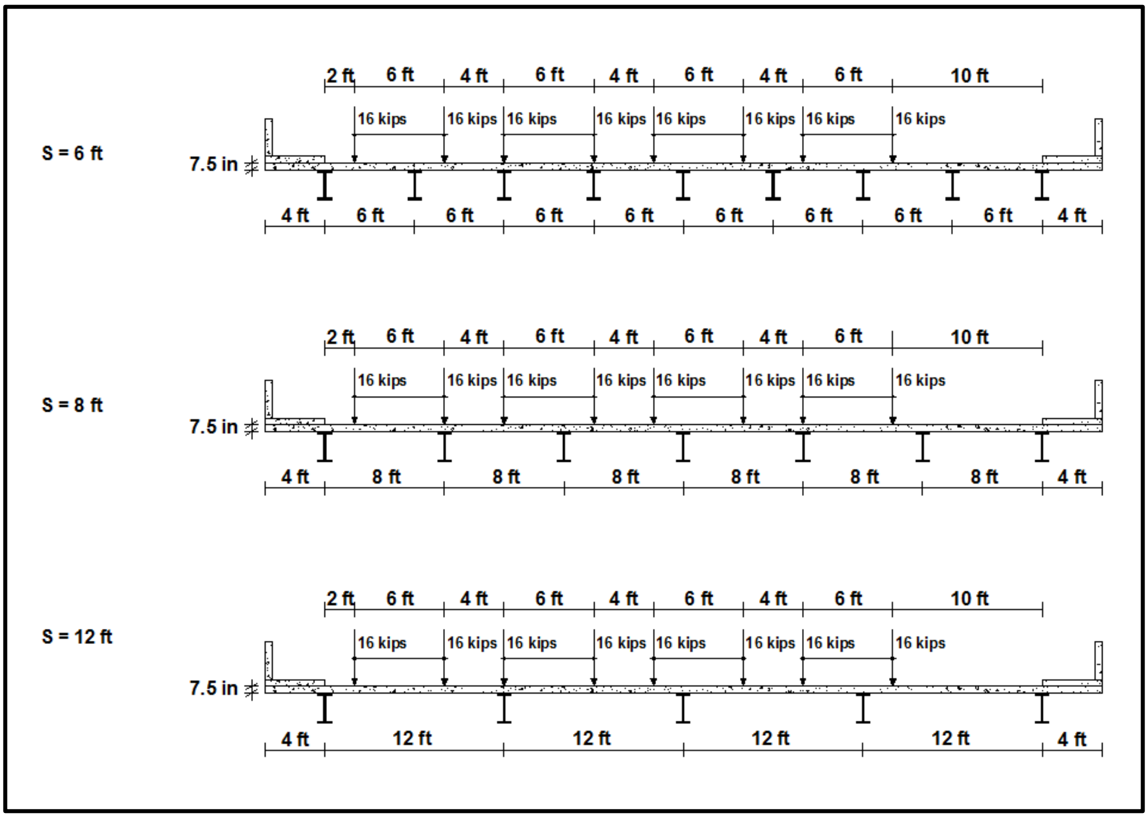


Figure 3.6. Typical Cross-Section of Four-Lane Bridges (Case 2SR) and The Critical Transverse Position of HS20 Trucks for the different Girder Spacing Considered (Nuwayhid et al. 2015)

3.3. Finite Element Analysis

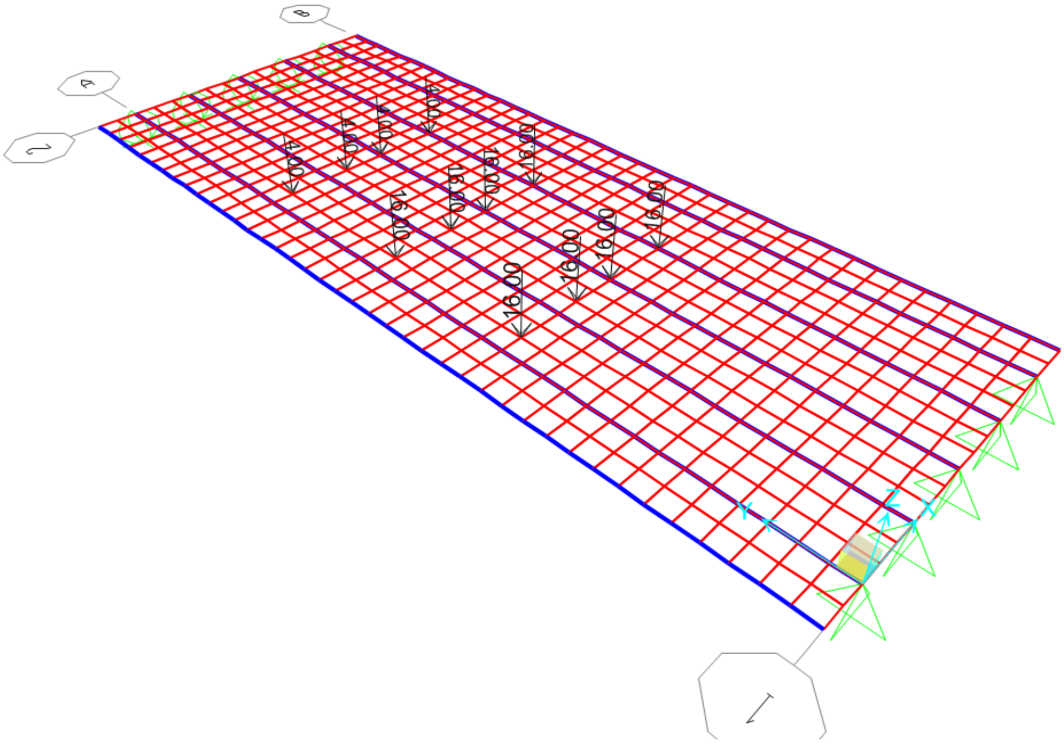
The geometry of a bridge superstructure can be idealized for theoretical analysis in many different ways. The various assumptions and simplifications used in formulating and idealizing the bridge superstructure can have a significant effect on how closely the calculated results match the actual behavior. 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 (2013) 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 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. Hinges were assigned at one bearing location and rollers at the other 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.

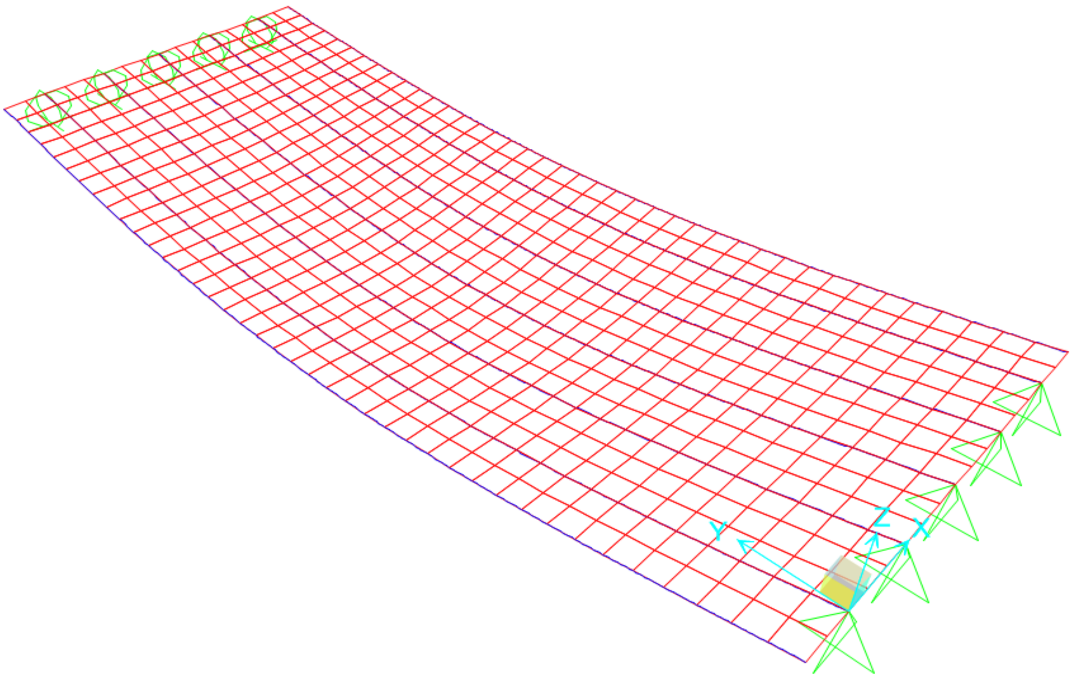
The relevant output to be extracted from the finite element analysis (SAP 2000) includes the deflection at the nodes and the maximum longitudinal bending moment in the girders as well as the transverse bending moments in the slab shell elements. SAP2000 generates the required longitudinal bending moment diagrams in the girders, and contour plots of the transverse bending moments in the slab. Figure 3.8 presents sample finite element plans of truck live loads, deflections, and moment contours for typical 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. Next, the maximum moment in the critical girder was divided by the moment calculated in a single girder subject to truck wheel loads in order to calculate the distribution factor. 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.

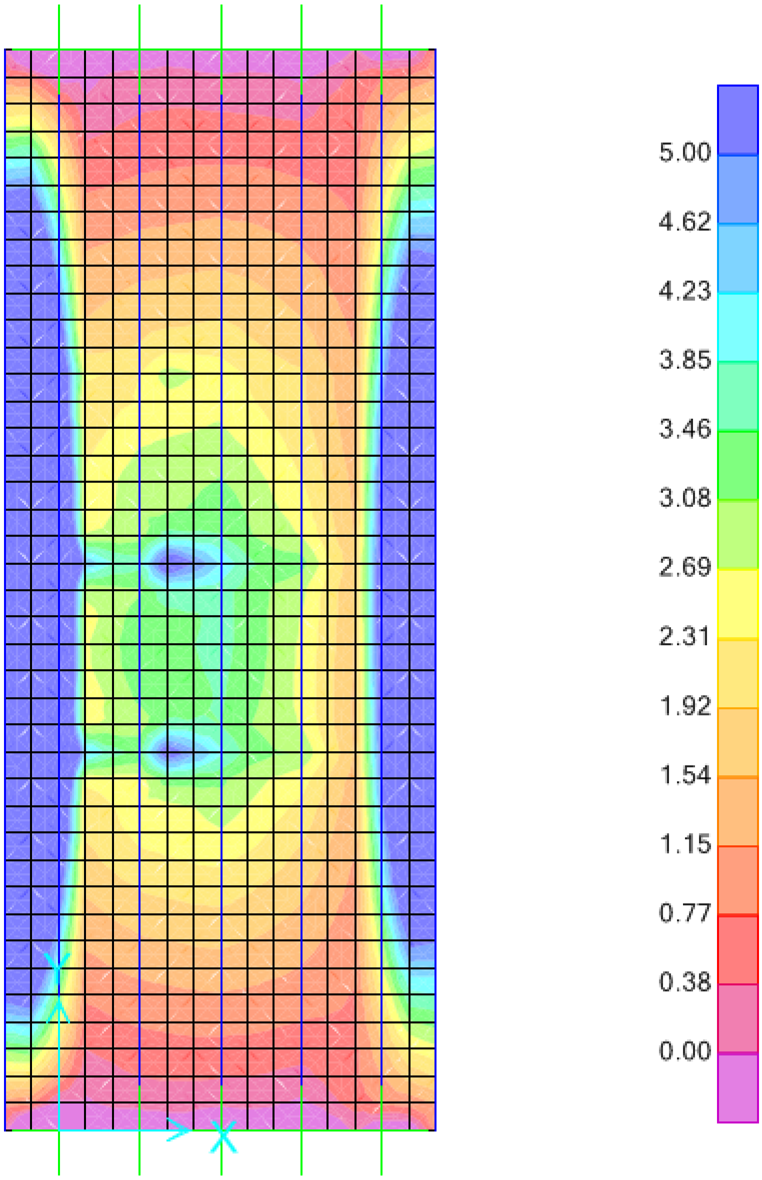
Finally, the finite element results for all bridges with different combinations of railings are summarized and compared to both the reference case (case without railings) as well as the AASHTO standard and LRFD procedures. Recommendations are given to assist bridge engineers in evaluating the capacity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



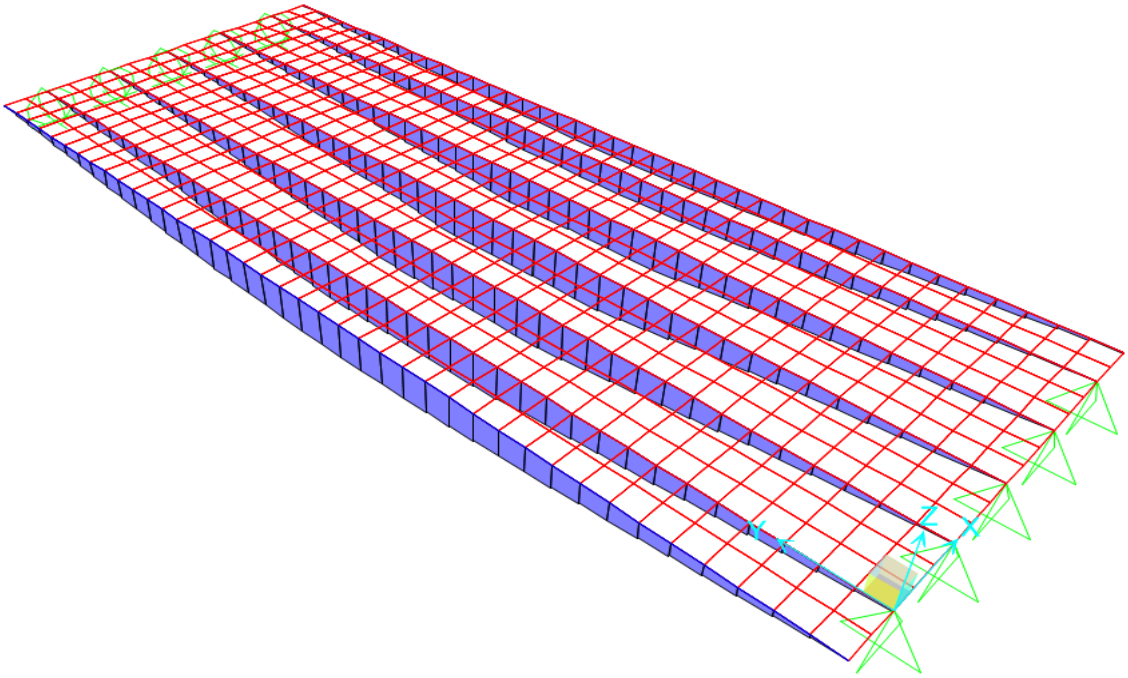
3.7(a) Truck Wheel Live Load



3.7(b) Deformed Shape



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



3.7(d) Longitudinal Bending Moment in Steel Girders

Figure 3.7. Finite Element Model for an 80 ft Span, Two-Lane One-Span Bridge, with 6 ft Girder Spacing (Case 2R)

CHAPTER 4

FINITE ELEMENT ANALYSIS AND DISCUSSION

4.1. Introduction

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 moments and their corresponding distribution factors for bridge cases with railings of stiffness x1. 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. The second section constitutes the core of the research which presents and the effect of increasing the stiffness of railings on resisting truck loads applied on steel girder bridges. This section is divided into 3 sub-sections: the first consists of extracting the maximum moments and their corresponding distribution factors for bridges with railings of stiffness x0.5 , x1 , x2 , x3 and x4 (all bridge cases). Then results are presented in both table and chart formats in such a way that shows clearly the effect of changing the stiffness of railings ; and results are finally assessed with both AASHTO Standard Specifications (2002) and AASHTO LRFD (2010) procedures. The second sub-section aims at pointing out the effect of changing the stiffness of girder and railing combined. 2 bridge cases were studied while varying both railing stiffness and girder stiffness and results were are presented in both table and chart form. The third sub-section consists of extracting the maximum deflections for bridges with railings of stiffness x0.5 , x1 , x2 , x3 and x4 (all bridge cases) . Then results are presented in both table and chart formats in such a way that shows clearly the effect of changing the stiffness of railings on deflections of steel girder bridges.

To note that AASHTO LRFD (2010) procedures are represented by AASHTO (NCHRP) which 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 railings assisted in resisting the wheel loads and the bending stresses were redistributed in the bridge deck. The 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 railings. Typically, the bending moment in an exterior girder is higher than the bending moment in an interior girder due to the contribution of railings. However, if the 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 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. Bridges with Railing Stiffness x1

The results for the one-span bridges with a railing stiffness factor x1 have been reported by Nuwayhid et al. (2015) 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 with railings of stiffness x1; the bridges cases with railings of stiffness x2,x3,x4,x0.5.

This section will show how railings contribute to decreasing the maximum moment in critical girder (whether interior or exterior). Only one railing stiffness is included in this section , since the aim here is only to show the influence of presence of railings . The influence of stiffness of railings will be shown in section 4.3.

4.2.1. Two-Lane Bridges

Table 4.1 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no railing on the bridge. However , when introducing a railing or parapet on either side or on both sides of the bridge deck the concrete slab and railing will contribute up to 45 % to the total bending moment of the exterior girder in bridges with railing stiffness x1.

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 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.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.

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 , stiffness x1)

(Stiff = 1, 2 Lanes, L=80 ft, S = 6 ft)								
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Total	Max
no R	Girder	514.8	503.3	472.9	405.4	311.5	2208.0	514.8
	Slab	28.1	30.4	28.2	20.9	14.0	121.6	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	542.9	533.7	501.1	426.4	325.5	2329.6	533.7
RL	Girder	348.3	407.9	429.0	398.8	336.2	1920.1	429.0
	Slab	22.4	29.4	28.7	21.9	15.5	118.0	
	Railling	291.5	0.0	0.0	0.0	0.0	291.5	
	Total	662.1	437.4	457.7	420.7	351.7	2329.6	457.7
RR	Girder	526.6	498.2	447.6	353.9	227.9	2054.1	526.6
	Slab	28.9	30.8	28.3	20.2	10.6	118.7	
	Railling	0.0	0.0	0.0	0.0	156.7	156.7	
	Total	555.4	529.0	475.9	374.1	395.2	2329.6	529.0
2R	Girder	353.6	398.1	398.6	339.0	238.0	1727.3	398.6
	Slab	23.1	30.0	29.0	21.2	11.8	115.2	
	Railling	305.5	0.0	0.0	0.0	181.6	487.1	
	Total	682.2	428.2	427.6	360.3	431.4	2329.6	428.2

The effective section of a concrete slab for the interior girders continue to contribute about 5 % to 10 % of the total bending moment regardless of the presence of 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 2-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.4 for all the bridges with railing stiffness x1.

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) for Bridges with Railing Stiffness x1

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

Maximum bending moment (steel + slab) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	237.0	236.4	236.5	236.0
	8	224.8	304.9	294.3	305.0	294.5
	12	224.8	430.2	422.6	426.1	418.3
60	6	403.2	383.1	349.0	383.5	338.8
	8	403.2	483.8	429.4	478.0	421.8
	12	403.2	637.9	586.9	606.3	552.1
80	6	582.4	533.7	457.7	529.0	428.2
	8	582.4	662.4	555.4	641.4	528.6
	12	582.4	861.9	751.7	786.6	670.1
100	6	825.4	736.2	612.0	720.2	565.0
	8	825.4	907.2	741.6	860.3	681.3
	12	825.4	1176.6	986.9	1038.2	844.0
120	6	1163.5	1015.4	830.5	977.7	758.9
	8	1163.5	1248.5	1014.3	1158.8	900.2
	12	1163.5	1622.5	1322.8	1391.2	1098.4

(b) Distribution factor = DF = Mmax/Mo

DF=Mmax/M0 (steel + slab) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	1.05	1.05	1.05	1.05
	8	224.8	1.36	1.31	1.36	1.31
	12	224.8	1.91	1.88	1.90	1.86
60	6	403.2	0.95	0.87	0.95	0.84
	8	403.2	1.20	1.07	1.19	1.05
	12	403.2	1.58	1.46	1.50	1.37
80	6	582.4	0.92	0.79	0.91	0.74
	8	582.4	1.14	0.95	1.10	0.91
	12	582.4	1.48	1.29	1.35	1.15
100	6	825.4	0.89	0.74	0.87	0.68
	8	825.4	1.10	0.90	1.04	0.83
	12	825.4	1.43	1.20	1.26	1.02
120	6	1163.5	0.87	0.71	0.84	0.65
	8	1163.5	1.07	0.87	1.00	0.77
	12	1163.5	1.39	1.14	1.20	0.94

Table 4.4. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26 for Bridges with Railing Stiffness x1

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

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel + slab) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	AASHTO	No	RL	RR	2R
40	6	1.09	-3	-4	-3	-4
	8	1.46	-7	-10	-7	-10
	12	2.18	-12	-14	-13	-15
60	6	1.09	-13	-21	-13	-23
	8	1.46	-18	-27	-19	-28
	12	2.18	-27	-33	-31	-37
80	6	1.09	-16	-28	-17	-33
	8	1.46	-22	-35	-25	-38
	12	2.18	-32	-41	-38	-47
100	6	1.09	-18	-32	-20	-37
	8	1.46	-25	-38	-29	-43
	12	2.18	-35	-45	-42	-53
120	6	1.09	-20	-35	-23	-40
	8	1.46	-27	-40	-32	-47
	12	2.18	-36	-48	-45	-57

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

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel + slab) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	NCHRP	No	RL	RR	2R
40	6	1.20	-12	-12	-12	-13
	8	1.48	-8	-12	-8	-12
	12	1.98	-3	-5	-4	-6
60	6	1.08	-12	-20	-12	-22
	8	1.32	-9	-19	-10	-21
	12	1.77	-11	-18	-15	-23
80	6	1.01	-9	-22	-10	-27
	8	1.23	-8	-22	-10	-26
	12	1.64	-10	-21	-18	-30
100	6	0.95	-6	-22	-8	-28
	8	1.16	-5	-23	-10	-29
	12	1.54	-7	-22	-18	-34
120	6	0.91	-4	-22	-8	-28
	8	1.10	-2	-21	-9	-30
	12	1.47	-5	-23	-19	-36

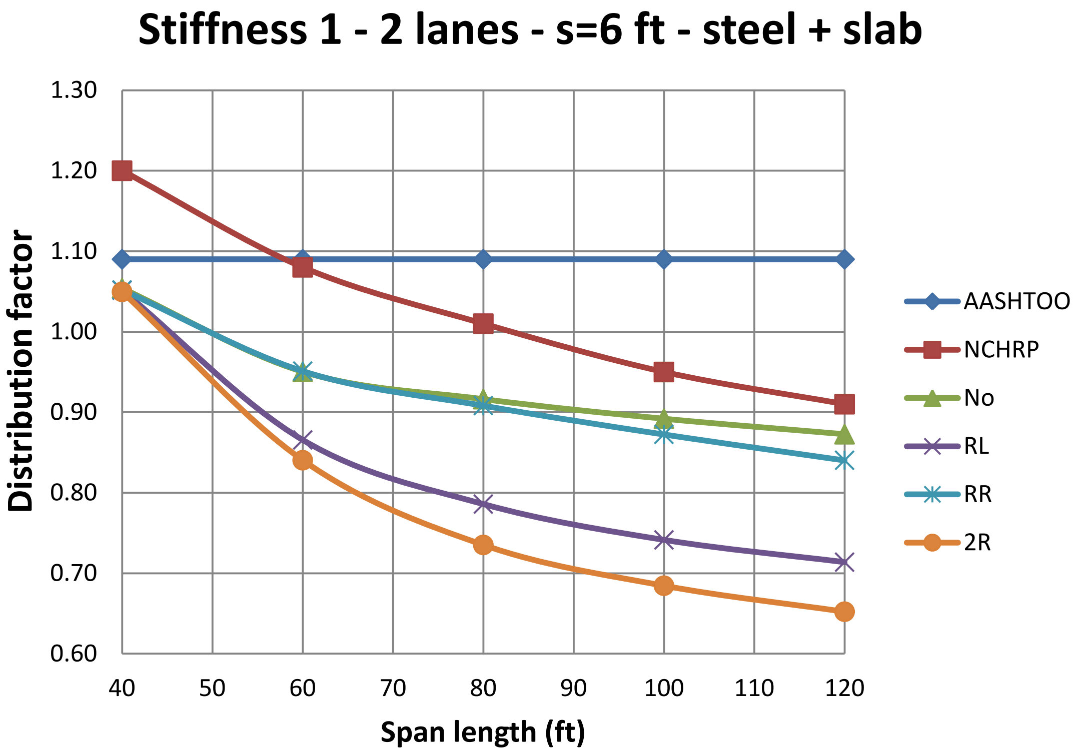


Figure 4.1. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

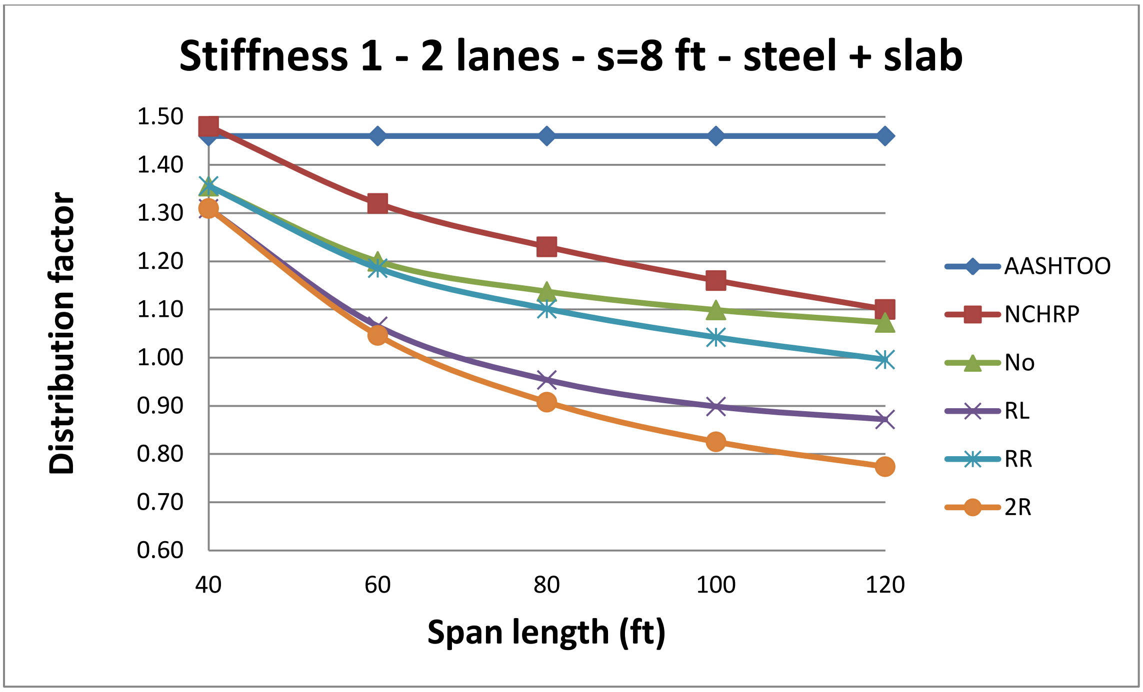


Figure 4.2. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

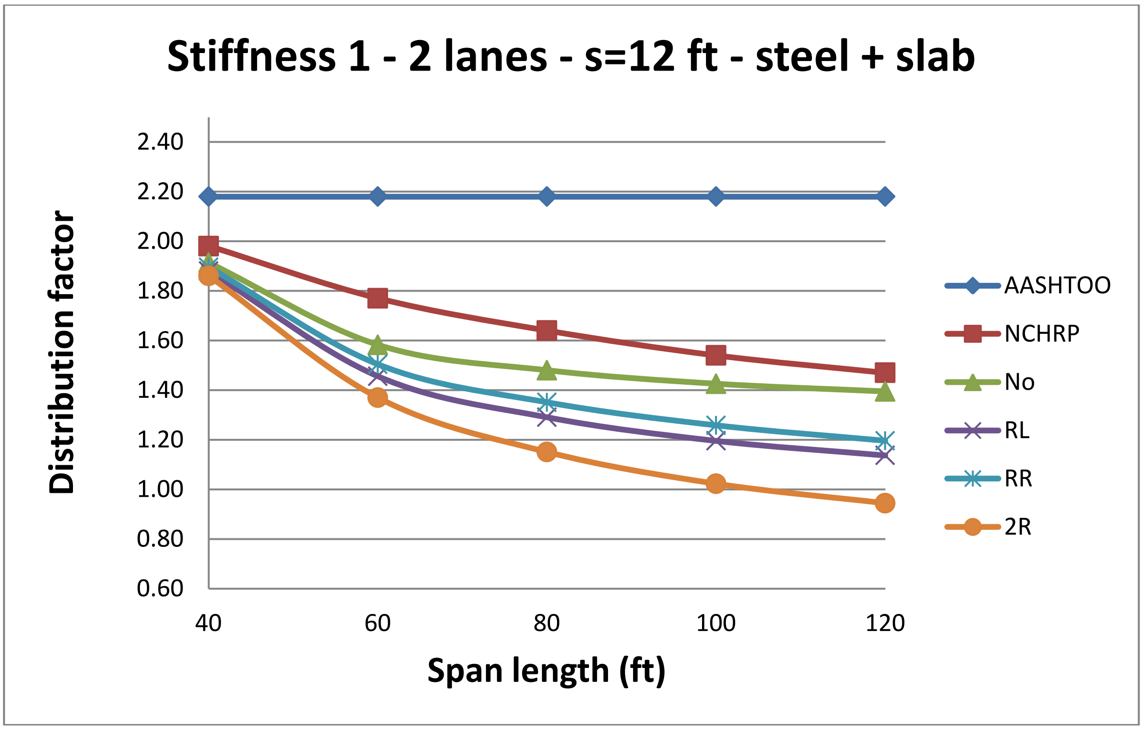


Figure 4.3. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

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 railings.

A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 60 2-lane bridges of x1 railing stiffness, 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 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.

Table 4.5. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel Only) for Bridges with Railing Stiffness x1

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

Maximum bending moment (steel only) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	220.0	219.1	219.5	218.6
	8	224.8	276.3	265.4	276.3	265.4
	12	224.8	370.9	362.9	366.6	358.5
60	6	403.2	358.3	324.8	358.3	314.3
	8	403.2	444.6	390.2	438.4	382.1
	12	403.2	559.6	509.1	546.4	474.8
80	6	582.4	514.8	429.0	526.6	398.6
	8	582.4	622.9	509.3	634.0	482.1
	12	582.4	795.0	661.1	797.8	581.9
100	6	825.4	729.8	578.2	742.6	530.2
	8	825.4	883.6	687.5	890.3	627.6
	12	825.4	1129.4	881.1	1113.4	743.5
120	6	1163.5	1016.9	789.8	1022.7	717.7
	8	1163.5	1233.3	948.7	1222.1	836.9
	12	1163.5	1580.8	1196.0	1521.4	981.2

(b) Distribution factor = DF = Mmax/Mo

DF=Mmax/M0 (steel only) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	0.98	0.97	0.98	0.97
	8	224.8	1.23	1.18	1.23	1.18
	12	224.8	1.65	1.61	1.63	1.59
60	6	403.2	0.89	0.81	0.89	0.78
	8	403.2	1.10	0.97	1.09	0.95
	12	403.2	1.39	1.26	1.36	1.18
80	6	582.4	0.88	0.74	0.90	0.68
	8	582.4	1.07	0.87	1.09	0.83
	12	582.4	1.37	1.14	1.37	1.00
100	6	825.4	0.88	0.70	0.90	0.64
	8	825.4	1.07	0.83	1.08	0.76
	12	825.4	1.37	1.07	1.35	0.90
120	6	1163.5	0.87	0.68	0.88	0.62
	8	1163.5	1.06	0.82	1.05	0.72
	12	1163.5	1.36	1.03	1.31	0.84

Table 4.6. Comparison of FEA Distribution Factors in All Steel Girders (Steel Only) with AASHTO (2002) and NCHRP 12-26 for Bridges with Railing Stiffness x1

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

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel only) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	AASHTO	No	RL	RR	2R
40	6	1.09	-10	-11	-10	-11
	8	1.46	-16	-19	-16	-19
	12	2.18	-24	-26	-25	-27
60	6	1.09	-18	-26	-18	-28
	8	1.46	-24	-34	-26	-35
	12	2.18	-36	-42	-38	-46
80	6	1.09	-19	-32	-17	-37
	8	1.46	-27	-40	-25	-43
	12	2.18	-37	-48	-37	-54
100	6	1.09	-19	-36	-17	-41
	8	1.46	-27	-43	-26	-48
	12	2.18	-37	-51	-38	-59
120	6	1.09	-20	-38	-19	-43
	8	1.46	-27	-44	-28	-51
	12	2.18	-38	-53	-40	-61

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

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel only) Stiffness 1 (2 lanes)						
L (ft)	s (ft)	NCHRP	No	RL	RR	2R
40	6	1.20	-18	-19	-19	-19
	8	1.48	-17	-20	-17	-20
	12	1.98	-17	-18	-18	-19
60	6	1.08	-18	-25	-18	-28
	8	1.32	-16	-27	-18	-28
	12	1.77	-22	-29	-23	-33
80	6	1.01	-12	-27	-10	-32
	8	1.23	-13	-29	-12	-33
	12	1.64	-17	-31	-16	-39
100	6	0.95	-7	-26	-5	-32
	8	1.16	-8	-28	-7	-34
	12	1.54	-11	-31	-12	-42
120	6	0.91	-4	-25	-3	-32
	8	1.10	-4	-26	-5	-35
	12	1.47	-8	-30	-11	-43

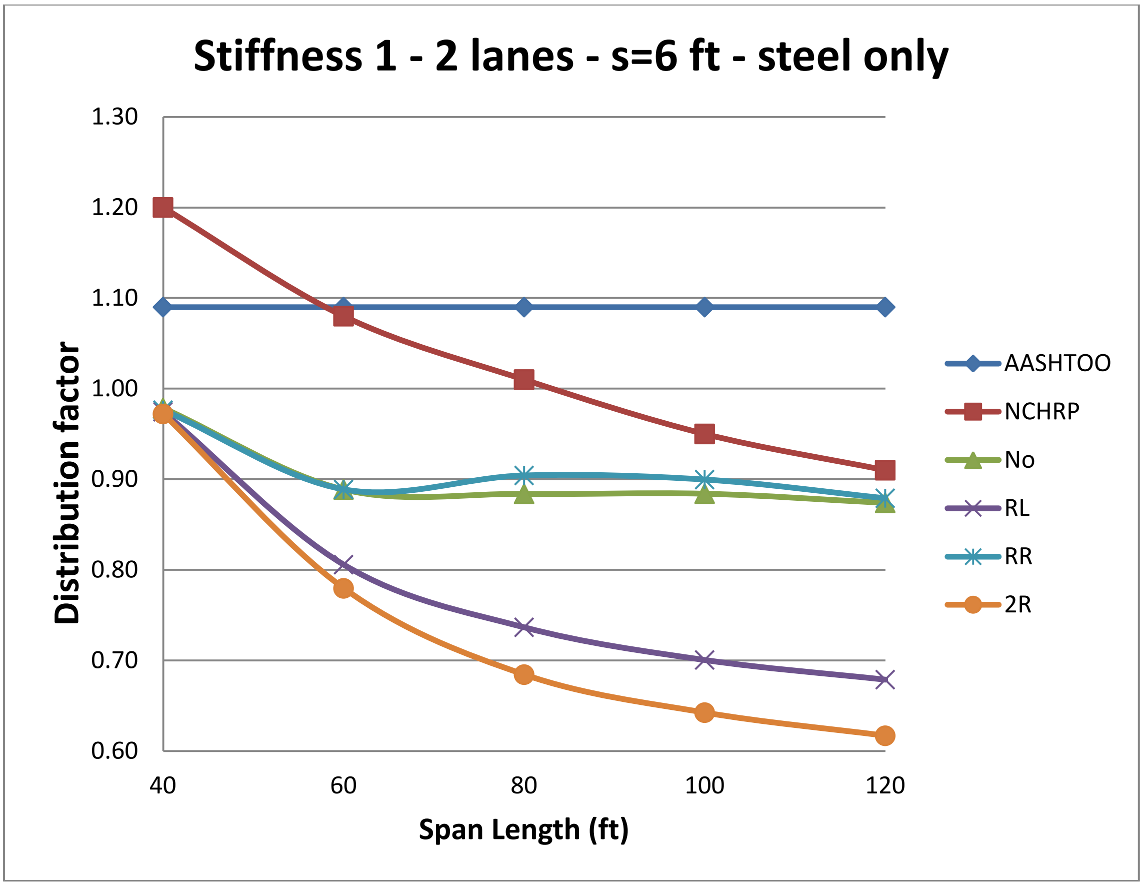


Figure 4.4. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6 ft)

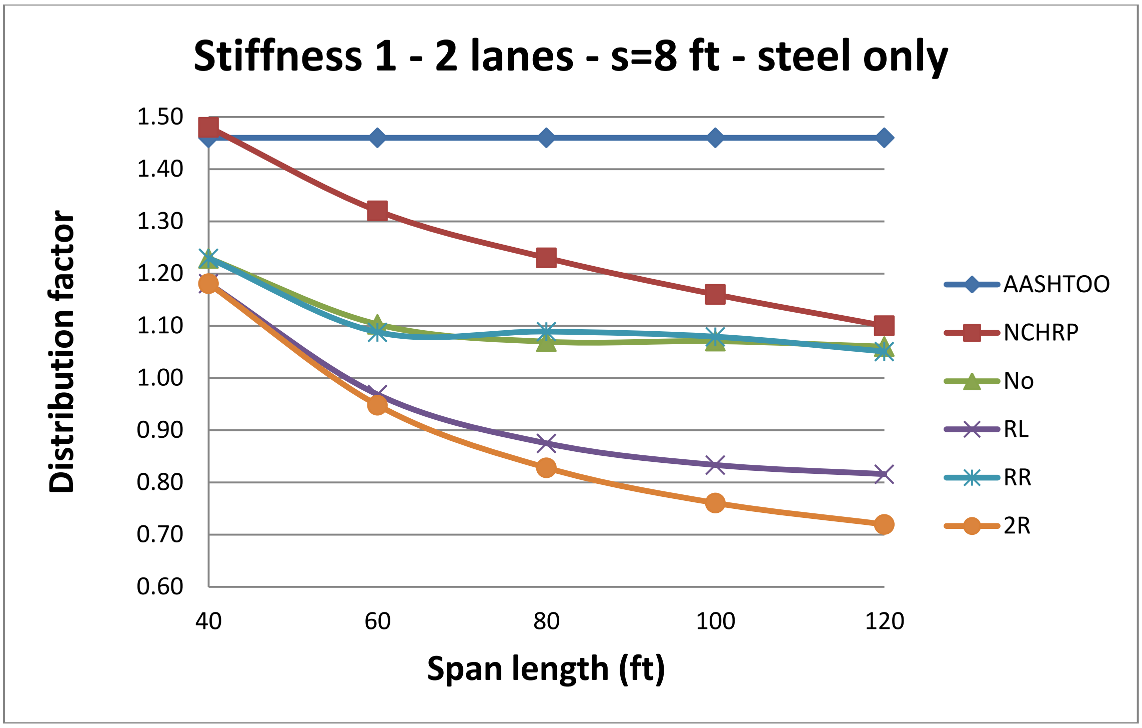


Figure 4.5. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8 ft)

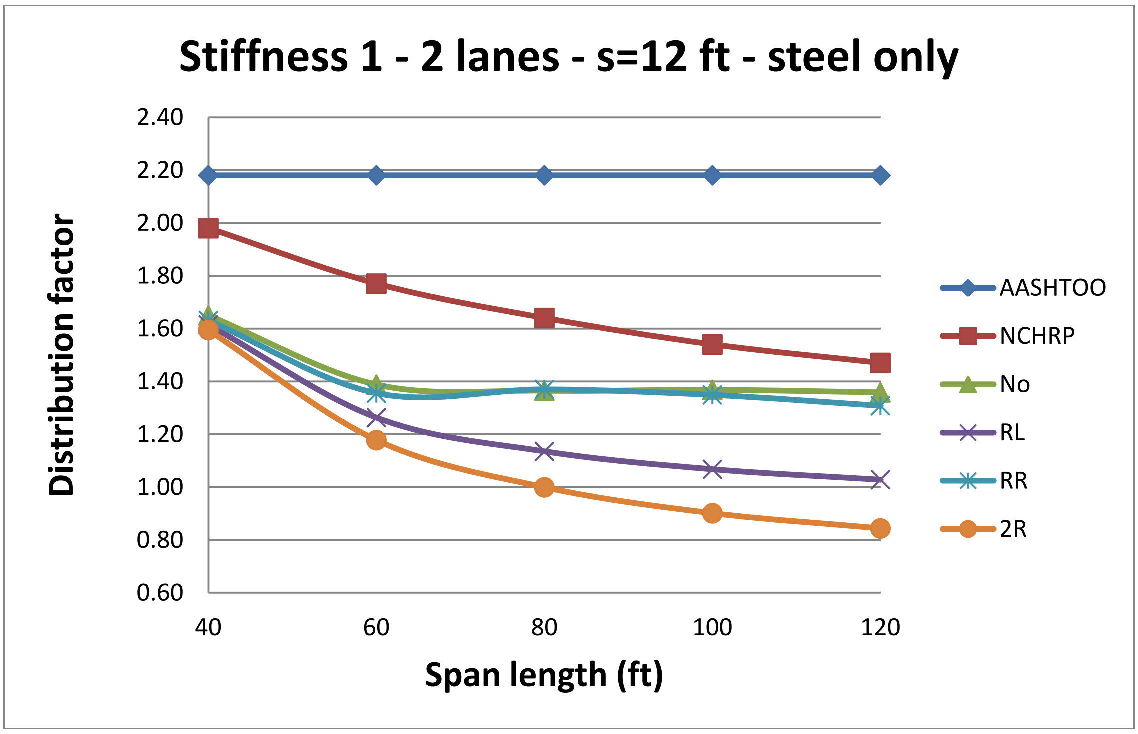


Figure 4.6. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12 ft)

After analyzing 2-lane bridges, 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 railings to a bridge deck:

1. No 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. Railing on one side (left or right): (1) is about 15 % higher than FEA distribution factors.
3. Railing on both sides: (1) is about 30 % higher than FEA distribution factors .

A conservative comparison of FEA distribution factors for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing railings to the bridge deck. First of all, it's important to note that for spans up to 40 ft, the addition of railings has a negligible effect on the distribution factor for all stiffness factors.

Otherwise, the average reductions in FEA distribution factors when introducing railing on one side were 6% for spans between 60 and 80 ft and 11% for spans between 80 and 120 ft compared to the base case (no railings).

The average reductions in FEA distribution factors when introducing railings on both sides were 16% for spans between 60 and 80 ft, and 25 % for spans between 80 and 120 ft compared to the base case (no railings).

The finite-element results show the effects of stiffness of 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 introducing railings in a bridge superstructure.

4.2.2. Three-Lane Bridges

Table A.4 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no railings on the bridge. 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

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 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 super-structure. The effective section of a concrete slab for the interior girders continue to contribute about 5 % to 12 % of the total bending moment regardless of the presence of 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 3-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.8 for all the bridges with railing stiffness x1.

Table 4.7. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab) for Bridges with Railing Stiffness x1

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

Maximum bending moment (steel + slab) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	268.7	268.9	268.6	268.8
	8	224.8	352.1	351.6	352.3	351.9
	12	224.8	492.2	493.1	486.7	487.6
60	6	403.2	440.9	424.0	441.6	423.9
	8	403.2	551.3	539.2	550.7	538.5
	12	403.2	766.9	764.6	729.2	720.7
80	6	582.4	597.5	547.2	601.0	544.8
	8	582.4	714.7	683.9	704.9	663.4
	12	582.4	1017.2	996.7	947.5	883.1
100	6	825.4	835.0	715.5	840.5	703.4
	8	825.4	941.6	882.6	912.7	825.7
	12	825.4	1368.5	1313.2	1248.2	1103.1
120	6	1163.5	1158.3	952.6	1161.5	922.8
	8	1163.5	1263.0	1161.5	1228.3	1053.0
	12	1163.5	1864.5	1750.5	1666.9	1414.8

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

DF=0.9*Mmax/M0 (steel + slab) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	1.08	1.08	1.08	1.08
	8	224.8	1.41	1.41	1.41	1.41
	12	224.8	1.97	1.97	1.95	1.95
60	6	403.2	0.98	0.95	0.99	0.95
	8	403.2	1.23	1.20	1.23	1.20
	12	403.2	1.71	1.71	1.63	1.61
80	6	582.4	0.92	0.85	0.93	0.84
	8	582.4	1.10	1.06	1.09	1.03
	12	582.4	1.57	1.54	1.46	1.36
100	6	825.4	0.91	0.78	0.92	0.77
	8	825.4	1.03	0.96	1.00	0.90
	12	825.4	1.49	1.43	1.36	1.20
120	6	1163.5	0.90	0.74	0.90	0.71
	8	1163.5	0.98	0.90	0.95	0.81
	12	1163.5	1.44	1.35	1.29	1.09

Table 4.8. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26 for Bridges with Railing Stiffness x1

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

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel + slab) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	AASHTO	No	RL	RR	2R
40	6	1.09	-1	-1	-1	-1
	8	1.46	-3	-4	-3	-4
	12	2.18	-10	-9	-11	-10
60	6	1.09	-10	-13	-10	-13
	8	1.46	-16	-18	-16	-18
	12	2.18	-21	-22	-25	-26
80	6	1.09	-15	-22	-15	-23
	8	1.46	-24	-28	-25	-30
	12	2.18	-28	-29	-33	-37
100	6	1.09	-16	-28	-16	-30
	8	1.46	-30	-34	-32	-38
	12	2.18	-32	-34	-38	-45
120	6	1.09	-18	-32	-18	-35
	8	1.46	-33	-38	-35	-44
	12	2.18	-34	-38	-41	-50

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

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel + slab) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	NCHRP	No	RL	RR	2R
40	6	1.20	-10	-10	-10	-10
	8	1.48	-5	-5	-5	-5
	12	1.98	0	0	-2	-1
60	6	1.08	-9	-12	-9	-12
	8	1.32	-7	-9	-7	-9
	12	1.77	-3	-4	-8	-9
80	6	1.01	-9	-16	-8	-17
	8	1.23	-10	-14	-11	-17
	12	1.64	-4	-6	-11	-17
100	6	0.95	-4	-18	-4	-19
	8	1.16	-11	-17	-14	-22
	12	1.54	-3	-7	-12	-22
120	6	0.91	-2	-19	-1	-22
	8	1.10	-11	-18	-14	-26
	12	1.47	-2	-8	-12	-26

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 3-lane bridges with railing stiffness x1, 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 and the 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.

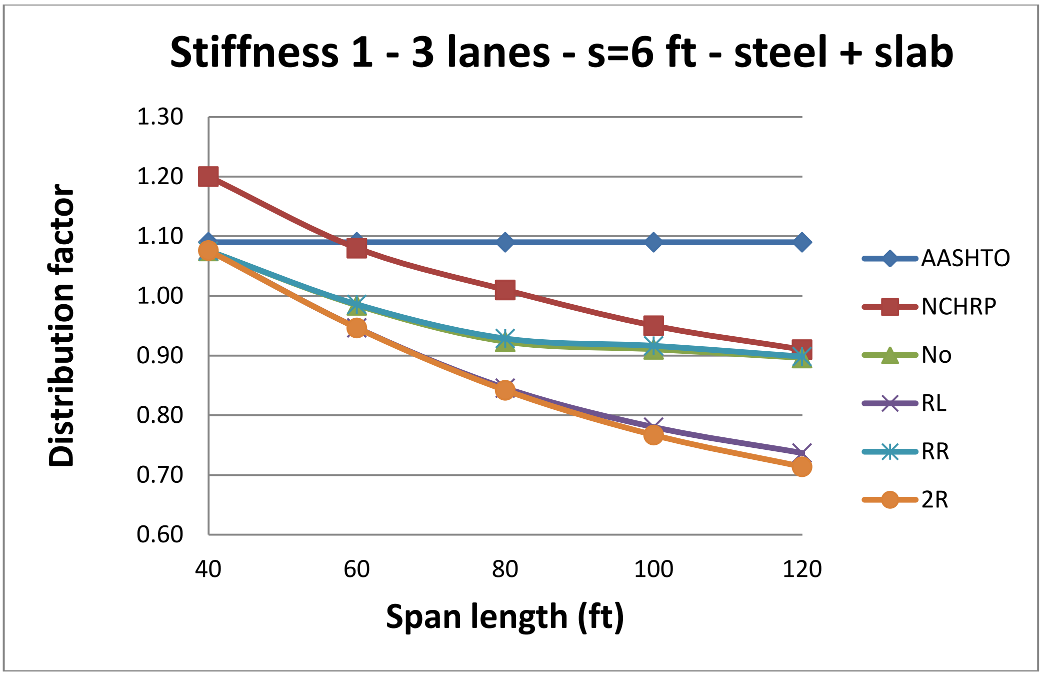


Figure 4.7. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

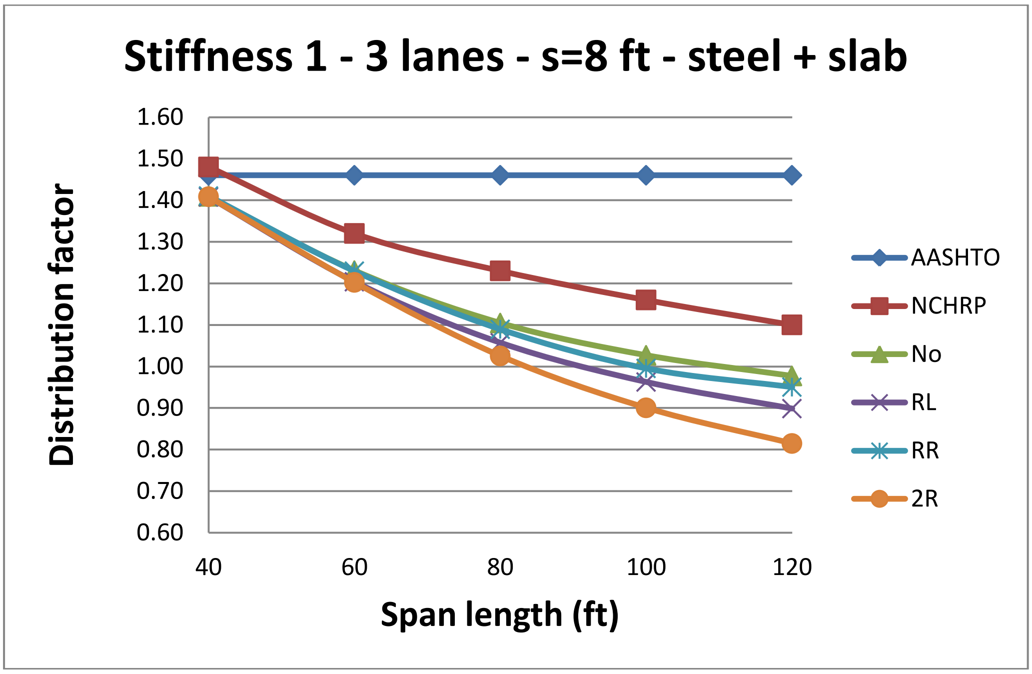


Figure 4.8. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

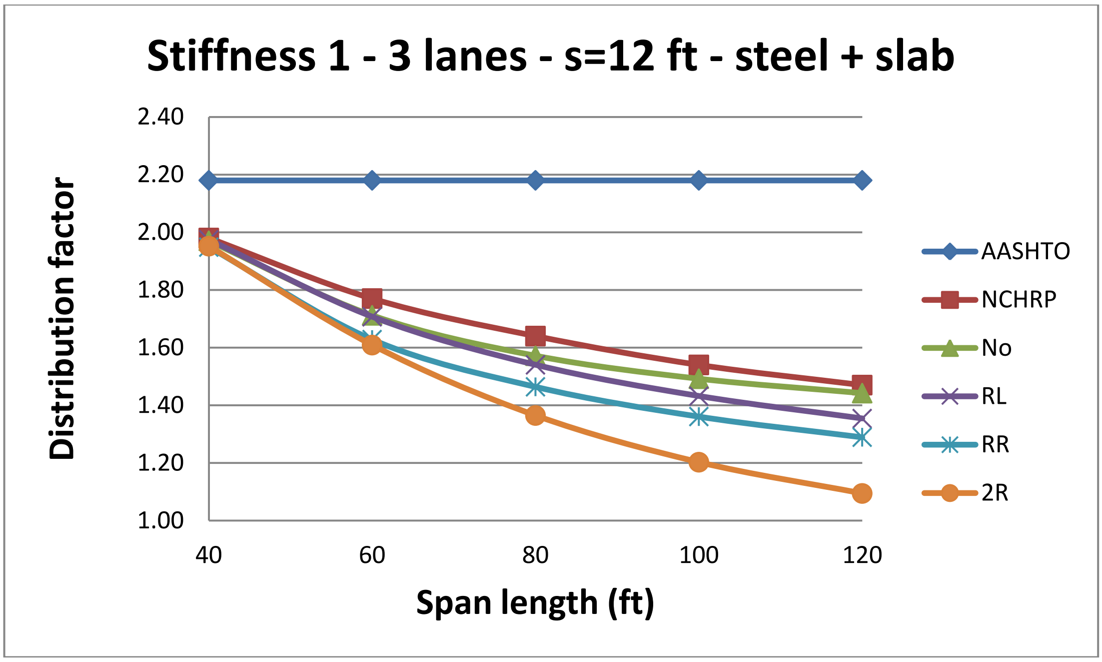


Figure 4.9. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

Table 4.9. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel girders (Steel Only) for Bridges with Railing Stiffness x1

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

Maximum bending moment (steel only) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	251.5	251.4	251.5	251.3
	8	224.8	321.9	321.3	322.1	321.5
	12	224.8	429.6	430.3	424.0	424.7
60	6	403.2	414.9	396.9	415.4	396.8
	8	403.2	506.0	493.4	505.0	492.3
	12	403.2	677.4	674.2	641.6	630.5
80	6	582.4	564.6	513.3	568.0	510.4
	8	582.4	658.5	624.0	648.0	605.9
	12	582.4	907.2	885.7	840.0	774.5
100	6	825.4	805.5	675.0	822.5	662.0
	8	825.4	874.4	811.9	844.6	757.8
	12	825.4	1235.4	1179.8	1119.3	975.8
120	6	1163.5	1147.3	903.5	1172.9	872.8
	8	1163.5	1181.0	1077.0	1150.5	972.4
	12	1163.5	1699.6	1667.6	1509.6	1264.2

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

DF=0.9*Mmax/M0 (steel only) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	1.01	1.01	1.01	1.01
	8	224.8	1.29	1.29	1.29	1.29
	12	224.8	1.72	1.72	1.70	1.70
60	6	403.2	0.93	0.89	0.93	0.89
	8	403.2	1.13	1.10	1.13	1.10
	12	403.2	1.51	1.50	1.43	1.41
80	6	582.4	0.87	0.79	0.88	0.79
	8	582.4	1.02	0.96	1.00	0.94
	12	582.4	1.40	1.37	1.30	1.20
100	6	825.4	0.88	0.74	0.90	0.72
	8	825.4	0.95	0.89	0.92	0.83
	12	825.4	1.35	1.29	1.22	1.06
120	6	1163.5	0.89	0.70	0.91	0.68
	8	1163.5	0.91	0.83	0.89	0.75
	12	1163.5	1.31	1.29	1.17	0.98

Table 4.10. Comparison of FEA Distribution Factors in All Steel girders (Steel Only) with AASHTO (2002) and NCHRP 12-26 for Bridges with Railing Stiffness x1

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

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel only) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	AASHTOO	No	RL	RR	2R
40	6	1.09	-8	-8	-8	-8
	8	1.46	-12	-12	-12	-12
	12	2.18	-21	-21	-22	-22
60	6	1.09	-15	-19	-15	-19
	8	1.46	-23	-25	-23	-25
	12	2.18	-31	-31	-34	-35
80	6	1.09	-20	-27	-19	-28
	8	1.46	-30	-34	-31	-36
	12	2.18	-36	-37	-40	-45
100	6	1.09	-19	-32	-18	-34
	8	1.46	-35	-39	-37	-43
	12	2.18	-38	-41	-44	-51
120	6	1.09	-19	-36	-17	-38
	8	1.46	-37	-43	-39	-48
	12	2.18	-40	-41	-46	-55

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

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel only) Stiffness 1 (3 lanes)						
L (ft)	s (ft)	NCHRP	No	RL	RR	2R
40	6	1.20	-16	-16	-16	-16
	8	1.48	-13	-13	-13	-13
	12	1.98	-13	-13	-14	-14
60	6	1.08	-14	-18	-14	-18
	8	1.32	-14	-17	-15	-17
	12	1.77	-15	-15	-19	-20
80	6	1.01	-14	-21	-13	-22
	8	1.23	-17	-22	-19	-24
	12	1.64	-15	-17	-21	-27
100	6	0.95	-8	-23	-6	-24
	8	1.16	-18	-24	-21	-29
	12	1.54	-13	-16	-21	-31
120	6	0.91	-2	-23	0	-26
	8	1.10	-17	-24	-19	-32
	12	1.47	-11	-12	-21	-33

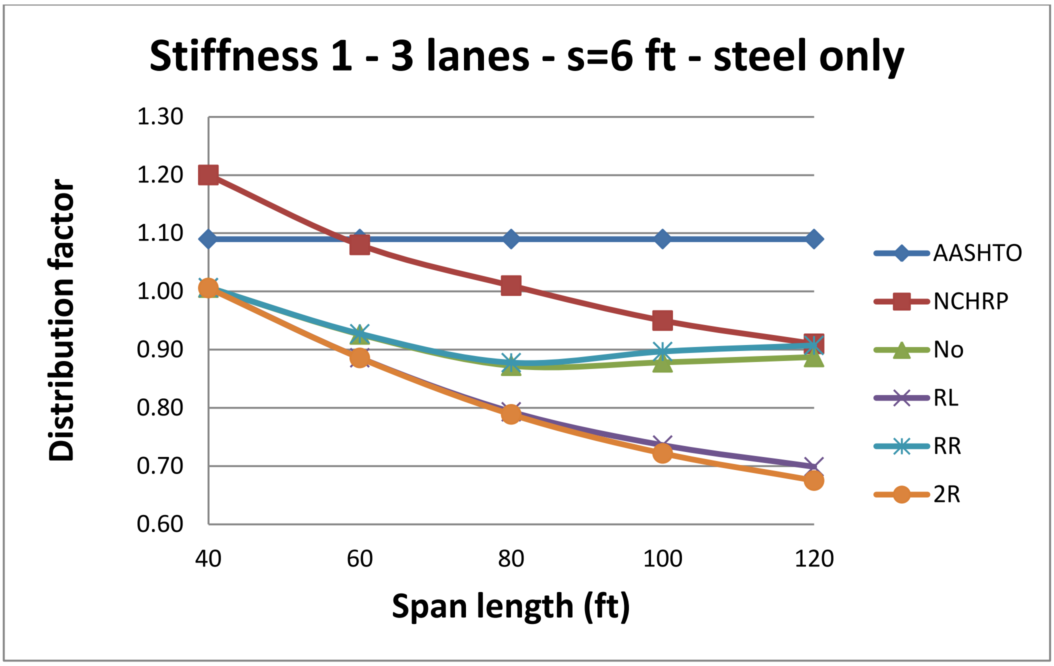


Figure 4.10. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6 ft)

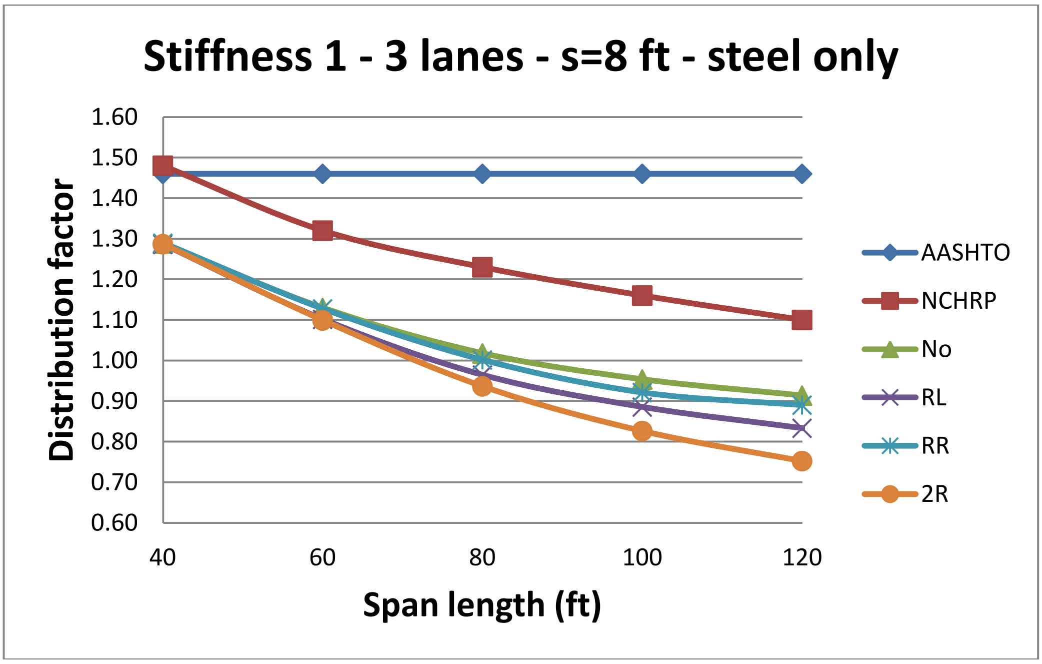


Figure 4.11. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8 ft)

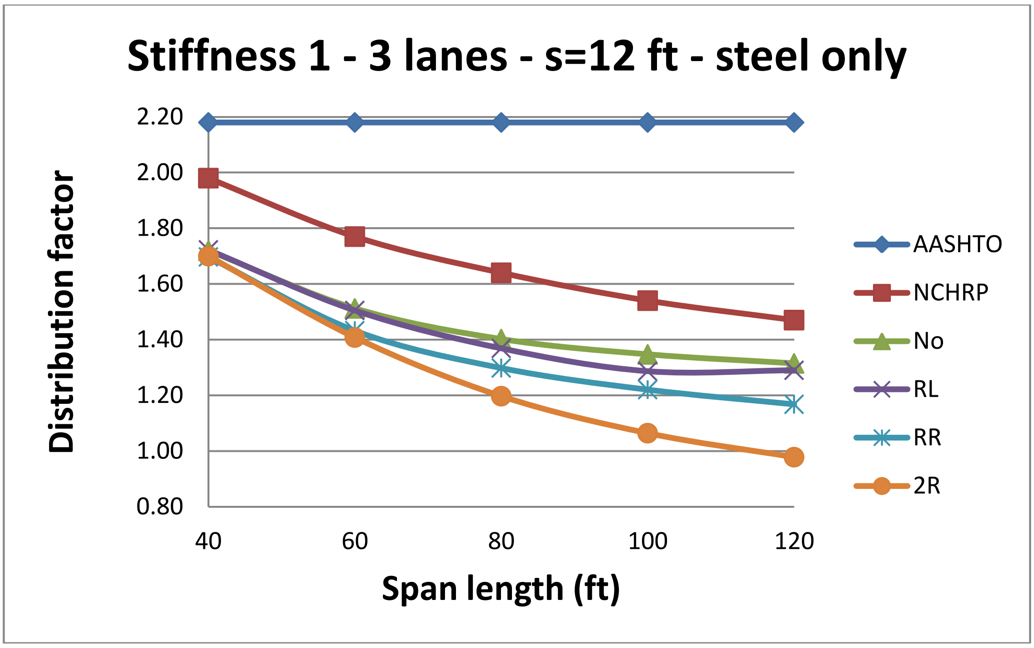


Figure 4.12. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12 ft)

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 railings to a bridge deck:

1. No railings: the FEA distribution factors are smaller than (1) by about 6 % for all spans.
2. Railing on one side (left or right): For spans up to 80ft (1) is about 7 % higher than FEA distribution factors for spans between 80 ft and 120 ft.
3. Railing on both sides: For spans between 40 ft and 80 ft (1) is higher than FEA distribution factors by 10 % and for spans between 80 ft and 120 ft (1) is higher than FEA distribution factors 25 % .

Considering the various bridge geometries, a conservative comparison of FEA distribution factors for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing railings to the bridge deck. First of all, it is important to note that for spans up to 40 ft, the addition or railings has a negligible effect on the distribution factor. Otherwise, the average reductions in FEA distribution factors when compared to the base case were : 2 % for spans between 40 and 80 ft (12 and 18 m), 4 % for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on one side, and 3 % for spans between 40 and 80 ft (12 and 24 m) and about 15% for spans between 80 and 120 ft (24 and 36 m) when introducing railings on both sides. 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 introducing railings in a bridge superstructure.

4.2.3. Four-Lane Bridges

Table A.7 shows that the contribution of bending moment from the concrete slab is about 5 % when there is no railing on the bridge. When 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.

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 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 continue to contribute about 5 % to 12 % of the total bending moment regardless of the presence of 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 4-lane bridges. A summary of the percent decrease in wheel load distribution factors is reported in Table 4.12 for all the bridges of railing stiffness x1.

Table 4.11. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab) for Bridges with Railing Stiffness x1

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

Maximum bending moment (steel + slab) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	278.2	280.2	278.2	280.1
	8	224.8	363.6	364.8	363.6	364.7
	12	224.8	538.2	540.5	538.2	540.5
60	6	403.2	475.7	473.6	476.0	473.9
	8	403.2	612.7	604.6	613.4	604.4
	12	403.2	873.2	869.9	871.2	867.8
80	6	582.4	644.6	624.1	645.1	624.5
	8	582.4	830.2	785.7	830.9	783.4
	12	582.4	1167.5	1106.8	1168.8	1088.6
100	6	825.4	885.0	812.4	887.7	808.6
	8	825.4	1132.1	1014.5	1138.9	1007.7
	12	825.4	1583.8	1408.6	1577.4	1351.7
120	6	1163.5	1242.5	1066.1	1254.9	1051.2
	8	1163.5	1574.5	1329.6	1582.7	1305.2
	12	1163.5	2156.7	1822.6	2129.6	1721.4

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

DF=0.75*Mmax/M0 (steel + slab) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	0.93	0.93	0.93	0.93
	8	224.8	1.21	1.22	1.21	1.22
	12	224.8	1.80	1.80	1.80	1.80
60	6	403.2	0.88	0.88	0.89	0.88
	8	403.2	1.14	1.12	1.14	1.12
	12	403.2	1.62	1.62	1.62	1.61
80	6	582.4	0.83	0.80	0.83	0.80
	8	582.4	1.07	1.01	1.07	1.01
	12	582.4	1.50	1.43	1.51	1.40
100	6	825.4	0.80	0.74	0.81	0.73
	8	825.4	1.03	0.92	1.03	0.92
	12	825.4	1.44	1.28	1.43	1.23
120	6	1163.5	0.80	0.69	0.81	0.68
	8	1163.5	1.01	0.86	1.02	0.84
	12	1163.5	1.39	1.17	1.37	1.11

Table 4.12. Comparison of FEA Distribution Factors in Interior Girders (Steel + Slab) with AASHTO (2002) and NCHRP 12-26 for Bridges with Railing Stiffness x1

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

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel + slab) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	AASHTO	No	RL	RR	2R
40	6	1.09	-15	-14	-15	-14
	8	1.46	-17	-17	-17	-17
	12	2.18	-18	-17	-18	-17
60	6	1.09	-19	-19	-19	-19
	8	1.46	-22	-23	-22	-23
	12	2.18	-25	-26	-26	-26
80	6	1.09	-24	-26	-24	-26
	8	1.46	-27	-31	-27	-31
	12	2.18	-31	-35	-31	-36
100	6	1.09	-26	-32	-26	-33
	8	1.46	-30	-37	-29	-37
	12	2.18	-34	-41	-34	-44
120	6	1.09	-27	-37	-26	-38
	8	1.46	-30	-41	-30	-42
	12	2.18	-36	-46	-37	-49

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

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel + slab) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	NCHRP	No	RL	RR	2R
40	6	1.20	-23	-22	-23	-22
	8	1.48	-18	-18	-18	-18
	12	1.98	-9	-9	-9	-9
60	6	1.08	-18	-18	-18	-18
	8	1.32	-14	-15	-14	-15
	12	1.77	-8	-9	-8	-9
80	6	1.01	-18	-20	-18	-20
	8	1.23	-13	-18	-13	-18
	12	1.64	-8	-13	-8	-15
100	6	0.95	-15	-22	-15	-23
	8	1.16	-11	-21	-11	-21
	12	1.54	-7	-17	-7	-20
120	6	0.91	-12	-24	-11	-26
	8	1.10	-8	-22	-7	-24
	12	1.47	-5	-20	-7	-25

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 railings. A summary of the FEA maximum bending moments and their corresponding wheel load distribution factors in the 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 and the railings. 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.

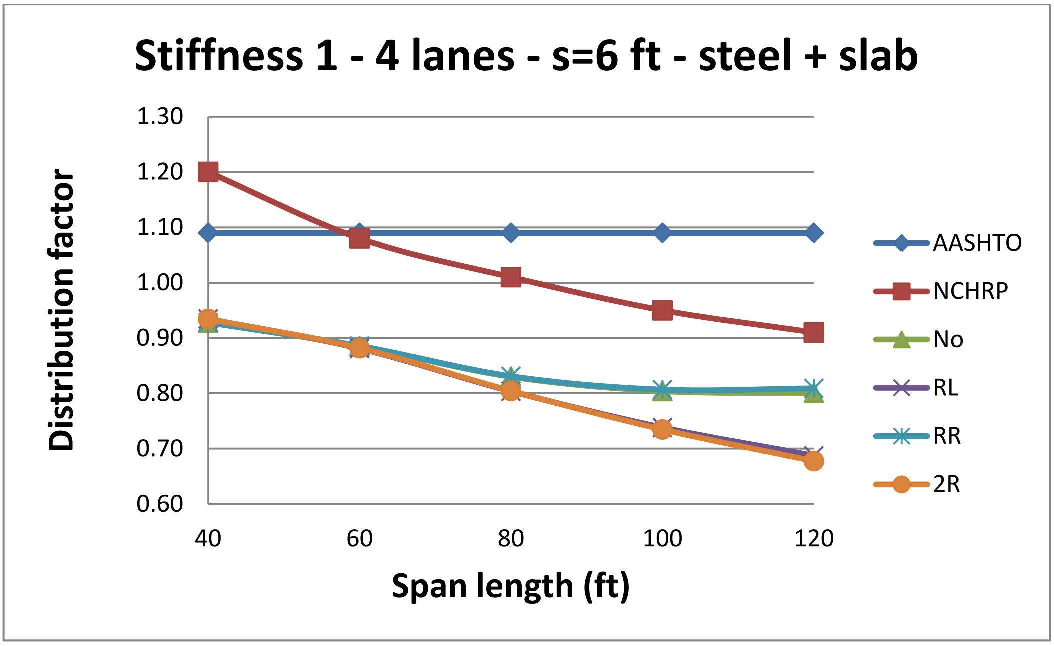


Figure 4.13. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 6 ft)

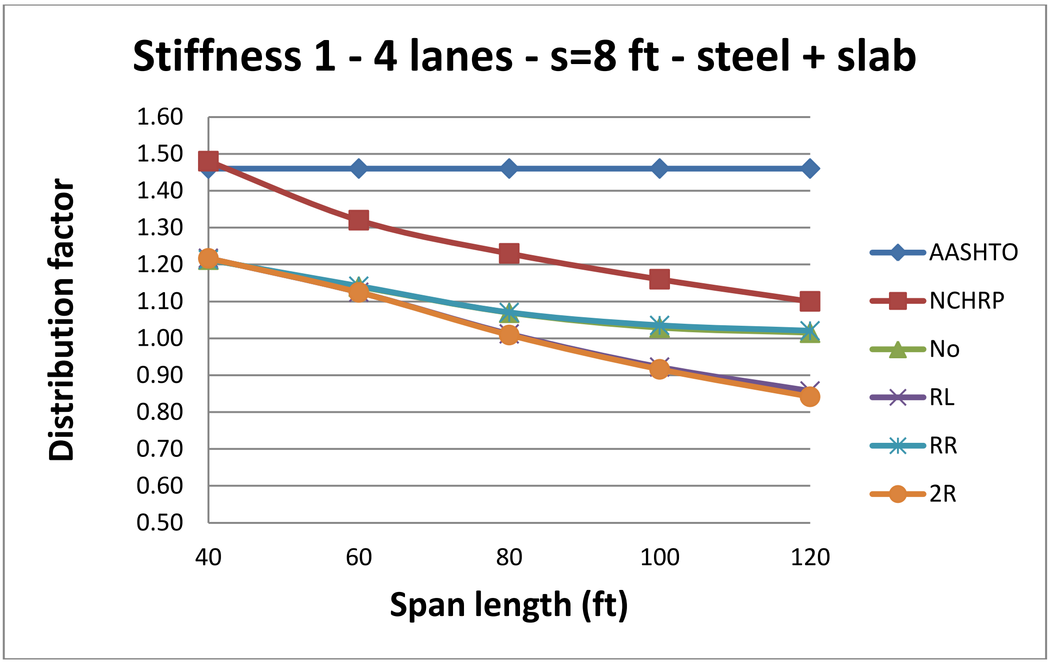


Figure 4.14. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 8 ft)

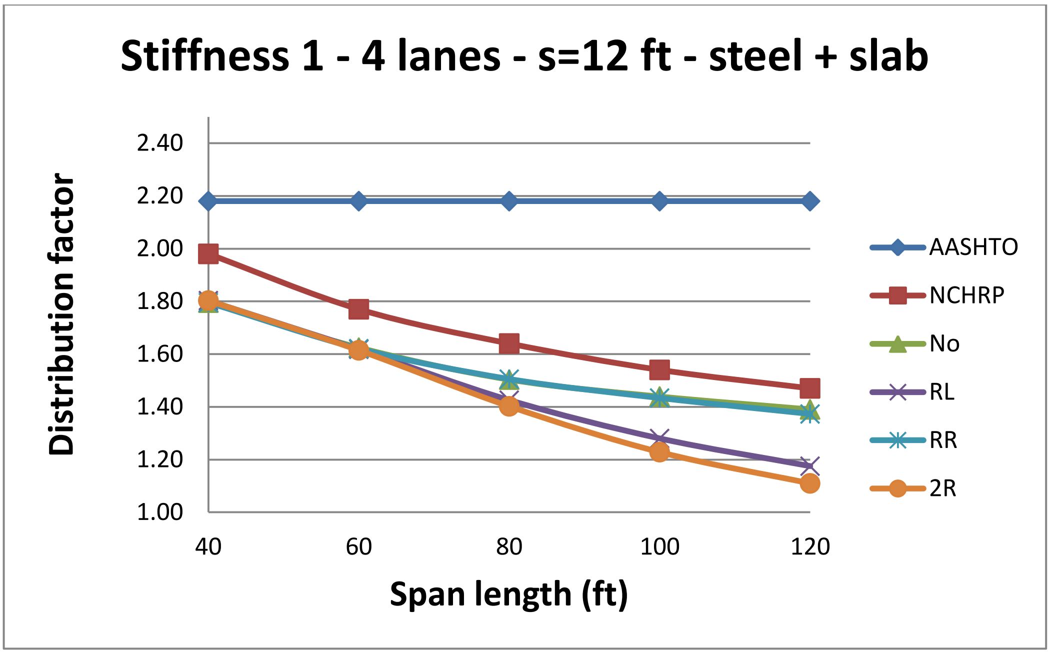


Figure 4.15. Sensitivity of Distribution Factor to Span Length for Interior Girders (Slab + Steel, Spacing = 12 ft)

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

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

Maximum bending moment (steel only) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	261.4	263.1	261.4	263.1
	8	224.8	330.4	331.4	330.3	331.2
	12	224.8	467.2	469.2	467.2	469.2
60	6	403.2	449.0	446.0	449.3	446.2
	8	403.2	563.5	554.0	564.1	553.6
	12	403.2	769.9	764.9	767.5	762.4
80	6	582.4	609.5	587.4	610.7	587.6
	8	582.4	767.5	721.0	767.6	718.5
	12	582.4	1047.5	975.9	1047.8	956.8
100	6	825.4	842.7	766.9	845.5	762.4
	8	825.4	1063.2	936.5	1069.2	928.5
	12	825.4	1434.9	1251.5	1426.9	1194.0
120	6	1163.5	1206.2	1009.9	1228.1	996.3
	8	1163.5	1504.4	1234.5	1533.9	1208.4
	12	1163.5	2006.6	1632.9	2040.9	1545.6

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

DF=0.75*Mmax/M0 (steel only) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	M0 (kip-ft)	No	RL	RR	2R
40	6	224.8	0.87	0.88	0.87	0.88
	8	224.8	1.10	1.11	1.10	1.10
	12	224.8	1.56	1.57	1.56	1.57
60	6	403.2	0.84	0.83	0.84	0.83
	8	403.2	1.05	1.03	1.05	1.03
	12	403.2	1.43	1.42	1.43	1.42
80	6	582.4	0.78	0.76	0.79	0.76
	8	582.4	0.99	0.93	0.99	0.93
	12	582.4	1.35	1.26	1.35	1.23
100	6	825.4	0.77	0.70	0.77	0.69
	8	825.4	0.97	0.85	0.97	0.84
	12	825.4	1.30	1.14	1.30	1.08
120	6	1163.5	0.78	0.65	0.79	0.64
	8	1163.5	0.97	0.80	0.99	0.78
	12	1163.5	1.29	1.05	1.32	1.00

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

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel only) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	AASHTO	No	RL	RR	2R
40	6	1.09	-20	-19	-20	-19
	8	1.46	-25	-24	-25	-24
	12	2.18	-29	-28	-29	-28
60	6	1.09	-23	-24	-23	-24
	8	1.46	-28	-29	-28	-29
	12	2.18	-34	-35	-35	-35
80	6	1.09	-28	-31	-28	-31
	8	1.46	-32	-36	-32	-37
	12	2.18	-38	-42	-38	-43
100	6	1.09	-30	-36	-30	-36
	8	1.46	-34	-42	-33	-42
	12	2.18	-40	-48	-41	-50
120	6	1.09	-29	-40	-27	-41
	8	1.46	-34	-45	-32	-47
	12	2.18	-41	-52	-40	-54

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

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel only) Stiffness 1 (4 lanes)						
L (ft)	s (ft)	NCHRP	No	RL	RR	2R
40	6	1.20	-27	-27	-27	-27
	8	1.48	-26	-25	-26	-25
	12	1.98	-21	-21	-21	-21
60	6	1.08	-23	-23	-23	-23
	8	1.32	-21	-22	-20	-22
	12	1.77	-19	-20	-19	-20
80	6	1.01	-22	-25	-22	-25
	8	1.23	-20	-25	-20	-25
	12	1.64	-18	-23	-18	-25
100	6	0.95	-19	-27	-19	-27
	8	1.16	-17	-27	-16	-27
	12	1.54	-15	-26	-16	-30
120	6	0.91	-15	-28	-13	-29
	8	1.10	-12	-28	-10	-29
	12	1.47	-12	-28	-11	-32

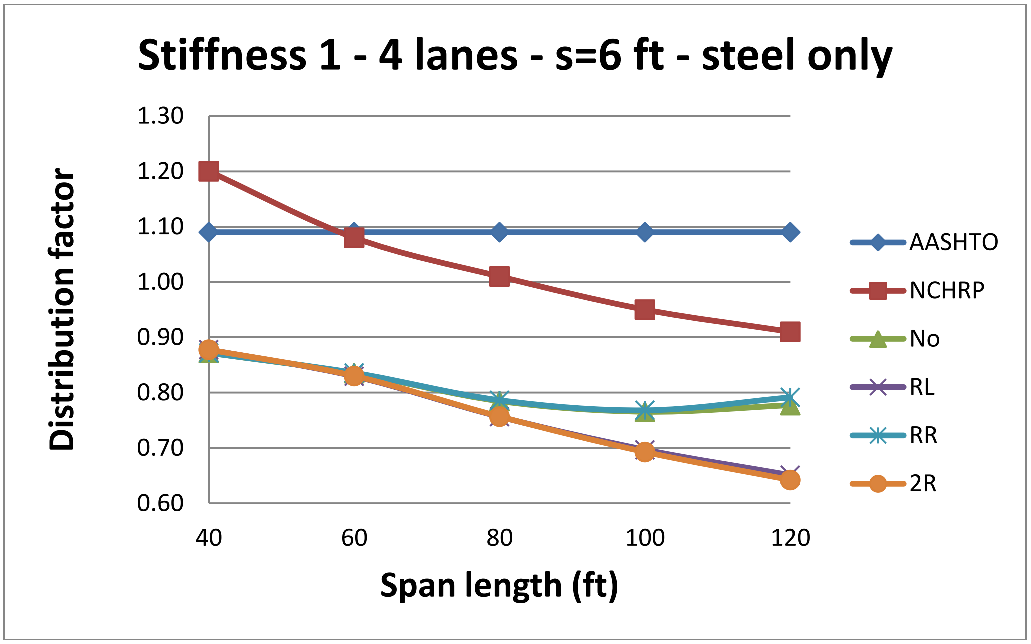


Figure 4.16. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 6ft)

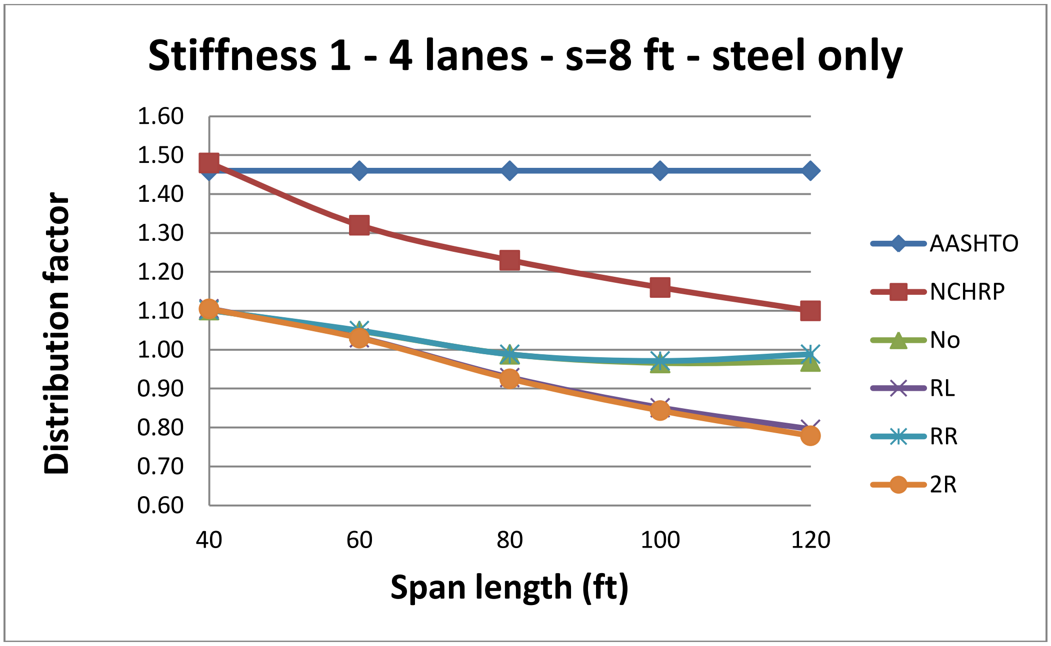


Figure 4.17. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 8ft)

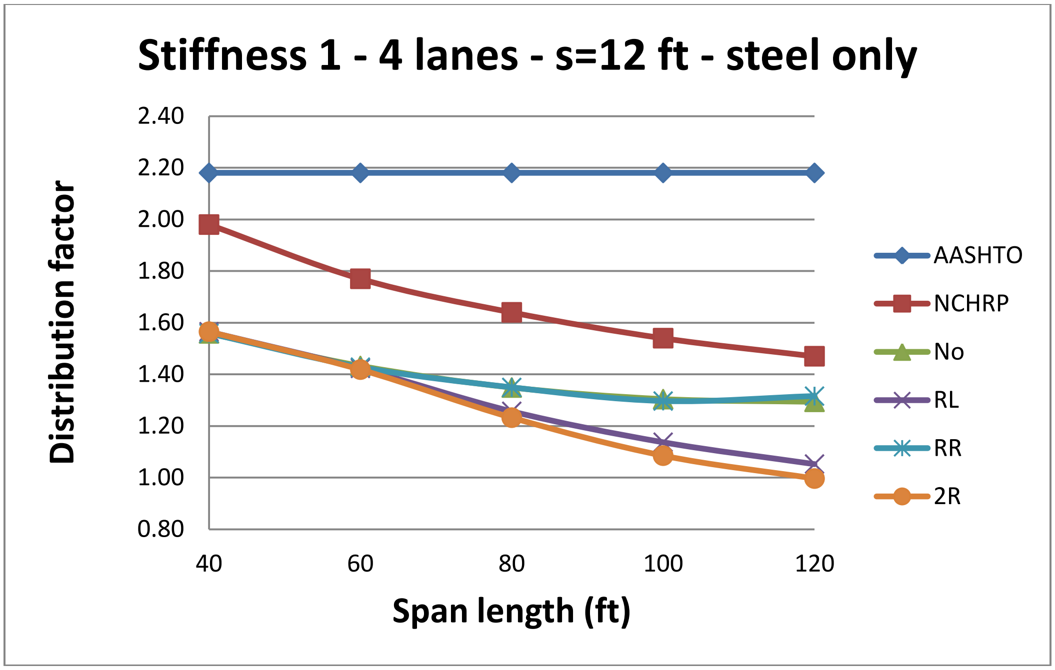


Figure 4.18. Sensitivity of Distribution Factor to Span Length for Steel Girders (Exterior and Interior, Steel Only, Spacing = 12ft)

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 4.14 (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 railings to a bridge deck:

1. No Railings: the FEA distribution factors are smaller than (1) by about 12 % for all spans.
2. Railing on one side (left or right):For spans up to 80 ft (1) is higher than FEA distribution factors by and about 16 % and for spans between 80 and 120 ft it is higher by about 12 %.

3. Railing on both sides: For spans up to 80 ft (1) is higher than FEA distribution factors by about 16 % and for spans between 80 and 120 ft it is higher by about 25% .

A conservative comparison of FEA distribution factors for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing railings to the bridge deck. First of all, it's important to note that for spans up to 60 ft, the addition of railings has a negligible effect on the distribution factor .Otherwise, the average reductions in FEA distribution factors when compared to the base case were; 0 % when introducing railings on one side; 4% for spans between 60 and 80 ft (18 and 24 m) and 10% for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on both sides. The finite-element results show the effects of stiffness of 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 introducing railings in a bridge superstructure.

4.3. Influence of Railing Stiffness on One-Span Bridges

We have mentioned in section 4.2 the influence of the presence of railings on steel girder bridges. And the results were presented in details for bridges with railings of stiffness x1. In this section , we will show ; the influence of stiffness of railings on the load carrying capacity and deflections of steel girder bridges. This is done by computing the FEA maximum bending moment in the critical girder and the deflection in each bridge case , while varying the stiffness of railings. In addition , a small section covering the influence of stiffness of railings while varying the stiffness of girders will be added.

4.3.1. Influence on Maximum Bending Moments

In this section we will be showing the values of the maximum bending moments in the critical girder for each bridge case (total of 720 bridge cases), while varying the stiffness of the railing(s). Then we can compute the FEA distribution factor for each case, and compare it with the distribution factors of AASHTO standard specifications and NCHRP.

4.3.1.1.<u>Two-lane bridges</u>

Tables 4.15 through 4.18 show the results for 240 2-lane bridges. Tables 4.15 and 4.16 show results while taking into consideration the contribution of the slab , which means that the critical girder is an interior girder. Whereas ; Tables 4.17 and 4.18 show results without taking into consideration the contribution of the slab , which means that the critical girder is either an interior or an exterior girder . Tables 4.15 and 4.17 show the maximum FEA bending moment in the critical girder , and the distribution factors , and Tables 4.16 and 4.18 show the % decrease of distribution factor compared to AASHTO and NCHRP and with respect to the FEA distribution factors of the reference case (No railings).

From the tables we can notice the decrease in the maximum bending moment when we increase the stiffness of railings , whether we have one railing or 2 railings. The effect increases with the increase in span length , girder spacing and when 2 railings are introduced [These differences can be seen clearly from tables 4.15 and 4.17].

Figures 4.19 and 4.20 show ; in chart form ; the sensitivity of distribution factor to railing stiffness for some bridge cases (arbitrarily chosen) [For a complete set of charts for two-lane bridges refer to the appendix A.10 to A.24].

Table 4.111. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Steel Girders (Steel + Slab) [2-lane Bridges]

a. <u>Maximum bending moment (kip .ft)</u>

Maximum bending moment Mmax (kip.ft) (steel + slab) (2 lanes)																		
L (ft)	s (ft)	Mo (kip-ft)	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	224.8	237.0	236.5	236.4	236.4	236.3	236.3	236.5	236.5	236.5	236.5	236.5	236.1	236.0	235.9	235.8	235.8
	8	224.8	304.9	296.8	294.3	292.3	291.5	291.0	304.9	305.0	305.1	305.2	305.2	296.8	294.5	292.5	291.7	291.2
	12	224.8	430.2	424.3	422.6	421.3	420.7	420.4	426.8	426.1	425.5	425.2	425.1	420.8	418.3	416.3	415.4	414.9
60	6	403.2	383.1	354.6	349.0	343.4	340.7	339.0	383.1	383.5	383.8	384.0	384.1	347.9	338.8	329.5	324.8	322.0
	8	403.2	483.8	446.6	429.4	412.9	404.9	400.2	479.5	478.0	476.6	475.9	475.4	441.6	421.8	402.2	392.5	386.7
	12	403.2	637.9	602.4	586.9	572.4	565.5	561.5	615.4	606.3	597.7	593.7	591.3	578.6	552.1	526.6	514.2	506.9
80	6	582.4	533.7	473.5	457.7	440.5	431.3	425.6	530.2	529.0	527.7	526.9	526.5	467.0	428.2	395.5	377.4	365.7
	8	582.4	662.4	593.0	555.4	534.1	527.8	524.0	648.3	641.4	634.2	630.5	628.2	576.7	528.6	475.4	446.6	428.5
	12	582.4	861.9	789.2	751.7	713.4	694.0	682.2	811.5	786.6	761.2	748.3	740.4	736.5	670.1	600.0	563.5	541.0
100	6	825.4	736.2	644.5	612.0	577.3	569.9	566.3	725.6	720.2	713.9	710.4	708.1	631.4	565.0	487.9	448.3	422.0
	8	825.4	907.2	799.8	741.6	713.4	697.8	688.0	877.0	860.3	841.5	831.2	824.6	766.8	681.3	580.9	523.8	487.0
	12	825.4	1176.6	1055.2	986.9	912.8	873.3	848.8	1087.3	1038.2	984.9	956.5	938.9	964.7	844.0	711.0	639.3	594.4
120	6	1163.5	1015.4	887.7	830.5	789.0	774.1	764.3	991.6	977.7	960.8	950.8	944.2	861.4	758.9	627.9	549.9	502.1
	8	1163.5	1248.5	1093.7	1014.3	958.4	926.4	905.7	1192.6	1158.8	1118.9	1096.0	1081.2	1035.8	900.2	735.6	639.4	576.3
	12	1163.5	1622.5	1435.1	1322.8	1195.2	1124.6	1079.9	1477.2	1391.2	1293.5	1239.5	1205.2	1293.0	1098.4	878.7	758.0	681.7

b. <u>Distribution factor</u>

				DF=Mmax/Mo (steel + slab) (2 lanes)															
L (ft)	s (ft)	AASHTO	NCHRP	No	RL					RR					2R				
					x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	1.09	1.2	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
	8	1.46	1.48	1.36	1.32	1.31	1.30	1.30	1.29	1.36	1.36	1.36	1.36	1.36	1.32	1.31	1.30	1.30	1.30
	12	2.18	1.98	1.91	1.89	1.88	1.87	1.87	1.87	1.90	1.90	1.89	1.89	1.89	1.87	1.86	1.85	1.85	1.85
60	6	1.09	1.08	0.95	0.88	0.87	0.85	0.85	0.84	0.95	0.95	0.95	0.95	0.95	0.86	0.84	0.82	0.81	0.80
	8	1.46	1.32	1.20	1.11	1.07	1.02	1.00	0.99	1.19	1.19	1.18	1.18	1.18	1.10	1.05	1.00	0.97	0.96
	12	2.18	1.77	1.58	1.49	1.46	1.42	1.40	1.39	1.53	1.50	1.48	1.47	1.47	1.44	1.37	1.31	1.28	1.26
80	6	1.09	1.01	0.92	0.81	0.79	0.76	0.74	0.73	0.91	0.91	0.91	0.90	0.90	0.80	0.74	0.68	0.65	0.63
	8	1.46	1.23	1.14	1.02	0.95	0.92	0.91	0.90	1.11	1.10	1.09	1.08	1.08	0.99	0.91	0.82	0.77	0.74
	12	2.18	1.64	1.48	1.35	1.29	1.22	1.19	1.17	1.39	1.35	1.31	1.28	1.27	1.26	1.15	1.03	0.97	0.93
100	6	1.09	0.95	0.89	0.78	0.74	0.70	0.69	0.69	0.88	0.87	0.86	0.86	0.86	0.76	0.68	0.59	0.54	0.51
	8	1.46	1.16	1.10	0.97	0.90	0.86	0.85	0.83	1.06	1.04	1.02	1.01	1.00	0.93	0.83	0.70	0.63	0.59
	12	2.18	1.54	1.43	1.28	1.20	1.11	1.06	1.03	1.32	1.26	1.19	1.16	1.14	1.17	1.02	0.86	0.77	0.72
120	6	1.09	0.91	0.87	0.76	0.71	0.68	0.67	0.66	0.85	0.84	0.83	0.82	0.81	0.74	0.65	0.54	0.47	0.43
	8	1.46	1.1	1.07	0.94	0.87	0.82	0.80	0.78	1.02	1.00	0.96	0.94	0.93	0.89	0.77	0.63	0.55	0.50
	12	2.18	1.47	1.39	1.23	1.14	1.03	0.97	0.93	1.27	1.20	1.11	1.07	1.04	1.11	0.94	0.76	0.65	0.59

Table 4.112. Comparison of FEA Distribution Factors in Interior Girders (Steel+ Slab)[2-lane bridges] with AASHTO (2002) , NCHRP 12-26 and FEA Distribution Factors of the No Railing Case

a. AASHTO

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel + slab) (2 lanes)																		
L (ft)	s (ft)	AASHTO	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-3	-3	-4	-4	-4	-4	-3	-3	-3	-4	-4	-4	-4	-4	-4	-4
	8	0	-7	-10	-10	-11	-11	-11	-7	-7	-7	-7	-7	-10	-10	-11	-11	-11
	12	0	-12	-13	-14	-14	-14	-14	-13	-13	-13	-13	-13	-14	-15	-15	-15	-15
60	6	0	-13	-19	-21	-22	-22	-23	-13	-13	-13	-13	-13	-21	-23	-25	-26	-27
	8	0	-18	-24	-27	-30	-31	-32	-19	-19	-19	-19	-19	-25	-28	-32	-33	-34
	12	0	-27	-31	-33	-35	-36	-36	-30	-31	-32	-32	-33	-34	-37	-40	-41	-42
80	6	0	-16	-25	-28	-31	-32	-33	-16	-17	-17	-17	-17	-26	-33	-38	-41	-42
	8	0	-22	-30	-35	-37	-38	-38	-24	-25	-25	-26	-26	-32	-38	-44	-47	-50
	12	0	-32	-38	-41	-44	-45	-46	-36	-38	-40	-41	-42	-42	-47	-53	-56	-57
100	6	0	-18	-28	-32	-36	-37	-37	-19	-20	-21	-21	-21	-30	-37	-46	-50	-53
	8	0	-25	-34	-38	-41	-42	-43	-27	-29	-30	-31	-32	-36	-43	-52	-57	-60
	12	0	-35	-41	-45	-49	-51	-53	-40	-42	-45	-47	-48	-46	-53	-60	-64	-67
120	6	0	-20	-30	-35	-38	-39	-40	-22	-23	-24	-25	-26	-32	-40	-50	-57	-60
	8	0	-27	-36	-40	-44	-45	-47	-30	-32	-34	-35	-36	-39	-47	-57	-62	-66
	12	0	-36	-43	-48	-53	-56	-57	-42	-45	-49	-51	-52	-49	-57	-65	-70	-73

b. NCHRP

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel + slab) (2 lanes)																		
L (ft)	s (ft)	NCHRP	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-13	-13	-13	-13
	8	0	-8	-11	-12	-12	-12	-13	-8	-8	-8	-8	-8	-11	-12	-12	-12	-12
	12	0	-3	-5	-5	-5	-5	-6	-4	-4	-4	-4	-5	-5	-6	-6	-7	-7
60	6	0	-12	-19	-20	-21	-22	-22	-12	-12	-12	-12	-12	-20	-22	-24	-25	-26
	8	0	-9	-16	-19	-22	-24	-25	-10	-10	-10	-11	-11	-17	-21	-24	-26	-27
	12	0	-11	-16	-18	-20	-21	-21	-14	-15	-16	-17	-17	-19	-23	-26	-28	-29
80	6	0	-9	-20	-22	-25	-27	-28	-10	-10	-10	-10	-10	-21	-27	-33	-36	-38
	8	0	-8	-17	-22	-25	-26	-27	-10	-10	-11	-12	-12	-19	-26	-34	-38	-40
	12	0	-10	-17	-21	-25	-27	-29	-15	-18	-20	-22	-22	-23	-30	-37	-41	-43
100	6	0	-6	-18	-22	-26	-27	-28	-7	-8	-9	-9	-10	-19	-28	-38	-43	-46
	8	0	-5	-16	-23	-25	-27	-28	-8	-10	-12	-13	-14	-20	-29	-39	-45	-49
	12	0	-7	-17	-22	-28	-31	-33	-14	-18	-23	-25	-26	-24	-34	-44	-50	-53
120	6	0	-4	-16	-22	-25	-27	-28	-6	-8	-9	-10	-11	-19	-28	-41	-48	-53
	8	0	-2	-15	-21	-25	-28	-29	-7	-9	-13	-14	-16	-19	-30	-43	-50	-55
	12	0	-5	-16	-23	-30	-34	-37	-14	-19	-24	-28	-30	-24	-36	-49	-56	-60

c. No R

% decrease in DF=[(Railing-NoR)/NoR]x100 (steel + slab) (2 lanes)																	
L (ft)	s (ft)	No	RL					RR					2R				
			x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	8	0	-3	-3	-4	-4	-5	0	0	0	0	0	-3	-3	-4	-4	-4
	12	0	-1	-2	-2	-2	-2	-1	-1	-1	-1	-1	-2	-3	-3	-3	-4
60	6	0	-7	-9	-10	-11	-12	0	0	0	0	0	-9	-12	-14	-15	-16
	8	0	-8	-11	-15	-16	-17	-1	-1	-1	-2	-2	-9	-13	-17	-19	-20
	12	0	-6	-8	-10	-11	-12	-4	-5	-6	-7	-7	-9	-13	-17	-19	-21
80	6	0	-11	-14	-17	-19	-20	-1	-1	-1	-1	-1	-12	-20	-26	-29	-31
	8	0	-10	-16	-19	-20	-21	-2	-3	-4	-5	-5	-13	-20	-28	-33	-35
	12	0	-8	-13	-17	-19	-21	-6	-9	-12	-13	-14	-15	-22	-30	-35	-37
100	6	0	-12	-17	-22	-23	-23	-1	-2	-3	-4	-4	-14	-23	-34	-39	-43
	8	0	-12	-18	-21	-23	-24	-3	-5	-7	-8	-9	-15	-25	-36	-42	-46
	12	0	-10	-16	-22	-26	-28	-8	-12	-16	-19	-20	-18	-28	-40	-46	-49
120	6	0	-13	-18	-22	-24	-25	-2	-4	-5	-6	-7	-15	-25	-38	-46	-51
	8	0	-12	-19	-23	-26	-27	-4	-7	-10	-12	-13	-17	-28	-41	-49	-54
	12	0	-12	-18	-26	-31	-33	-9	-14	-20	-24	-26	-20	-32	-46	-53	-58

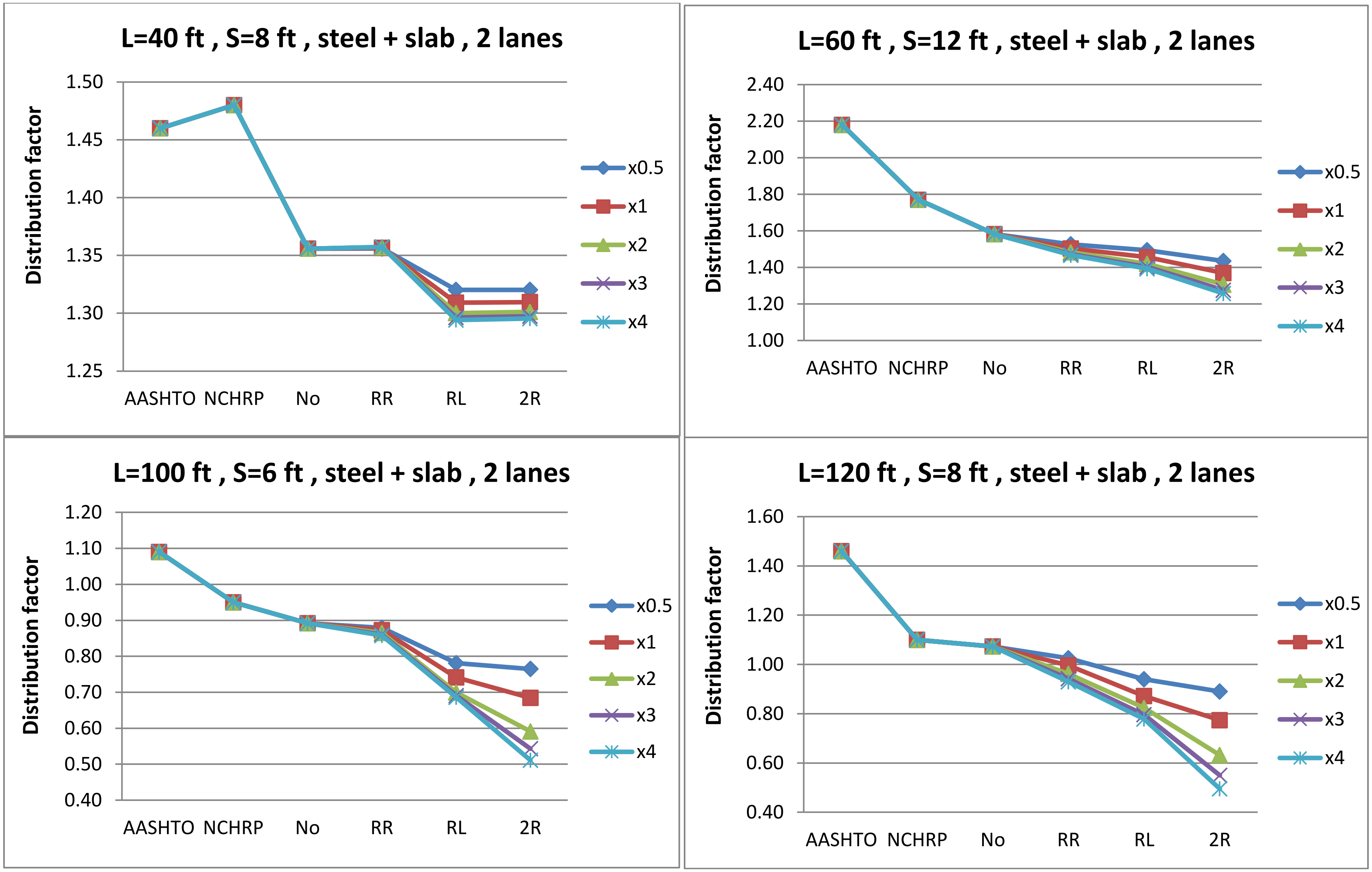


Figure 4.19. Sensitivity of Distribution Factor to Railing stiffness for Interior Girders (Steel +Slab, 2-lane bridges

Table 4.17. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel only) [2-lane Bridges]

a. Maximum bending moment (kip .ft)

Maximum bending moment Mmax (kip.ft) (steel only) (2 lanes)																		
L (ft)	s (ft)	Mo (kip-ft)	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	224.8	220.0	219.3	219.1	218.9	218.9	218.8	219.6	219.5	219.5	219.4	219.4	218.8	218.6	218.3	218.3	218.2
	8	224.8	276.3	268.1	265.4	263.3	262.4	261.9	276.2	276.3	276.4	276.4	276.4	268.0	265.4	263.4	262.5	262.0
	12	224.8	370.9	364.7	362.9	361.4	360.8	360.4	367.4	366.6	366.0	365.7	365.6	361.2	358.5	356.3	355.3	354.8
60	6	403.2	358.3	330.8	324.8	319.0	316.1	314.4	358.1	358.3	358.5	358.6	358.7	323.8	314.3	304.5	299.6	296.6
	8	403.2	444.6	407.5	390.2	373.7	365.7	360.9	440.1	438.4	436.7	436.0	435.5	402.1	382.1	362.2	352.4	346.5
	12	403.2	559.6	524.5	509.1	494.7	487.8	483.8	544.1	546.4	548.6	549.6	550.2	501.2	474.8	449.4	437.0	429.6
80	6	582.4	514.8	444.9	429.0	411.6	402.3	396.5	521.8	526.6	531.8	534.6	536.3	436.9	398.6	366.0	347.7	335.9
	8	582.4	622.9	546.3	509.3	488.0	481.6	477.7	629.5	634.0	638.8	641.3	642.9	529.9	482.1	429.2	400.5	382.6
	12	582.4	795.0	697.5	661.1	623.9	605.0	593.5	796.1	797.8	799.6	800.5	801.1	646.5	581.9	513.6	478.1	456.2
100	6	825.4	729.8	609.3	578.2	548.2	542.3	538.5	736.8	742.6	749.3	753.1	755.6	596.0	530.2	454.4	414.8	388.5
	8	825.4	883.6	744.4	687.5	659.4	647.2	649.4	886.8	890.3	894.3	896.5	897.9	711.6	627.6	528.8	472.7	436.5
	12	825.4	1129.4	946.8	881.1	828.8	825.2	823.0	1118.0	1113.4	1108.4	1105.8	1104.2	859.8	743.5	615.3	546.2	503.0
120	6	1163.5	1016.9	845.0	789.8	754.6	755.5	758.3	1019.0	1022.7	1027.4	1030.1	1031.9	834.0	717.7	588.8	512.0	464.7
	8	1163.5	1233.3	1026.2	948.7	917.4	914.9	913.3	1224.9	1222.1	1218.9	1217.2	1216.0	981.0	836.9	676.2	582.2	520.6
	12	1163.5	1580.8	1303.2	1196.0	1162.1	1145.5	1135.0	1541.9	1521.4	1498.3	1485.5	1477.5	1199.2	981.2	771.2	655.7	582.7

b. <u>Distribution factor</u>

				DF=Mmax/Mo (steel only) (2 lanes)															
L (ft)	s (ft)	AASHTO	NCHRP	No	RL					RR					2R				
					x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	1.09	1.2	0.98	0.98	0.97	0.97	0.97	0.97	0.98	0.98	0.98	0.98	0.98	0.97	0.97	0.97	0.97	0.97
	8	1.46	1.48	1.23	1.19	1.18	1.17	1.17	1.16	1.23	1.23	1.23	1.23	1.23	1.19	1.18	1.17	1.17	1.17
	12	2.18	1.98	1.65	1.62	1.61	1.61	1.60	1.60	1.63	1.63	1.63	1.63	1.63	1.61	1.59	1.58	1.58	1.58
60	6	1.09	1.08	0.89	0.82	0.81	0.79	0.78	0.78	0.89	0.89	0.89	0.89	0.89	0.80	0.78	0.76	0.74	0.74
	8	1.46	1.32	1.10	1.01	0.97	0.93	0.91	0.90	1.09	1.09	1.08	1.08	1.08	1.00	0.95	0.90	0.87	0.86
	12	2.18	1.77	1.39	1.30	1.26	1.23	1.21	1.20	1.35	1.36	1.36	1.36	1.36	1.24	1.18	1.11	1.08	1.07
80	6	1.09	1.01	0.88	0.76	0.74	0.71	0.69	0.68	0.90	0.90	0.91	0.92	0.92	0.75	0.68	0.63	0.60	0.58
	8	1.46	1.23	1.07	0.94	0.87	0.84	0.83	0.82	1.08	1.09	1.10	1.10	1.10	0.91	0.83	0.74	0.69	0.66
	12	2.18	1.64	1.37	1.20	1.14	1.07	1.04	1.02	1.37	1.37	1.37	1.37	1.38	1.11	1.00	0.88	0.82	0.78
100	6	1.09	0.95	0.88	0.74	0.70	0.66	0.66	0.65	0.89	0.90	0.91	0.91	0.92	0.72	0.64	0.55	0.50	0.47
	8	1.46	1.16	1.07	0.90	0.83	0.80	0.78	0.79	1.07	1.08	1.08	1.09	1.09	0.86	0.76	0.64	0.57	0.53
	12	2.18	1.54	1.37	1.15	1.07	1.00	1.00	1.00	1.35	1.35	1.34	1.34	1.34	1.04	0.90	0.75	0.66	0.61
120	6	1.09	0.91	0.87	0.73	0.68	0.65	0.65	0.65	0.88	0.88	0.88	0.89	0.89	0.72	0.62	0.51	0.44	0.40
	8	1.46	1.1	1.06	0.88	0.82	0.79	0.79	0.78	1.05	1.05	1.05	1.05	1.05	0.84	0.72	0.58	0.50	0.45
	12	2.18	1.47	1.36	1.12	1.03	1.00	0.98	0.98	1.33	1.31	1.29	1.28	1.27	1.03	0.84	0.66	0.56	0.50

Table 4.18. Comparison of FEA Distribution Factors in All Steel girders (Steel only)[2-lane Bridges] with AASHTO (2002), NCHRP 12-26 and FEA Distribution Factors of the No Railing Case

a. <u>AASHTO</u>

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel only) (2 lanes)																		
L (ft)	s (ft)	AASHTO	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-10	-11	-11	-11	-11	-11	-10	-10	-10	-10	-10	-11	-11	-11	-11	-11
	8	0	-16	-18	-19	-20	-20	-20	-16	-16	-16	-16	-16	-18	-19	-20	-20	-20
	12	0	-24	-26	-26	-26	-26	-26	-25	-25	-25	-25	-25	-26	-27	-27	-28	-28
60	6	0	-18	-25	-26	-27	-28	-28	-19	-18	-18	-18	-18	-26	-28	-31	-32	-33
	8	0	-24	-31	-34	-37	-38	-39	-25	-26	-26	-26	-26	-32	-35	-38	-40	-41
	12	0	-36	-40	-42	-44	-44	-45	-38	-38	-38	-37	-37	-43	-46	-49	-50	-51
80	6	0	-19	-30	-32	-35	-37	-38	-18	-17	-16	-16	-16	-31	-37	-42	-45	-47
	8	0	-27	-36	-40	-43	-43	-44	-26	-25	-25	-25	-24	-38	-43	-50	-53	-55
	12	0	-37	-45	-48	-51	-52	-53	-37	-37	-37	-37	-37	-49	-54	-60	-62	-64
100	6	0	-19	-32	-36	-39	-40	-40	-18	-17	-17	-16	-16	-34	-41	-49	-54	-57
	8	0	-27	-38	-43	-45	-46	-46	-26	-26	-26	-26	-25	-41	-48	-56	-61	-64
	12	0	-37	-47	-51	-54	-54	-54	-38	-38	-38	-39	-39	-52	-59	-66	-70	-72
120	6	0	-20	-33	-38	-40	-40	-40	-20	-19	-19	-19	-19	-34	-43	-54	-60	-63
	8	0	-27	-40	-44	-46	-46	-46	-28	-28	-28	-28	-28	-42	-51	-60	-66	-69
	12	0	-38	-49	-53	-54	-55	-55	-39	-40	-41	-41	-42	-53	-61	-70	-74	-77

b. <u>NCHRP</u>

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel only) (2 lanes)																		
L (ft)	s (ft)	NCHRP	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-18	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19
	8	0	-17	-19	-20	-21	-21	-21	-17	-17	-17	-17	-17	-19	-20	-21	-21	-21
	12	0	-17	-18	-18	-19	-19	-19	-17	-18	-18	-18	-18	-19	-19	-20	-20	-20
60	6	0	-18	-24	-25	-27	-27	-28	-18	-18	-18	-18	-18	-26	-28	-30	-31	-32
	8	0	-16	-23	-27	-30	-31	-32	-17	-18	-18	-18	-18	-24	-28	-32	-34	-35
	12	0	-22	-26	-29	-31	-32	-32	-24	-23	-23	-23	-23	-30	-33	-37	-39	-40
80	6	0	-12	-24	-27	-30	-32	-33	-11	-10	-10	-9	-9	-26	-32	-38	-41	-43
	8	0	-13	-24	-29	-32	-33	-33	-12	-12	-11	-10	-10	-26	-33	-40	-44	-47
	12	0	-17	-27	-31	-35	-37	-38	-17	-16	-16	-16	-16	-32	-39	-46	-50	-52
100	6	0	-7	-22	-26	-30	-31	-31	-6	-5	-4	-4	-4	-24	-32	-42	-47	-50
	8	0	-8	-22	-28	-31	-32	-32	-7	-7	-7	-6	-6	-26	-34	-45	-51	-54
	12	0	-11	-26	-31	-35	-35	-35	-12	-12	-13	-13	-13	-32	-42	-52	-57	-60
120	6	0	-4	-20	-25	-29	-29	-28	-4	-3	-3	-3	-3	-21	-32	-44	-52	-56
	8	0	-4	-20	-26	-28	-29	-29	-4	-5	-5	-5	-5	-23	-35	-47	-55	-59
	12	0	-8	-24	-30	-32	-33	-34	-10	-11	-12	-13	-14	-30	-43	-55	-62	-66

c. <u>No R</u>

% decrease in DF=[(Railing-NoR)/NoR]x100 (steel only) (2 lanes)																	
L (ft)	s (ft)	No	RL					RR					2R				
			x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	0	0	0	-1	-1	0	0	0	0	0	-1	-1	-1	-1	-1
	8	0	-3	-4	-5	-5	-5	0	0	0	0	0	-3	-4	-5	-5	-5
	12	0	-2	-2	-3	-3	-3	-1	-1	-1	-1	-1	-3	-3	-4	-4	-4
60	6	0	-8	-9	-11	-12	-12	0	0	0	0	0	-10	-12	-15	-16	-17
	8	0	-8	-12	-16	-18	-19	-1	-1	-2	-2	-2	-10	-14	-19	-21	-22
	12	0	-6	-9	-12	-13	-14	-3	-2	-2	-2	-2	-10	-15	-20	-22	-23
80	6	0	-14	-17	-20	-22	-23	1	2	3	4	4	-15	-23	-29	-32	-35
	8	0	-12	-18	-22	-23	-23	1	2	3	3	3	-15	-23	-31	-36	-39
	12	0	-12	-17	-22	-24	-25	0	0	1	1	1	-19	-27	-35	-40	-43
100	6	0	-17	-21	-25	-26	-26	1	2	3	3	4	-18	-27	-38	-43	-47
	8	0	-16	-22	-25	-27	-27	0	1	1	1	2	-19	-29	-40	-47	-51
	12	0	-16	-22	-27	-27	-27	-1	-1	-2	-2	-2	-24	-34	-46	-52	-55
120	6	0	-17	-22	-26	-26	-25	0	1	1	1	1	-18	-29	-42	-50	-54
	8	0	-17	-23	-26	-26	-26	-1	-1	-1	-1	-1	-20	-32	-45	-53	-58
	12	0	-18	-24	-26	-28	-28	-2	-4	-5	-6	-7	-24	-38	-51	-59	-63

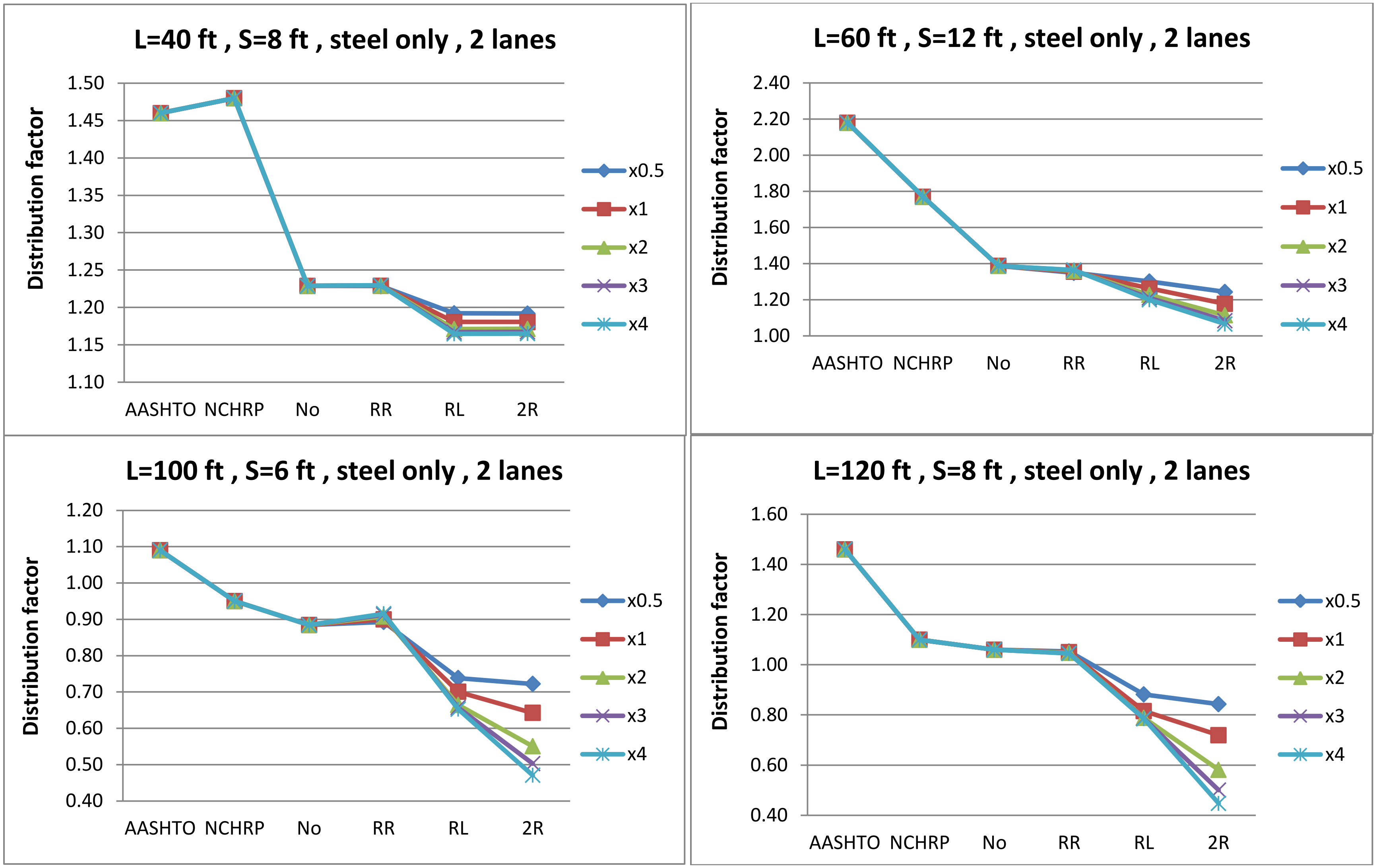


Figure 4.20. Sensitivity of Distribution Factor to Railing Stiffness for Interior and exterior Girders (Steel only, 2-lane bridges)

After analyzing 240 2-lane bridges (including all railing stiffnesses) , 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. b. is used to extract the following general conclusions for interior girders when introducing railings to a bridge deck:

1. No 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. Railing on one side (left or right): (1) is about 12 % higher than FEA distribution factors for x0.5, 15% for x1 , 16 % for x2 , 17% for x3 and 17 % for x4 .
3. Railing on both sides: (1) is about 20 % higher than FEA distribution factors for x0.5, 30% for x1 , 45 % for x2 , 50% for x3 , 60 % for x4 .

A conservative comparison of FEA distribution factors for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing railings to the bridge deck and compared to the distribution factors of the No R case as shown in Table 4.16.c. First of all, it's important to note that for spans up to 40 ft, the addition of railings has a negligible effect on the distribution factor for all stiffness factors. Otherwise, the average reductions in FEA distribution factors when introducing railing on one side for spans between 60 and 80 ft were between 5% and 11% for stiffness factors x0.5 through x4, and between 8 % to 20 % for stiffness factors x0.5 through x4 for spans between 80 and 120 ft compared to the base case (no railings).

The average reductions in FEA distribution factors when introducing railings on both sides for spans between 60 and 80 ft were between 12% and 28% for stiffness factors x0.5 through x4, and between 16 % to 50 % for stiffness factors x0.5 through x4 for spans between 80 and 120 ft compared to the base case (no railings).

The finite-element results show the effects of stiffness of 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 when increasing the stiffness of railings in a bridge superstructure.

4.3.1.2. Three-lane bridges

Tables 4.19 through 4.22 show the results for 240 3-lane bridges. Tables 4.19 and 4.20 show results while taking into consideration the contribution of the slab , which means that the critical girder is an interior girder. Whereas ; Tables 4.21 and 4.22 show results without taking into consideration the contribution of the slab , which means that the critical girder is either an interior or an exterior girder . Tables 4.19 and 4.21 show the maximum FEA bending moment in the critical girder , and the distribution factors , and Tables 4.20 and 4.22 show the % decrease of distribution factor compared to AASHTO and NCHRP and with respect to the FEA distribution factors of the reference case (No railings).

From the tables we can notice the decrease in the maximum bending moment when we increase the stiffness of railings , whether we have one railing or 2 railings. The effect increases with the increase in span length, girder spacing and when 2 railings are introduced [These differences can be seen clearly from tables 4.19 and 4.21].

Figures 4.21 and 4.22 show ; in chart form ; the sensitivity of distribution factor to railing stiffness for some bridge cases (arbitrarily chosen) [For a complete set of charts for two-lane bridges refer to the appendix A.25 to A.39].

Table 4.19. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab) [3-lane Bridges]

a. Maximum bending moment (kip .ft)

Maximum bending moment Mmax (kip.ft) (steel + slab) (3 lanes)																		
L (ft)	s (ft)	Mo (kip-ft)	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	224.8	268.7	268.4	268.9	269.4	269.6	269.7	268.6	268.6	268.7	268.7	268.7	268.3	268.8	269.3	269.5	269.6
	8	224.8	352.1	351.6	351.6	351.7	351.7	351.8	352.1	352.3	352.4	352.5	352.5	351.7	351.9	352.1	352.2	352.2
	12	224.8	492.2	492.8	493.1	493.3	493.4	493.5	487.8	486.7	485.9	485.5	485.3	488.4	487.6	487.0	486.7	486.6
60	6	403.2	440.9	428.9	424.0	423.4	423.1	423.0	441.2	441.6	442.0	442.2	442.3	429.2	423.9	421.9	421.5	421.3
	8	403.2	551.3	543.0	539.2	537.3	537.5	537.6	550.5	550.7	550.9	551.0	551.1	542.2	538.5	534.7	532.6	531.3
	12	403.2	766.9	765.0	764.6	764.2	764.1	764.0	736.5	729.2	728.8	728.6	728.4	734.2	720.7	707.9	701.7	698.1
80	6	582.4	597.5	565.5	547.2	536.9	532.5	529.7	599.2	601.0	603.0	604.0	604.7	564.0	544.8	523.2	514.0	509.3
	8	582.4	714.7	689.3	683.9	679.2	676.6	674.9	708.1	704.9	701.2	699.2	697.9	682.2	663.4	641.0	628.1	619.8
	12	582.4	1017.2	1003.2	996.7	990.0	986.5	984.4	955.4	947.5	939.2	935.0	932.5	929.5	883.1	833.7	807.8	791.7
100	6	825.4	835.0	752.6	715.5	689.0	675.9	667.3	837.8	840.5	843.7	845.5	846.7	745.8	703.4	650.7	619.1	601.4
	8	825.4	941.6	895.8	882.6	866.6	857.1	850.9	923.5	912.7	899.4	893.3	893.6	871.8	825.7	767.0	731.1	706.9
	12	825.4	1368.5	1332.8	1313.2	1291.6	1280.0	1272.7	1271.2	1248.2	1222.8	1209.1	1200.6	1201.8	1103.1	990.2	927.4	887.4
120	6	1163.5	1158.3	1017.4	952.6	901.1	878.3	870.7	1159.3	1161.5	1164.4	1166.1	1167.2	1016.0	922.8	822.5	759.2	715.6
	8	1163.5	1263.0	1189.5	1161.5	1125.9	1110.6	1108.6	1231.6	1228.3	1224.1	1221.6	1219.9	1139.6	1053.0	938.1	865.3	815.0
	12	1163.5	1864.5	1793.1	1750.5	1701.4	1673.9	1656.3	1715.7	1666.9	1610.6	1579.0	1558.8	1588.3	1414.8	1208.1	1089.1	1011.7

b. Distribution factor

				Distribution factor DF=Mmax/Mo (steel + slab) (3 lanes)															
L (ft)	s (ft)	AASHTO	NCHRP	No	RL					RR					2R				
					x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	1.09	1.2	1.08	1.07	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.07	1.08	1.08	1.08	1.08
	8	1.46	1.48	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41
	12	2.18	1.98	1.97	1.97	1.97	1.97	1.98	1.98	1.95	1.95	1.94	1.94	1.94	1.96	1.95	1.95	1.95	1.95
60	6	1.09	1.08	0.98	0.96	0.95	0.95	0.94	0.94	0.98	0.99	0.99	0.99	0.99	0.96	0.95	0.94	0.94	0.94
	8	1.46	1.32	1.23	1.21	1.20	1.20	1.20	1.20	1.23	1.23	1.23	1.23	1.23	1.21	1.20	1.19	1.19	1.19
	12	2.18	1.77	1.71	1.71	1.71	1.71	1.71	1.71	1.64	1.63	1.63	1.63	1.63	1.64	1.61	1.58	1.57	1.56
80	6	1.09	1.01	0.92	0.87	0.85	0.83	0.82	0.82	0.93	0.93	0.93	0.93	0.93	0.87	0.84	0.81	0.79	0.79
	8	1.46	1.23	1.10	1.07	1.06	1.05	1.05	1.04	1.09	1.09	1.08	1.08	1.08	1.05	1.03	0.99	0.97	0.96
	12	2.18	1.64	1.57	1.55	1.54	1.53	1.52	1.52	1.48	1.46	1.45	1.44	1.44	1.44	1.36	1.29	1.25	1.22
100	6	1.09	0.95	0.91	0.82	0.78	0.75	0.74	0.73	0.91	0.92	0.92	0.92	0.92	0.81	0.77	0.71	0.68	0.66
	8	1.46	1.16	1.03	0.98	0.96	0.94	0.93	0.93	1.01	1.00	0.98	0.97	0.97	0.95	0.90	0.84	0.80	0.77
	12	2.18	1.54	1.49	1.45	1.43	1.41	1.40	1.39	1.39	1.36	1.33	1.32	1.31	1.31	1.20	1.08	1.01	0.97
120	6	1.09	0.91	0.90	0.79	0.74	0.70	0.68	0.67	0.90	0.90	0.90	0.90	0.90	0.79	0.71	0.64	0.59	0.55
	8	1.46	1.1	0.98	0.92	0.90	0.87	0.86	0.86	0.95	0.95	0.95	0.94	0.94	0.88	0.81	0.73	0.67	0.63
	12	2.18	1.47	1.44	1.39	1.35	1.32	1.29	1.28	1.33	1.29	1.25	1.22	1.21	1.23	1.09	0.93	0.84	0.78

Table 4.20. Comparison of FEA Distribution Factors in Interior Girders (Steel+ Slab)[3-lane Bridges] with AASHTO (2002),NCHRP 12-26 and FEA Distribution Factors of the No Railing Case

a. AASHTO

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel + slab) (3 lanes)																		
L (ft)	s (ft)	AASHTO	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
	8	0	-3	-4	-4	-4	-4	-4	-3	-3	-3	-3	-3	-4	-4	-3	-3	-3
	12	0	-10	-10	-9	-9	-9	-9	-10	-11	-11	-11	-11	-10	-10	-11	-11	-11
60	6	0	-10	-12	-13	-13	-13	-13	-10	-10	-9	-9	-9	-12	-13	-14	-14	-14
	8	0	-16	-17	-18	-18	-18	-18	-16	-16	-16	-16	-16	-17	-18	-18	-19	-19
	12	0	-21	-22	-22	-22	-22	-22	-25	-25	-25	-25	-25	-25	-26	-28	-28	-29
80	6	0	-15	-20	-22	-24	-25	-25	-15	-15	-15	-14	-14	-20	-23	-26	-27	-28
	8	0	-24	-27	-28	-28	-28	-29	-25	-25	-26	-26	-26	-28	-30	-32	-34	-34
	12	0	-28	-29	-29	-30	-30	-30	-32	-33	-33	-34	-34	-34	-37	-41	-43	-44
100	6	0	-16	-25	-28	-31	-32	-33	-16	-16	-16	-15	-15	-25	-30	-35	-38	-40
	8	0	-30	-33	-34	-35	-36	-36	-31	-32	-33	-33	-33	-35	-38	-43	-45	-47
	12	0	-32	-33	-34	-35	-36	-36	-36	-38	-39	-40	-40	-40	-45	-50	-54	-56
120	6	0	-18	-28	-32	-36	-38	-38	-18	-18	-17	-17	-17	-28	-35	-42	-46	-49
	8	0	-33	-37	-38	-40	-41	-41	-35	-35	-35	-35	-35	-40	-44	-50	-54	-57
	12	0	-34	-36	-38	-40	-41	-41	-39	-41	-43	-44	-45	-44	-50	-57	-61	-64

b. NCHRP

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel + slab) (3 lanes)																		
L (ft)	s (ft)	NCHRP	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-11	-10	-10	-10	-10
	8	0	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5
	12	0	0	0	0	0	0	0	-1	-2	-2	-2	-2	-1	-1	-2	-2	-2
60	6	0	-9	-11	-12	-12	-13	-13	-9	-9	-9	-9	-9	-11	-12	-13	-13	-13
	8	0	-7	-8	-9	-9	-9	-9	-7	-7	-7	-7	-7	-8	-9	-10	-10	-10
	12	0	-3	-4	-4	-4	-4	-4	-7	-8	-8	-8	-8	-7	-9	-11	-11	-12
80	6	0	-9	-13	-16	-18	-19	-19	-8	-8	-8	-8	-7	-14	-17	-20	-21	-22
	8	0	-10	-13	-14	-15	-15	-15	-11	-11	-12	-12	-12	-14	-17	-19	-21	-22
	12	0	-4	-5	-6	-7	-7	-7	-10	-11	-11	-12	-12	-12	-17	-21	-24	-25
100	6	0	-4	-14	-18	-21	-22	-23	-4	-4	-3	-3	-3	-14	-19	-25	-29	-31
	8	0	-11	-16	-17	-19	-19	-20	-13	-14	-15	-16	-16	-18	-22	-28	-31	-34
	12	0	-3	-6	-7	-9	-9	-10	-10	-12	-13	-14	-15	-15	-22	-30	-34	-37
120	6	0	-2	-14	-19	-23	-25	-26	-1	-1	-1	-1	-1	-14	-22	-30	-35	-39
	8	0	-11	-16	-18	-21	-22	-22	-13	-14	-14	-14	-14	-20	-26	-34	-39	-43
	12	0	-2	-6	-8	-10	-12	-13	-10	-12	-15	-17	-18	-16	-26	-36	-43	-47

c. <u>No R</u>

% decrease in DF=[(Railing-NoR)/NoR]x100 (steel + slab) (3 lanes)																	
L (ft)	s (ft)	No	RL					RR					2R				
			x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	12	0	0	0	0	0	0	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
60	6	0	-3	-4	-4	-4	-4	0	0	0	0	0	-3	-4	-4	-4	-4
	8	0	-2	-2	-3	-3	-2	0	0	0	0	0	-2	-2	-3	-3	-4
	12	0	0	0	0	0	0	-4	-5	-5	-5	-5	-4	-6	-8	-9	-9
80	6	0	-5	-8	-10	-11	-11	0	1	1	1	1	-6	-9	-12	-14	-15
	8	0	-4	-4	-5	-5	-6	-1	-1	-2	-2	-2	-5	-7	-10	-12	-13
	12	0	-1	-2	-3	-3	-3	-6	-7	-8	-8	-8	-9	-13	-18	-21	-22
100	6	0	-10	-14	-17	-19	-20	0	1	1	1	1	-11	-16	-22	-26	-28
	8	0	-5	-6	-8	-9	-10	-2	-3	-4	-5	-5	-7	-12	-19	-22	-25
	12	0	-3	-4	-6	-6	-7	-7	-9	-11	-12	-12	-12	-19	-28	-32	-35
120	6	0	-12	-18	-22	-24	-25	0	0	1	1	1	-12	-20	-29	-34	-38
	8	0	-6	-8	-11	-12	-12	-2	-3	-3	-3	-3	-10	-17	-26	-31	-35
	12	0	-4	-6	-9	-10	-11	-8	-11	-14	-15	-16	-15	-24	-35	-42	-46

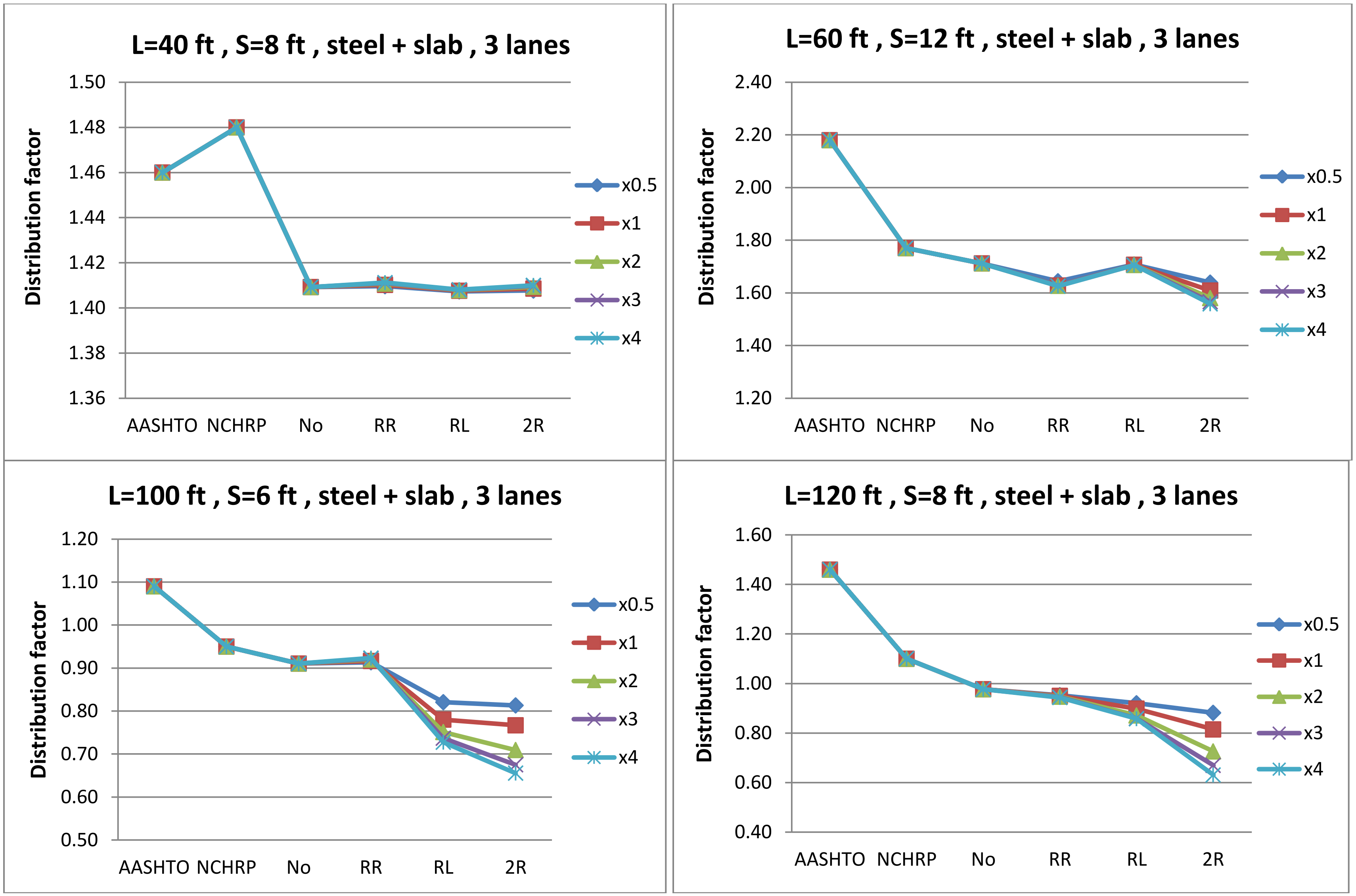


Figure 4.21. Sensitivity of Distribution Factor to Railing Stiffness for Interior and exterior Girders (Steel + Slab , 3-lane bridges)

Table 4.21. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel only) [3-lane Bridges]

a. Maximum bending moment (kip .ft)

Maximum bending moment Mmax (kip.ft) (steel only) (3 lanes)																		
L (ft)	s (ft)	Mo (kip-ft)	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	224.8	251.5	250.9	251.4	251.8	252.0	252.1	251.4	251.5	251.5	251.5	251.5	250.8	251.3	251.7	251.9	252.0
	8	224.8	321.9	321.3	321.3	321.3	321.3	321.3	321.9	322.1	322.2	322.2	322.3	321.3	321.5	321.6	321.7	321.7
	12	224.8	429.6	430.1	430.3	430.5	430.6	430.6	425.1	424.0	423.0	422.6	422.4	425.6	424.7	423.9	423.6	423.4
60	6	403.2	414.9	402.3	396.9	396.0	395.6	395.3	415.1	415.4	415.8	415.9	416.0	402.4	396.8	394.1	393.5	393.2
	8	403.2	506.0	497.4	493.4	489.1	488.2	488.2	505.0	505.0	505.0	505.0	505.0	496.3	492.3	487.9	485.6	484.1
	12	403.2	677.4	674.9	674.2	673.5	673.2	673.0	647.3	641.6	640.8	640.4	640.1	644.4	630.5	617.3	610.9	607.2
80	6	582.4	564.6	532.0	513.3	502.4	497.7	494.7	566.4	568.0	569.8	570.8	571.4	530.2	510.4	488.1	478.1	473.1
	8	582.4	658.5	632.9	624.0	618.8	615.9	614.0	651.5	648.0	643.9	641.6	640.2	625.4	605.9	582.8	569.4	560.7
	12	582.4	907.2	892.6	885.7	878.5	874.8	872.6	848.5	840.0	831.2	826.7	824.0	820.6	774.5	725.3	699.4	683.5
100	6	825.4	805.5	712.3	675.0	647.9	634.4	625.6	815.3	822.5	831.1	836.1	839.3	704.9	662.0	608.6	576.6	557.9
	8	825.4	874.4	825.5	811.9	795.3	785.5	779.1	855.9	844.6	830.9	829.8	829.9	804.3	757.8	698.5	662.3	637.9
	12	825.4	1235.4	1199.5	1179.8	1170.3	1175.9	1179.4	1142.5	1119.3	1093.7	1079.9	1071.3	1072.5	975.8	865.3	803.7	764.5
120	6	1163.5	1147.3	968.9	903.5	851.7	830.0	822.2	1161.5	1172.9	1187.3	1196.0	1201.7	966.9	872.8	772.4	709.0	665.2
	8	1163.5	1181.0	1105.2	1077.0	1048.5	1045.0	1042.7	1154.3	1150.5	1153.0	1164.1	1171.6	1058.6	972.4	858.2	785.8	735.9
	12	1163.5	1699.6	1660.4	1667.6	1675.9	1680.6	1683.6	1557.6	1509.6	1492.9	1498.3	1501.8	1432.2	1264.2	1063.9	948.6	873.6

b. Distribution factor

				Distribution factor DF=Mmax/Mo (steel only) (3 lanes)															
L (ft)	s (ft)	AASHTO	NCHRP	No	RL					RR					2R				
					x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	1.09	1.2	1.01	1.00	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.00	1.01	1.01	1.01	1.01
	8	1.46	1.48	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29	1.29
	12	2.18	1.98	1.72	1.72	1.72	1.72	1.72	1.72	1.70	1.70	1.69	1.69	1.69	1.70	1.70	1.70	1.70	1.69
60	6	1.09	1.08	0.93	0.90	0.89	0.88	0.88	0.88	0.93	0.93	0.93	0.93	0.93	0.90	0.89	0.88	0.88	0.88
	8	1.46	1.32	1.13	1.11	1.10	1.09	1.09	1.09	1.13	1.13	1.13	1.13	1.13	1.11	1.10	1.09	1.08	1.08
	12	2.18	1.77	1.51	1.51	1.50	1.50	1.50	1.50	1.44	1.43	1.43	1.43	1.43	1.44	1.41	1.38	1.36	1.36
80	6	1.09	1.01	0.87	0.82	0.79	0.78	0.77	0.76	0.88	0.88	0.88	0.88	0.88	0.82	0.79	0.75	0.74	0.73
	8	1.46	1.23	1.02	0.98	0.96	0.96	0.95	0.95	1.01	1.00	1.00	0.99	0.99	0.97	0.94	0.90	0.88	0.87
	12	2.18	1.64	1.40	1.38	1.37	1.36	1.35	1.35	1.31	1.30	1.28	1.28	1.27	1.27	1.20	1.12	1.08	1.06
100	6	1.09	0.95	0.88	0.78	0.74	0.71	0.69	0.68	0.89	0.90	0.91	0.91	0.92	0.77	0.72	0.66	0.63	0.61
	8	1.46	1.16	0.95	0.90	0.89	0.87	0.86	0.85	0.93	0.92	0.91	0.90	0.90	0.88	0.83	0.76	0.72	0.70
	12	2.18	1.54	1.35	1.31	1.29	1.28	1.28	1.29	1.25	1.22	1.19	1.18	1.17	1.17	1.06	0.94	0.88	0.83
120	6	1.09	0.91	0.89	0.75	0.70	0.66	0.64	0.64	0.90	0.91	0.92	0.93	0.93	0.75	0.68	0.60	0.55	0.51
	8	1.46	1.1	0.91	0.85	0.83	0.81	0.81	0.81	0.89	0.89	0.89	0.90	0.91	0.82	0.75	0.66	0.61	0.57
	12	2.18	1.47	1.31	1.28	1.29	1.30	1.30	1.30	1.20	1.17	1.15	1.16	1.16	1.11	0.98	0.82	0.73	0.68

Table 4.22. Comparison of FEA Distribution Factors in All Steel girders (Steel only)[3-lane Bridges] with AASHTO (2002) , NCHRP 12-26 and FEA distribution factors of the No railing case

a. AASHTO

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel only) (3 lanes)																		
L (ft)	s (ft)	AASHTO	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-8	-8	-8	-8	-7	-7	-8	-8	-8	-8	-8	-8	-8	-8	-7	-7
	8	0	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12	-12
	12	0	-21	-21	-21	-21	-21	-21	-22	-22	-22	-22	-22	-22	-22	-22	-22	-22
60	6	0	-15	-18	-19	-19	-19	-19	-15	-15	-15	-15	-15	-18	-19	-19	-19	-19
	8	0	-23	-24	-25	-25	-25	-25	-23	-23	-23	-23	-23	-24	-25	-25	-26	-26
	12	0	-31	-31	-31	-31	-31	-31	-34	-34	-34	-34	-34	-34	-35	-37	-37	-38
80	6	0	-20	-25	-27	-29	-29	-30	-20	-19	-19	-19	-19	-25	-28	-31	-32	-33
	8	0	-30	-33	-34	-35	-35	-35	-31	-31	-32	-32	-32	-34	-36	-38	-40	-41
	12	0	-36	-37	-37	-38	-38	-38	-40	-40	-41	-41	-42	-42	-45	-49	-50	-52
100	6	0	-19	-29	-32	-35	-37	-37	-18	-18	-17	-16	-16	-29	-34	-39	-42	-44
	8	0	-35	-38	-39	-41	-41	-42	-36	-37	-38	-38	-38	-40	-43	-48	-51	-52
	12	0	-38	-40	-41	-41	-41	-41	-43	-44	-45	-46	-46	-46	-51	-57	-60	-62
120	6	0	-19	-31	-36	-40	-41	-42	-18	-17	-16	-15	-15	-31	-38	-45	-50	-53
	8	0	-37	-41	-43	-44	-45	-45	-39	-39	-39	-38	-38	-44	-48	-55	-58	-61
	12	0	-40	-41	-41	-41	-40	-40	-45	-46	-47	-47	-47	-49	-55	-62	-66	-69

b. NCHRP

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel only) (3 lanes)																		
L (ft)	s (ft)	NCHRP	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-16	-16	-16	-16	-16	-16	-16	-16	-16	-16	-16	-16	-16	-16	-16	-16
	8	0	-13	-13	-13	-13	-13	-13	-13	-13	-13	-13	-13	-13	-13	-13	-13	-13
	12	0	-13	-13	-13	-13	-13	-13	-14	-14	-14	-15	-15	-14	-14	-14	-14	-14
60	6	0	-14	-17	-18	-18	-18	-18	-14	-14	-14	-14	-14	-17	-18	-19	-19	-19
	8	0	-14	-16	-17	-17	-17	-17	-15	-15	-15	-15	-15	-16	-17	-17	-18	-18
	12	0	-15	-15	-15	-15	-15	-15	-18	-19	-19	-19	-19	-19	-20	-22	-23	-23
80	6	0	-14	-19	-21	-23	-24	-24	-13	-13	-13	-13	-13	-19	-22	-25	-27	-28
	8	0	-17	-20	-22	-22	-23	-23	-18	-19	-19	-19	-20	-21	-24	-27	-28	-30
	12	0	-15	-16	-17	-17	-18	-18	-20	-21	-22	-22	-22	-23	-27	-32	-34	-36
100	6	0	-8	-18	-23	-26	-27	-28	-6	-6	-5	-4	-4	-19	-24	-30	-34	-36
	8	0	-18	-22	-24	-25	-26	-27	-20	-21	-22	-22	-22	-24	-29	-34	-38	-40
	12	0	-13	-15	-16	-17	-17	-16	-19	-21	-23	-24	-24	-24	-31	-39	-43	-46
120	6	0	-2	-18	-23	-28	-29	-30	-1	0	1	2	2	-18	-26	-34	-40	-43
	8	0	-17	-22	-24	-26	-27	-27	-19	-19	-19	-18	-18	-26	-32	-40	-45	-48
	12	0	-11	-13	-12	-12	-12	-11	-18	-21	-21	-21	-21	-25	-33	-44	-50	-54

c. No R

% decrease in DF=[(Railing-NoR)/NoR]x100 (steel only) (3 lanes)																	
L (ft)	s (ft)	No	RL					RR					2R				
			x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	12	0	0	0	0	0	0	-1	-1	-2	-2	-2	-1	-1	-1	-1	-1
60	6	0	-3	-4	-5	-5	-5	0	0	0	0	0	-3	-4	-5	-5	-5
	8	0	-2	-2	-3	-4	-4	0	0	0	0	0	-2	-3	-4	-4	-4
	12	0	0	0	-1	-1	-1	-4	-5	-5	-5	-6	-5	-7	-9	-10	-10
80	6	0	-6	-9	-11	-12	-12	0	1	1	1	1	-6	-10	-14	-15	-16
	8	0	-4	-5	-6	-6	-7	-1	-2	-2	-3	-3	-5	-8	-12	-14	-15
	12	0	-2	-2	-3	-4	-4	-6	-7	-8	-9	-9	-10	-15	-20	-23	-25
100	6	0	-12	-16	-20	-21	-22	1	2	3	4	4	-12	-18	-24	-28	-31
	8	0	-6	-7	-9	-10	-11	-2	-3	-5	-5	-5	-8	-13	-20	-24	-27
	12	0	-3	-5	-5	-5	-5	-8	-9	-11	-13	-13	-13	-21	-30	-35	-38
120	6	0	-16	-21	-26	-28	-28	1	2	3	4	5	-16	-24	-33	-38	-42
	8	0	-6	-9	-11	-12	-12	-2	-3	-2	-1	-1	-10	-18	-27	-33	-38
	12	0	-2	-2	-1	-1	-1	-8	-11	-12	-12	-12	-16	-26	-37	-44	-49

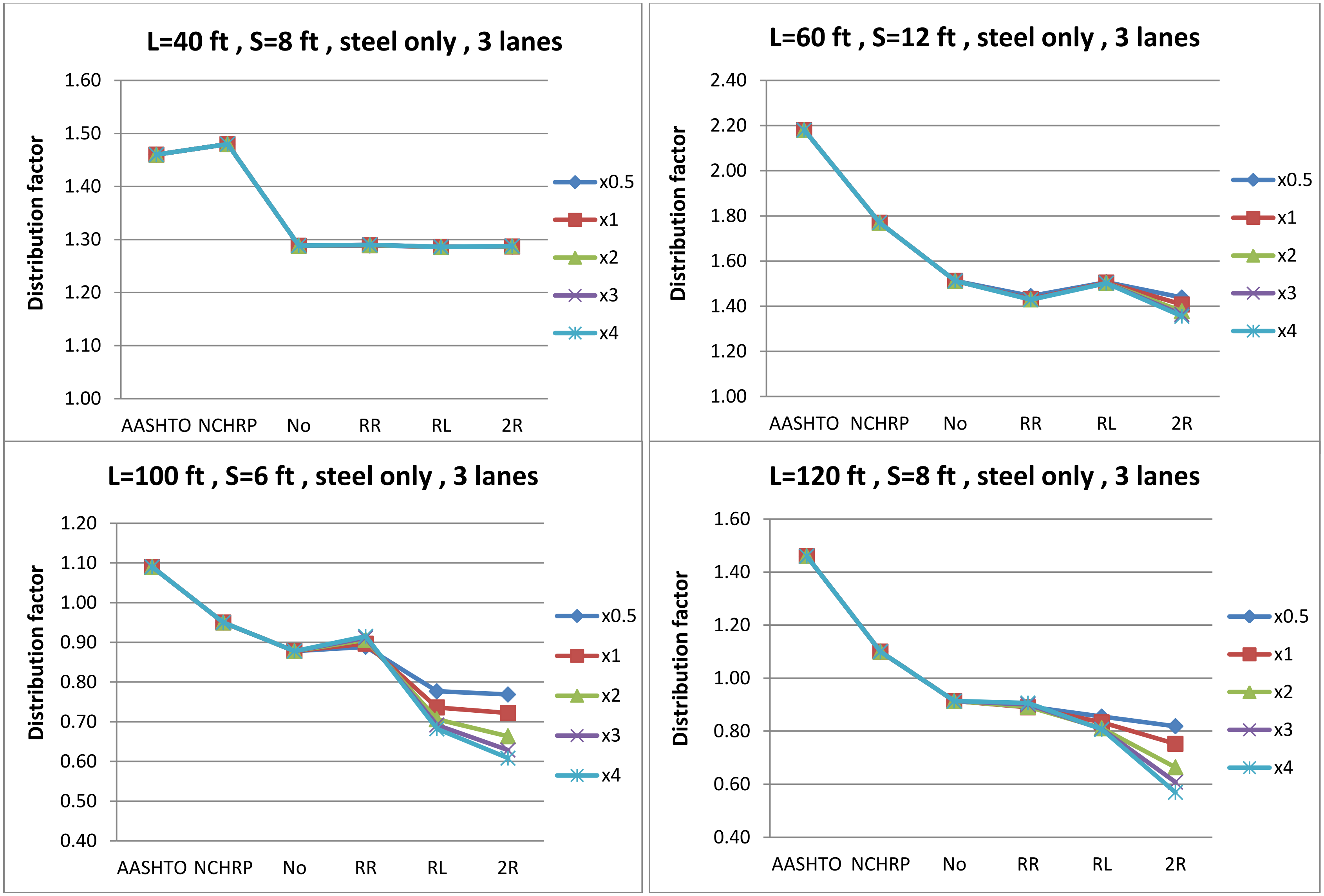


Figure 4.22. Sensitivity of Distribution Factor to Railing Stiffness for Interior and exterior Girders (Steel only, 3-lane bridges)

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.b is used to extract the following general conclusions for interior girders when introducing railings to a bridge deck:

1. No railings: the FEA distribution factors are smaller than (1) by about 6 % for all spans.
2. Railing on one side (left or right): For spans up to 80ft (1) is about 7 % higher than FEA distribution factors for all railing stiffnesses and for spans between 80 ft and 120 ft it is 10 to 13 % higher depending on railing stiffness.
3. Railing on both sides: For spans between 40 ft and 80 ft (1) is higher than FEA distribution factors by 10 % to 12% depending on railing stiffness , and for spans between 80 ft and 120 ft (1) is higher than FEA distribution factors by 20 % for x0.5 railing stiffness , 25 % for x1 , 40 % for x2 ,45 % for x3 and 50 % for x4.

Considering the various bridge geometries, a conservative comparison of FEA distribution factors for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing railings of several stiffnesses to the bridge deck and compared to the distribution factors of the No R case as shown in details in Table 4.20.c. First of all, it is important to note that for spans up to 40 ft, the addition or railings has a negligible effect on the distribution factor. Otherwise, the average reductions in FEA distribution factors when compared to the base case were : 2 % for spans between 40 and

80 ft (12 and 18 m) (almost same for all railing stiffnesses), 4 % to 6% (depending on stiffness of railing) for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on one side, and 3 % to 6 % (depending on railing stiffness) for spans between 40 and 80 ft (12 and 24 m) and 11% to 30% (depending on railing stiffness) for spans between 80 and 120 ft (24 and 36 m) when introducing railings on both sides. 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 increase of stiffness of railings in a bridge superstructure.

4.3.1.3.Four-lane bridges

Tables 4.23 through 4.26 show the results for 240 4-lane bridges. Tables 4.23 and 4.24 show results while taking into consideration the contribution of the slab , which means that the critical girder is an interior girder. Whereas ; Tables 4.25 and 4.26 show results without taking into consideration the contribution of the slab , which means that the critical girder is either an interior or an exterior girder . Tables 4.23 and 4.25 show the maximum FEA bending moment in the critical girder and the distribution factors , and Tables 4.24 and 4.26 show the % decrease of distribution factor compared to AASHTO and NCHRP and with respect to the FEA distribution factors of the reference case (No railings).

From the tables we can notice the decrease in the maximum bending moment when we increase the stiffness of railings , whether we have one railing or 2 railings. The effect increases with the increase in span length, girder spacing and when 2 railings are introduced [These differences can be seen clearly from tables 4.23 and 4.25].

Figures 4.23 and 4.24 show ; in chart form ; the sensitivity of distribution factor to railing stiffness for some bridge cases (arbitrarily chosen) [For a complete set of charts for two-lane bridges refer to the appendix A.40 to A.54].

Table 4.23. Maximum Bending Moments and Wheel Load Distribution Factors in Interior Girders (Steel + Slab) [4-lane Bridges]

a. Maximum bending moment (kip .ft)

Maximum bending moment Mmax (kip.ft) (steel + slab) (4 lanes)																		
L (ft)	s (ft)	M0 (kip-ft)	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	224.8	278.2	279.6	280.2	280.6	280.8	280.9	278.2	278.2	278.2	278.2	278.2	279.6	280.1	280.5	280.7	280.8
	8	224.8	363.6	364.4	364.8	365.2	365.4	365.5	363.6	363.6	363.6	363.6	363.6	364.3	364.7	365.1	365.3	365.4
	12	224.8	538.2	539.8	540.5	541.0	541.2	541.3	538.1	538.2	538.2	538.2	538.2	539.8	540.5	541.0	541.3	541.4
60	6	403.2	475.7	474.2	473.6	473.0	472.8	472.6	475.8	476.0	476.2	476.3	476.4	474.3	473.9	473.6	473.4	473.3
	8	403.2	612.7	604.0	604.6	605.6	606.0	606.3	613.1	613.4	613.8	614.0	614.1	604.3	604.4	605.6	606.1	606.4
	12	403.2	873.2	870.6	869.9	869.2	868.8	868.6	871.4	871.2	871.0	870.9	870.8	868.8	867.8	866.9	866.4	866.1
80	6	582.4	644.6	631.5	624.1	615.8	611.3	608.5	644.6	645.1	645.6	646.2	646.5	631.5	624.5	616.8	612.6	610.0
	8	582.4	830.2	799.6	785.7	781.1	778.6	777.1	830.3	830.9	831.5	831.9	832.1	799.6	783.4	773.1	769.5	767.3
	12	582.4	1167.5	1118.6	1106.8	1094.6	1088.4	1084.5	1167.8	1168.8	1169.8	1170.3	1170.6	1106.3	1088.6	1069.9	1060.2	1054.2
100	6	825.4	885.0	833.2	812.4	787.2	776.3	772.0	886.3	887.7	891.0	893.1	894.5	830.7	808.6	781.4	765.3	754.6
	8	825.4	1132.1	1053.9	1014.5	993.3	982.2	975.0	1135.8	1138.9	1142.6	1144.7	1146.0	1049.7	1007.7	957.8	938.3	926.1
	12	825.4	1583.8	1443.2	1408.6	1370.2	1349.4	1336.2	1578.9	1577.4	1575.7	1574.8	1574.2	1422.7	1351.7	1287.2	1251.1	1228.1
120	6	1163.5	1242.5	1119.5	1066.1	1012.6	994.2	981.8	1249.2	1254.9	1262.3	1266.8	1269.8	1118.6	1051.2	986.4	945.8	917.8
	8	1163.5	1574.5	1402.9	1329.6	1275.6	1247.6	1229.2	1578.5	1582.7	1587.8	1590.8	1592.8	1392.4	1305.2	1198.3	1133.1	1098.4
	12	1163.5	2156.7	1911.9	1822.6	1739.1	1691.9	1661.4	2138.7	2129.6	2118.9	2112.9	2109.1	1890.0	1721.4	1553.3	1467.2	1410.6

b. <u>Distribution factor</u>

				Distribution factor DF=Mmax/Mo (steel + slab) (4 lanes)															
L (ft)	s (ft)	AASHTO	NCHRP	No	RL					RR					2R				
					x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	1.09	1.2	0.93	0.93	0.93	0.94	0.94	0.94	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.94	0.94	0.94
	8	1.46	1.48	1.21	1.22	1.22	1.22	1.22	1.22	1.21	1.21	1.21	1.21	1.21	1.22	1.22	1.22	1.22	1.22
	12	2.18	1.98	1.80	1.80	1.80	1.80	1.81	1.81	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.81	1.81
60	6	1.09	1.08	0.88	0.88	0.88	0.88	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.88	0.88	0.88	0.88	0.88
	8	1.46	1.32	1.14	1.12	1.12	1.13	1.13	1.13	1.14	1.14	1.14	1.14	1.14	1.12	1.12	1.13	1.13	1.13
	12	2.18	1.77	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.62	1.61	1.61	1.61	1.61
80	6	1.09	1.01	0.83	0.81	0.80	0.79	0.79	0.78	0.83	0.83	0.83	0.83	0.83	0.81	0.80	0.79	0.79	0.79
	8	1.46	1.23	1.07	1.03	1.01	1.01	1.00	1.00	1.07	1.07	1.07	1.07	1.07	1.03	1.01	1.00	0.99	0.99
	12	2.18	1.64	1.50	1.44	1.43	1.41	1.40	1.40	1.50	1.51	1.51	1.51	1.51	1.42	1.40	1.38	1.37	1.36
100	6	1.09	0.95	0.80	0.76	0.74	0.72	0.71	0.70	0.81	0.81	0.81	0.81	0.81	0.75	0.73	0.71	0.70	0.69
	8	1.46	1.16	1.03	0.96	0.92	0.90	0.89	0.89	1.03	1.03	1.04	1.04	1.04	0.95	0.92	0.87	0.85	0.84
	12	2.18	1.54	1.44	1.31	1.28	1.25	1.23	1.21	1.43	1.43	1.43	1.43	1.43	1.29	1.23	1.17	1.14	1.12
120	6	1.09	0.91	0.80	0.72	0.69	0.65	0.64	0.63	0.81	0.81	0.81	0.82	0.82	0.72	0.68	0.64	0.61	0.59
	8	1.46	1.1	1.01	0.90	0.86	0.82	0.80	0.79	1.02	1.02	1.02	1.03	1.03	0.90	0.84	0.77	0.73	0.71
	12	2.18	1.47	1.39	1.23	1.17	1.12	1.09	1.07	1.38	1.37	1.37	1.36	1.36	1.22	1.11	1.00	0.95	0.91

Table 4.24. Comparison of FEA Distribution Factors in Interior Steel Girders (Steel + Slab)[4-lane Bridges] with AASHTO (2002), NCHRP 12-26 and FEA Distribution Factors of the No Railing Case

a. <u>AASHTO</u>

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel + slab) (4 lanes)																		
L (ft)	s (ft)	AASHTO	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-15	-14	-14	-14	-14	-14	-15	-15	-15	-15	-15	-14	-14	-14	-14	-14
	8	0	-17	-17	-17	-17	-17	-16	-17	-17	-17	-17	-17	-17	-17	-17	-17	-17
	12	0	-18	-17	-17	-17	-17	-17	-18	-18	-18	-18	-18	-17	-17	-17	-17	-17
60	6	0	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19	-19
	8	0	-22	-23	-23	-23	-23	-23	-22	-22	-22	-22	-22	-23	-23	-23	-23	-23
	12	0	-25	-26	-26	-26	-26	-26	-26	-26	-26	-26	-26	-26	-26	-26	-26	-26
80	6	0	-24	-25	-26	-27	-28	-28	-24	-24	-24	-24	-24	-25	-26	-27	-28	-28
	8	0	-27	-29	-31	-31	-31	-31	-27	-27	-27	-27	-27	-29	-31	-32	-32	-32
	12	0	-31	-34	-35	-35	-36	-36	-31	-31	-31	-31	-31	-35	-36	-37	-37	-38
100	6	0	-26	-31	-32	-34	-35	-36	-26	-26	-26	-26	-25	-31	-33	-35	-36	-37
	8	0	-30	-34	-37	-38	-39	-39	-29	-29	-29	-29	-29	-35	-37	-40	-42	-42
	12	0	-34	-40	-41	-43	-44	-44	-34	-34	-34	-34	-34	-41	-44	-46	-48	-49
120	6	0	-27	-34	-37	-40	-41	-42	-26	-26	-25	-25	-25	-34	-38	-42	-44	-46
	8	0	-30	-38	-41	-44	-45	-46	-30	-30	-30	-30	-30	-39	-42	-47	-50	-52
	12	0	-36	-43	-46	-49	-50	-51	-37	-37	-37	-38	-38	-44	-49	-54	-57	-58

b. <u>NCHRP</u>

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel + slab) (4 lanes)																		
L (ft)	s (ft)	NCHRP	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-23	-22	-22	-22	-22	-22	-23	-23	-23	-23	-23	-22	-22	-22	-22	-22
	8	0	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18
	12	0	-9	-9	-9	-9	-9	-9	-9	-9	-9	-9	-9	-9	-9	-9	-9	-9
60	6	0	-18	-18	-18	-19	-19	-19	-18	-18	-18	-18	-18	-18	-18	-18	-18	-18
	8	0	-14	-15	-15	-15	-15	-15	-14	-14	-13	-13	-13	-15	-15	-15	-15	-15
	12	0	-8	-8	-9	-9	-9	-9	-8	-8	-8	-8	-8	-9	-9	-9	-9	-9
80	6	0	-18	-19	-20	-21	-22	-22	-18	-18	-18	-18	-18	-19	-20	-21	-22	-22
	8	0	-13	-16	-18	-18	-18	-19	-13	-13	-13	-13	-13	-16	-18	-19	-19	-20
	12	0	-8	-12	-13	-14	-15	-15	-8	-8	-8	-8	-8	-13	-15	-16	-17	-17
100	6	0	-15	-20	-22	-25	-26	-26	-15	-15	-15	-15	-14	-21	-23	-25	-27	-28
	8	0	-11	-17	-21	-22	-23	-24	-11	-11	-11	-10	-10	-18	-21	-25	-27	-27
	12	0	-7	-15	-17	-19	-20	-21	-7	-7	-7	-7	-7	-16	-20	-24	-26	-28
120	6	0	-12	-21	-24	-28	-30	-30	-12	-11	-11	-10	-10	-21	-26	-30	-33	-35
	8	0	-8	-18	-22	-25	-27	-28	-7	-7	-7	-7	-7	-18	-24	-30	-34	-36
	12	0	-5	-16	-20	-24	-26	-27	-6	-7	-7	-7	-8	-17	-25	-32	-36	-38

c. <u>No R</u>

% decrease in DF=[(Railing-NoR)/NoR]x100 (steel + slab) (4 lanes)																	
L (ft)	s (ft)	No	RL					RR					2R				
			x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	1	1	1	1	1	0	0	0	0	0	0	1	1	1	1
	8	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
	12	0	0	0	1	1	1	0	0	0	0	0	0	0	1	1	1
60	6	0	0	0	-1	-1	-1	0	0	0	0	0	0	0	0	0	-1
	8	0	-1	-1	-1	-1	-1	0	0	0	0	0	-1	-1	-1	-1	-1
	12	0	0	0	0	-1	-1	0	0	0	0	0	-1	-1	-1	-1	-1
80	6	0	-2	-3	-4	-5	-6	0	0	0	0	0	-2	-3	-4	-5	-5
	8	0	-4	-5	-6	-6	-6	0	0	0	0	0	-4	-6	-7	-7	-8
	12	0	-4	-5	-6	-7	-7	0	0	0	0	0	-5	-7	-8	-9	-10
100	6	0	-6	-8	-11	-12	-13	0	0	1	1	1	-6	-9	-12	-14	-15
	8	0	-7	-10	-12	-13	-14	0	1	1	1	1	-7	-11	-15	-17	-18
	12	0	-9	-11	-13	-15	-16	0	0	-1	-1	-1	-10	-15	-19	-21	-22
120	6	0	-10	-14	-19	-20	-21	1	1	2	2	2	-10	-15	-21	-24	-26
	8	0	-11	-16	-19	-21	-22	0	1	1	1	1	-12	-17	-24	-28	-30
	12	0	-11	-15	-19	-22	-23	-1	-1	-2	-2	-2	-12	-20	-28	-32	-35

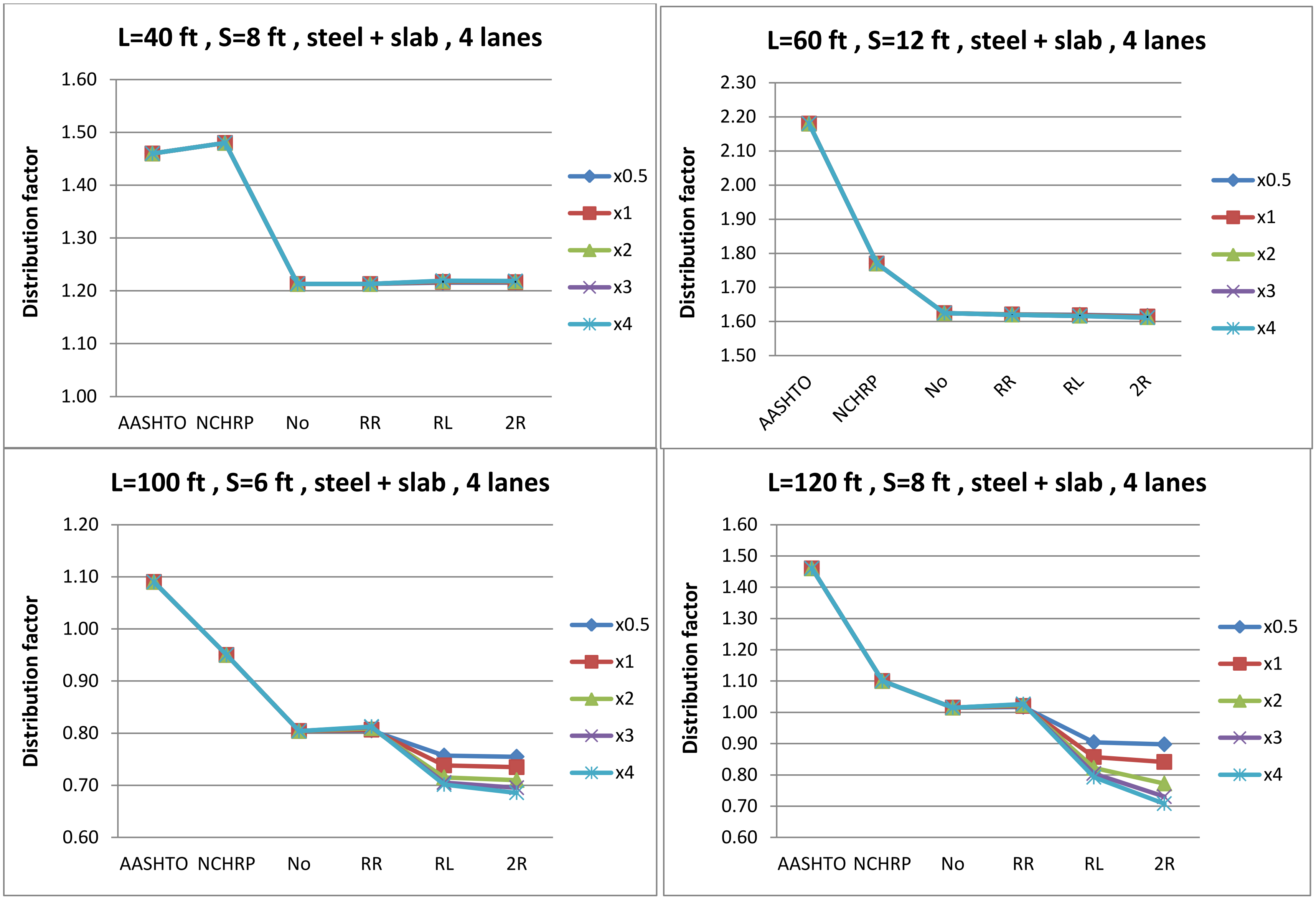


Figure 4.23. Sensitivity of Distribution Factor to Railing Stiffness for Interior Girders (Steel + Slab, 4-lane bridges)

Table 4.25. Maximum Bending Moments and Wheel Load Distribution Factors in All Steel Girders (Steel only) [4-lane Bridges]

a. Maximum bending moment (kip .ft)

Maximum bending moment Mmax (kip.ft) (steel only) (4 lanes)																		
L (ft)	s (ft)	M0 (kip-ft)	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	224.8	261.4	262.7	263.1	263.5	263.7	263.8	261.4	261.4	261.4	261.4	261.3	262.6	263.1	263.5	263.7	263.7
	8	224.8	330.4	330.9	331.4	331.8	331.9	332.0	330.3	330.3	330.3	330.3	330.3	330.8	331.2	331.6	331.8	331.9
	12	224.8	467.2	468.6	469.2	469.6	469.8	469.9	467.2	467.2	467.3	467.3	467.3	468.6	469.2	469.6	469.8	469.9
60	6	403.2	449.0	446.9	446.0	445.1	444.7	444.4	449.1	449.3	449.5	449.6	449.6	446.9	446.2	445.6	445.2	445.0
	8	403.2	563.5	554.0	554.0	554.6	554.9	555.0	563.8	564.1	564.4	564.6	564.7	554.2	553.6	554.3	554.7	554.9
	12	403.2	769.9	766.2	764.9	763.6	763.0	762.7	767.9	767.5	767.1	766.9	766.8	764.1	762.4	760.7	759.9	759.5
80	6	582.4	609.5	595.4	587.4	578.6	573.8	570.7	610.0	610.7	611.6	612.1	612.4	595.3	587.6	579.1	574.5	571.5
	8	582.4	767.5	736.0	721.0	715.6	712.7	710.9	767.2	767.6	768.1	768.3	768.4	735.6	718.5	706.6	702.4	699.8
	12	582.4	1047.5	988.6	975.9	962.8	956.1	952.0	1047.2	1047.8	1048.4	1048.7	1048.9	975.8	956.8	936.8	926.3	919.8
100	6	825.4	842.7	788.5	766.9	740.9	727.9	723.2	843.6	845.5	849.0	851.0	852.3	785.6	762.4	733.8	716.9	705.8
	8	825.4	1063.2	976.3	936.5	913.7	901.9	894.3	1066.4	1069.2	1072.4	1074.2	1075.4	971.5	928.5	877.4	855.8	843.0
	12	825.4	1434.9	1286.7	1251.5	1212.4	1191.2	1177.8	1429.1	1426.9	1424.5	1423.2	1422.3	1277.0	1194.0	1128.2	1091.4	1067.9
120	6	1163.5	1206.2	1065.6	1009.9	955.4	934.8	922.0	1218.5	1228.1	1240.4	1247.9	1252.9	1064.1	996.3	927.4	885.8	857.1
	8	1163.5	1504.4	1307.6	1234.5	1179.0	1150.4	1135.1	1521.2	1533.9	1549.4	1558.6	1564.6	1306.5	1208.4	1100.8	1035.2	998.0
	12	1163.5	2006.6	1732.6	1632.9	1550.0	1503.0	1477.3	2026.5	2040.9	2057.8	2067.4	2073.6	1709.7	1545.6	1365.3	1279.7	1223.3

b. Distribution factor

				Distribution factor DF=Mmax/M0 (steel only) (4 lanes)															
L (ft)	s (ft)	AASHTO	NCHRP	No	RL					RR					2R				
					x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	1.09	1.2	0.87	0.88	0.88	0.88	0.88	0.88	0.87	0.87	0.87	0.87	0.87	0.88	0.88	0.88	0.88	0.88
	8	1.46	1.48	1.10	1.10	1.11	1.11	1.11	1.11	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.11	1.11	1.11
	12	2.18	1.98	1.56	1.56	1.57	1.57	1.57	1.57	1.56	1.56	1.56	1.56	1.56	1.56	1.57	1.57	1.57	1.57
60	6	1.09	1.08	0.84	0.83	0.83	0.83	0.83	0.83	0.84	0.84	0.84	0.84	0.84	0.83	0.83	0.83	0.83	0.83
	8	1.46	1.32	1.05	1.03	1.03	1.03	1.03	1.03	1.05	1.05	1.05	1.05	1.05	1.03	1.03	1.03	1.03	1.03
	12	2.18	1.77	1.43	1.43	1.42	1.42	1.42	1.42	1.43	1.43	1.43	1.43	1.43	1.42	1.42	1.42	1.41	1.41
80	6	1.09	1.01	0.78	0.77	0.76	0.75	0.74	0.73	0.79	0.79	0.79	0.79	0.79	0.77	0.76	0.75	0.74	0.74
	8	1.46	1.23	0.99	0.95	0.93	0.92	0.92	0.92	0.99	0.99	0.99	0.99	0.99	0.95	0.93	0.91	0.90	0.90
	12	2.18	1.64	1.35	1.27	1.26	1.24	1.23	1.23	1.35	1.35	1.35	1.35	1.35	1.26	1.23	1.21	1.19	1.18
100	6	1.09	0.95	0.77	0.72	0.70	0.67	0.66	0.66	0.77	0.77	0.77	0.77	0.77	0.71	0.69	0.67	0.65	0.64
	8	1.46	1.16	0.97	0.89	0.85	0.83	0.82	0.81	0.97	0.97	0.97	0.98	0.98	0.88	0.84	0.80	0.78	0.77
	12	2.18	1.54	1.30	1.17	1.14	1.10	1.08	1.07	1.30	1.30	1.29	1.29	1.29	1.16	1.08	1.03	0.99	0.97
120	6	1.09	0.91	0.78	0.69	0.65	0.62	0.60	0.59	0.79	0.79	0.80	0.80	0.81	0.69	0.64	0.60	0.57	0.55
	8	1.46	1.1	0.97	0.84	0.80	0.76	0.74	0.73	0.98	0.99	1.00	1.00	1.01	0.84	0.78	0.71	0.67	0.64
	12	2.18	1.47	1.29	1.12	1.05	1.00	0.97	0.95	1.31	1.32	1.33	1.33	1.34	1.10	1.00	0.88	0.82	0.79

Table 4.26. Comparison of FEA Distribution Factors in All Steel girders (Steel only)[4-lane bridges] with AASHTO (2002) , NCHRP 12-26 and FEA Distribution Factors of the No Railing Case

a. <u>AASHTO</u>

% decrease in DF=[(FEA-AASHTO)/AASHTO]x100 (steel only) (4 lanes)																		
L (ft)	s (ft)	AASHTO	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-20	-20	-19	-19	-19	-19	-20	-20	-20	-20	-20	-20	-19	-19	-19	-19
	8	0	-25	-24	-24	-24	-24	-24	-25	-25	-25	-25	-25	-24	-24	-24	-24	-24
	12	0	-29	-28	-28	-28	-28	-28	-29	-29	-29	-28	-28	-28	-28	-28	-28	-28
60	6	0	-23	-24	-24	-24	-24	-24	-23	-23	-23	-23	-23	-24	-24	-24	-24	-24
	8	0	-28	-29	-29	-29	-29	-29	-28	-28	-28	-28	-28	-29	-29	-29	-29	-29
	12	0	-34	-35	-35	-35	-35	-35	-34	-35	-35	-35	-35	-35	-35	-35	-35	-35
80	6	0	-28	-30	-31	-32	-32	-33	-28	-28	-28	-28	-28	-30	-31	-32	-32	-32
	8	0	-32	-35	-36	-37	-37	-37	-32	-32	-32	-32	-32	-35	-37	-38	-38	-38
	12	0	-38	-42	-42	-43	-44	-44	-38	-38	-38	-38	-38	-42	-43	-45	-45	-46
100	6	0	-30	-34	-36	-38	-39	-40	-30	-30	-29	-29	-29	-35	-36	-39	-40	-41
	8	0	-34	-39	-42	-43	-44	-44	-34	-33	-33	-33	-33	-40	-42	-45	-47	-48
	12	0	-40	-46	-48	-49	-50	-51	-40	-41	-41	-41	-41	-47	-50	-53	-55	-55
120	6	0	-29	-37	-40	-43	-45	-45	-28	-27	-27	-26	-26	-37	-41	-45	-48	-49
	8	0	-34	-42	-45	-48	-49	-50	-33	-32	-32	-31	-31	-42	-47	-51	-54	-56
	12	0	-41	-49	-52	-54	-56	-56	-40	-40	-39	-39	-39	-49	-54	-60	-62	-64

b. NCHRP

% decrease in DF=[(FEA-NCHRP)/NCHRP]x100 (steel only) (4 lanes)																		
L (ft)	s (ft)	NCHRP	No	RL					RR					2R				
				x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	-27	-27	-27	-27	-27	-27	-27	-27	-27	-27	-27	-27	-27	-27	-27	-27
	8	0	-26	-25	-25	-25	-25	-25	-26	-26	-26	-26	-26	-25	-25	-25	-25	-25
	12	0	-21	-21	-21	-21	-21	-21	-21	-21	-21	-21	-21	-21	-21	-21	-21	-21
60	6	0	-23	-23	-23	-23	-23	-23	-23	-23	-23	-23	-23	-23	-23	-23	-23	-23
	8	0	-21	-22	-22	-22	-22	-22	-21	-20	-20	-20	-20	-22	-22	-22	-22	-22
	12	0	-19	-19	-20	-20	-20	-20	-19	-19	-19	-19	-19	-20	-20	-20	-20	-20
80	6	0	-22	-24	-25	-26	-27	-27	-22	-22	-22	-22	-22	-24	-25	-26	-27	-27
	8	0	-20	-23	-25	-25	-25	-26	-20	-20	-20	-20	-20	-23	-25	-26	-26	-27
	12	0	-18	-22	-23	-24	-25	-25	-18	-18	-18	-18	-18	-23	-25	-26	-27	-28
100	6	0	-19	-25	-27	-29	-30	-31	-19	-19	-19	-19	-18	-25	-27	-30	-31	-32
	8	0	-17	-24	-27	-28	-29	-30	-16	-16	-16	-16	-16	-24	-27	-31	-33	-34
	12	0	-15	-24	-26	-28	-30	-31	-16	-16	-16	-16	-16	-25	-30	-33	-36	-37
120	6	0	-15	-25	-28	-32	-34	-35	-14	-13	-12	-12	-11	-25	-29	-34	-37	-39
	8	0	-12	-23	-28	-31	-33	-33	-11	-10	-9	-9	-8	-23	-29	-35	-39	-42
	12	0	-12	-24	-28	-32	-34	-35	-11	-11	-10	-9	-9	-25	-32	-40	-44	-46

c. <u>No R</u>

% decrease in DF=[(Railing-NoR)/NoR]x100 (steel only) (4 lanes)																	
L (ft)	s (ft)	No	RL					RR					2R				
			x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
40	6	0	0	1	1	1	1	0	0	0	0	0	0	1	1	1	1
	8	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
	12	0	0	0	1	1	1	0	0	0	0	0	0	0	1	1	1
60	6	0	0	-1	-1	-1	-1	0	0	0	0	0	0	-1	-1	-1	-1
	8	0	-2	-2	-2	-2	-2	0	0	0	0	0	-2	-2	-2	-2	-2
	12	0	0	-1	-1	-1	-1	0	0	0	0	0	-1	-1	-1	-1	-1
80	6	0	-2	-4	-5	-6	-6	0	0	0	0	0	-2	-4	-5	-6	-6
	8	0	-4	-6	-7	-7	-7	0	0	0	0	0	-4	-6	-8	-8	-9
	12	0	-6	-7	-8	-9	-9	0	0	0	0	0	-7	-9	-11	-12	-12
100	6	0	-6	-9	-12	-14	-14	0	0	1	1	1	-7	-10	-13	-15	-16
	8	0	-8	-12	-14	-15	-16	0	1	1	1	1	-9	-13	-17	-20	-21
	12	0	-10	-13	-16	-17	-18	0	-1	-1	-1	-1	-11	-17	-21	-24	-26
120	6	0	-12	-16	-21	-22	-24	1	2	3	3	4	-12	-17	-23	-27	-29
	8	0	-13	-18	-22	-24	-25	1	2	3	4	4	-13	-20	-27	-31	-34
	12	0	-14	-19	-23	-25	-26	1	2	3	3	3	-15	-23	-32	-36	-39

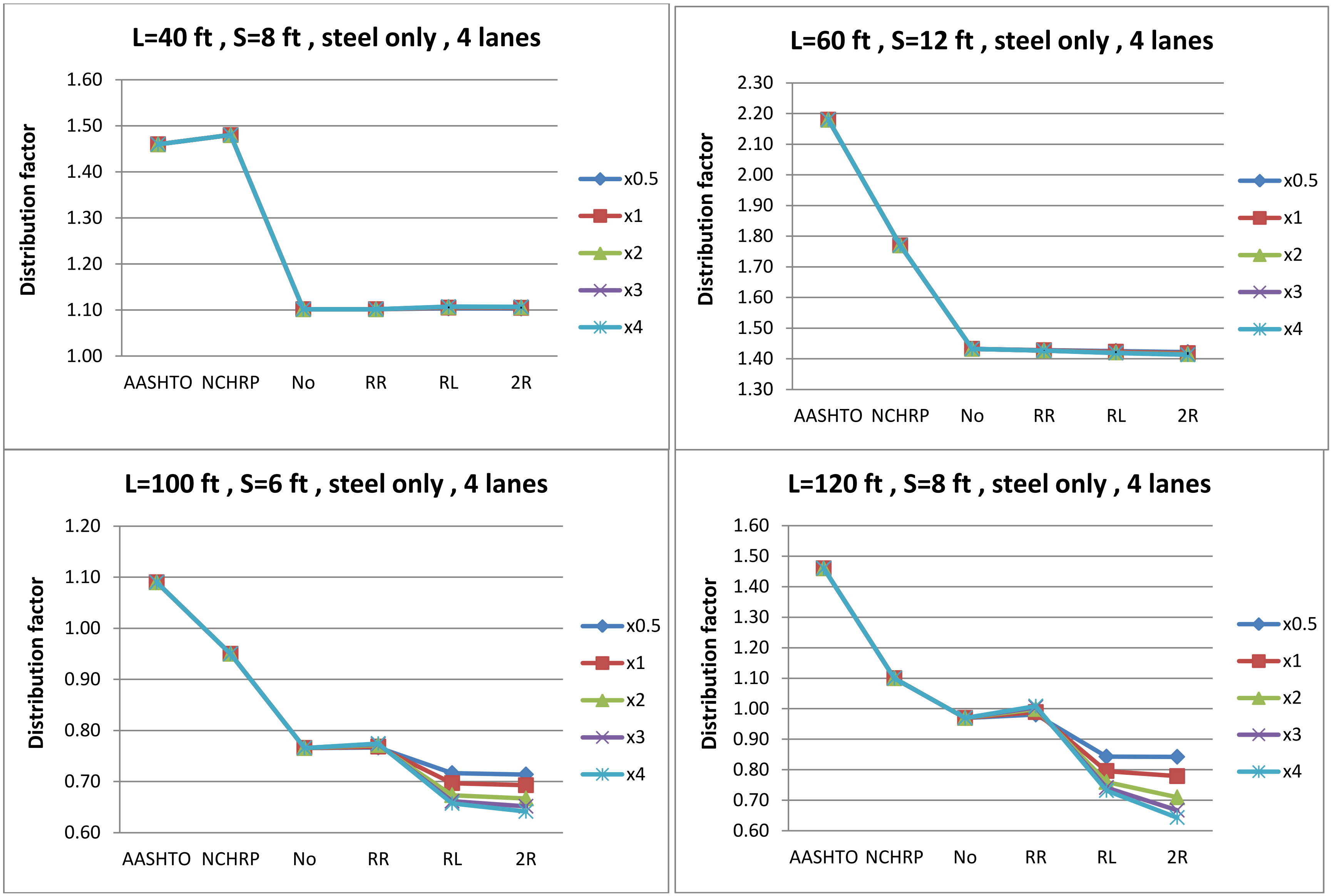


Figure 4.24. Sensitivity of Distribution Factor to Railing Stiffness for Interior and exterior Girders (Steel only, 4-lane bridges)

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.b is used to extract the following general conclusions for interior girders when introducing railings to a bridge deck:

1. No Railings: the FEA distribution factors are smaller than (1) by about 12 % for all spans.
2. Railing on one side (left or right):For spans up to 80 ft (1) is higher than FEA distribution factors by and about 16 % (for any railing stiffness) and for spans between 80 and 120 ft it is higher by about 12 % (for any railing stiffness).
3. Railing on both sides: For spans up to 80 ft (1) is higher than FEA distribution factors by about 16 % (for any railing stiffness) and for spans between 80 and 120 ft it is higher by 22% to 40 % (depending on railing stiffness).

A conservative comparison of FEA distribution factors for interior girders was also performed. The reference base selected was the distribution factors obtained from the FEA models without railings. The maximum FEA distribution factors were calculated for all bridge cases after introducing railings to the bridge deck and compared to the distribution factors of the No R case as shown in Table 4.24.c. First of all, it's important to note that for spans up to 60 ft, the addition of railings has a negligible effect on the distribution factor .Otherwise, the average reductions in FEA distribution factors when compared to the base case were; 0 % when introducing railings on one side; 4% to 10% (depending on railing stiffness) for spans between

60 and 80 ft (18 and 24 m) and 8 % to 20 % (depending on railing stiffness) for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on both sides. The finite-element results show the effects of stiffness of 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 increasing the stiffness of railings in a bridge superstructure.

4.3.2. Influence on Bridges with Reduced-Stiffness Girders

This section aims at studying the influence of stiffness of railings on the maximum bending moment of steel girder bridges provided that we have a girder less stiff than the original girder. For that , we studied 2 cases of bridges:

L=40 ft, 3 lanes, S = 8 ft , 2R , with all (x0.5 , x1 , x2 , x3 , x4 railings) [5 bridge cases].

L=80 ft, 3 lanes, S = 8 ft , 2R , with all (x0.5 , x1 , x2 , x3 , x4 railings)[5 bridge cases].

For each of the 10 cases we compute the bending moments at the critical section using 2 girder stiffnesses , one of which is the original girder W36x160 , and the other also the W36x160 but with a reduction factor on the moment of inertia of 1/3 for the girders of the 40 ft bridge , and 2/3 for the girders of the 80 ft bridge.

Tables 4.27 and 4.28 show the calculated Bending Moments (kip-ft) at the critical Section Based on FEA Results (3 lanes, Span L=40 ft, Spacing S = 8 ft , 2R) using original Girders and reduced stiffness girders , respectively.

We can notice the difference in bending moment distribution in the 2 cases from tables 4.27 and 4.28. Reducing the stiffness of girders implies a reduced bending moment in the girders, and an increase in bending moment in the slab and railing. But of course, the total moment on the bridge doesn't change since the loads and the span length are the same.

L=40 ft:

Table 4.27. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (3 lanes, Span L=40 ft, Spacing S = 8 ft , 2R , original Girder)

Bending Moments (kip.ft) (3 Lanes, L=40 ft, S = 8 ft, 2R, Original Girder)									
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Total	Max
No R	Girder	92.1	238.4	321.9	309.6	212.9	54.7	1229.4	321.9
	Slab	6.8	27.8	30.2	33.5	17.7	3.3	119.4	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	98.9	266.2	352.1	343.1	230.6	58.1	1348.8	352.1
x0.5	Girder	77.6	232.5	321.3	309.4	209.6	49.6	1200.0	321.3
	Slab	5.8	27.7	30.3	33.6	17.6	2.7	117.6	
	Railling	21.3	0.0	0.0	0.0	0.0	9.9	31.2	
	Total	104.7	260.2	351.7	342.9	227.2	62.2	1348.8	351.7
x1	Girder	69.1	230.5	321.5	309.7	208.7	45.7	1185.2	321.5
	Slab	5.6	27.7	30.4	33.6	17.6	2.6	117.5	
	Railling	31.5	0.0	0.0	0.0	0.0	14.6	46.1	
	Total	106.3	258.2	351.9	343.3	226.3	62.9	1348.8	351.9
x2	Girder	60.9	228.6	321.6	309.9	207.9	42.0	1170.9	321.6
	Slab	5.4	27.8	30.5	33.7	17.6	2.5	117.5	
	Railling	41.4	0.0	0.0	0.0	0.0	19.1	60.5	
	Total	107.7	256.3	352.1	343.6	225.5	63.6	1348.8	352.1
x3	Girder	56.9	227.7	321.7	310.1	207.5	40.2	1163.9	321.7
	Slab	5.3	27.8	30.5	33.7	17.6	2.5	117.4	
	Railling	46.2	0.0	0.0	0.0	0.0	21.3	67.5	
	Total	108.4	255.4	352.2	343.8	225.2	63.9	1348.8	352.2
x4	Girder	54.5	227.1	321.7	310.1	207.3	39.1	1159.8	321.7
	Slab	5.3	27.8	30.6	33.7	17.6	2.4	117.4	
	Railling	49.0	0.0	0.0	0.0	0.0	22.6	71.6	
	Total	108.8	254.9	352.2	343.9	224.9	64.1	1348.8	352.2

Table 4.28. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (3 lanes, Span L=40 ft, Spacing S = 8 ft , 2R , x(1/3) Girder)

Bending Moments (kip.ft) (3 Lanes, L=40 ft, S = 8 ft, x(1/3) Girder)									
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Total	Max
No R	Girder	124.5	208.4	259.3	249.8	188.6	91.5	1122.2	259.3
	Slab	17.3	48.2	55.5	58.0	36.0	11.7	226.7	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	141.8	256.6	314.8	307.8	224.6	103.2	1348.8	314.8
x0.5	Girder	78.4	187.8	252.7	244.8	174.2	60.0	997.9	252.7
	Slab	12.4	46.4	55.8	58.4	34.8	8.1	215.9	
	Railling	78.9	0.0	0.0	0.0	0.0	56.2	135.1	
	Total	169.7	234.2	308.5	303.2	208.9	124.4	1348.8	308.5
x1	Girder	63.3	181.8	251.0	243.5	170.0	49.4	959.1	251.0
	Slab	11.1	46.0	55.9	58.6	34.5	7.2	213.3	
	Railling	103.0	0.0	0.0	0.0	0.0	73.4	176.5	
	Total	177.5	227.8	306.9	302.1	204.5	130.1	1348.8	306.9
x2	Girder	51.7	177.3	249.7	242.5	166.8	41.1	929.1	249.7
	Slab	10.1	45.7	56.0	58.7	34.3	6.5	211.3	
	Railling	121.6	0.0	0.0	0.0	0.0	86.8	208.4	
	Total	183.4	222.9	305.7	301.2	201.2	134.4	1348.8	305.7
x3	Girder	46.8	175.4	249.1	242.1	165.5	37.7	916.6	249.1
	Slab	9.7	45.6	56.1	58.8	34.2	6.2	210.5	
	Railling	129.4	0.0	0.0	0.0	0.0	92.4	221.7	
	Total	185.9	220.9	305.2	300.9	199.8	136.3	1348.8	305.2
x4	Girder	44.2	174.3	248.8	241.9	164.8	35.8	909.7	248.8
	Slab	9.4	45.5	56.1	58.8	34.2	6.1	210.1	
	Railling	133.6	0.0	0.0	0.0	0.0	95.4	229.1	
	Total	187.2	219.8	304.9	300.7	199.0	137.3	1348.8	304.9

Tables 4.27 and 4.28 are used to extract the maximum bending moments in the critical girders for the studied bridge case (3 lanes , 40 ft span , 8 ft girder spacing , railings on both sides) and thus FEA distribution factors can be computed and compared to AASHTO (2002) and NCHRP 12-26 and the difference in distribution factor with respect to the No Railing case. This is done while considering the effect of the slab (Steel + slab) ; which means that the critical girder is interior , and while neglecting the effect of the slab ; which means that the critical girder is either interior or exterior.

a.Steel + Slab:

Table 4.29. Maximum Bending Moments in Interior Steel Girders (3 lanes, L=40 ft, S =8 ft , 2 R , Steel + slab)

(3 Lanes, L=40 ft, S = 8 ft, 2R ,Steel + Slab)							
		Maximum moment (kip-ft)					
	M0	No R	x0.5	x1	x2	x3	x4
x (1/3) Girder	224.8	314.8	308.5	306.9	305.7	305.2	304.9
Orignal Girder	224.8	352.1	351.7	351.9	352.1	352.2	352.2

Table 4.30. Comparison of FEA Distribution Factors in Interior Steel girders (3 lanes, L=40 ft, S =8 ft, 2 R , Steel + slab) with AASHTO (2002) and NCHRP 12-26

DF=0.9*Mmax/M0 (3 lanes, L=40 ft, S = 8 ft, 2R ,Steel + Slab)								
	AASHTO	NCHRP	No R	x0.5	x1	x2	x3	x4
x (1/3) Girder	1.46	1.48	1.26	1.23	1.23	1.22	1.22	1.22
Orignal Girder	1.46	1.48	1.41	1.41	1.41	1.41	1.41	1.41

Table 4.31. Percentage Decrease in Distribution Factor (3 lanes, L=40 ft, S =8 ft , 2 R , Steel + slab) with respect to No Railing Case

% decrease in DF=[(FEA-NoR)/NoR]x100 (steel+slab) (3 lanes, s=8ft, L=40ft , 2R)					
	x0.5	x1	x2	x3	x4
x (1/3) Girder	-2.0	-2.5	-2.9	-3.0	-3.1
Orignal Girder	-0.1	0.0	0.0	0.0	0.1

b.Steel only:

Table 4.32. Maximum Bending Moments in All Steel Girders (3 lanes, L=40 ft, S =8 ft , 2 R , Steel only)

(3 Lanes, L=40 ft, S = 8 ft, 2R ,Steel only)							
		Maximum moment (kip-ft)					
	M0	No R	x0.5	x1	x2	x3	x4
x (1/3) Girder	224.8	259.3	252.7	251.0	249.7	249.1	248.8
Orignal Girder	224.8	321.9	321.3	321.5	321.6	321.7	321.7

Table 4.33. Comparison of FEA Distribution Factors in All Steel girders (3 lanes, L=40 ft, S=8ft, 2 R , Steel Only) with AASHTO (2002) and NCHRP 12-26

DF=0.9*Mmax/M0 (3 lanes, L=40 ft, S = 8 ft, 2R ,Steel only)								
	AASHTO	NCHRP	No R	x0.5	x1	x2	x3	x4
x (1/3) Girder	1.46	1.48	1.04	1.01	1.00	1.00	1.00	1.00
Orignal Girder	1.46	1.48	1.29	1.29	1.29	1.29	1.29	1.29

Table 4.34. Percentage decrease in distribution factor (3 lanes, L=40 ft, S =8 ft , 2 R , Steel only) with respect to the No Railing Case

% decrease in DF=[(FEA-NoR)/NoR]x100 (steel only) (3 lanes, s=8ft, L=40ft, 2R)					
	x0.5	x1	x2	x3	x4
x (1/3) Girder	-2.5	-3.2	-3.7	-3.9	-4.1
Orignal Girder	-0.2	-0.1	-0.1	-0.1	-0.1

Tables 4.31 and 4.34 show the % decrease in the Distribution factor of a 3-lane , 40ft span , 8 ft girder spacing , 2-Railing bridge with respect to the reference case (No railing) when varying the stiffness of railing and the stiffness of girders. For the case of bridge with original girder no influence of increasing the stiffness of railings on the distribution factors , whether we considered the effect of the slab or we neglected it. Whereas, when reducing the stiffness of the girder by a factor of 1/3 , the % decrease in distribution factor ranged from 2% to 3% when

considering the effect of slab (Steel + Slab), and from 2.5% to 4% when neglecting it (Steel only).

The following charts (Figures 4.25 and 4.26) represent a comparison between the distribution factors of the bridges with original girders , and with reduced stiffness girders.

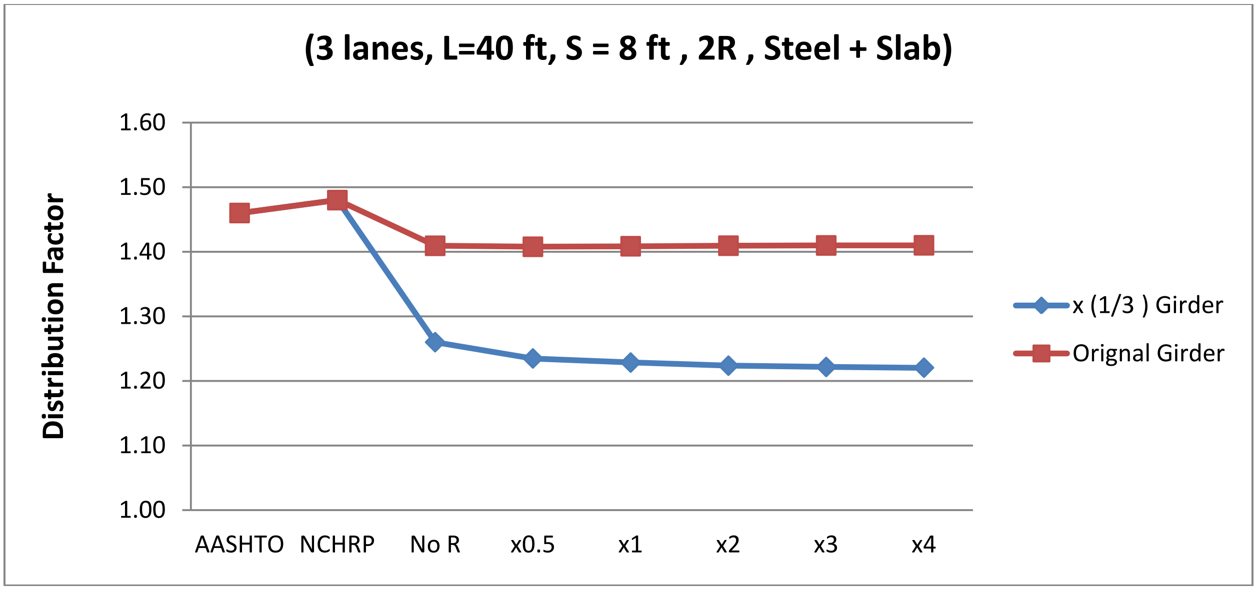


Figure 4.25 : Sensitivity of Distribution Factor to Railing Stiffness in Reduced-Stiffness Girder Bridges (3 lanes, L= 40 ft, S = 8 ft , 2 R , Steel + Slab)

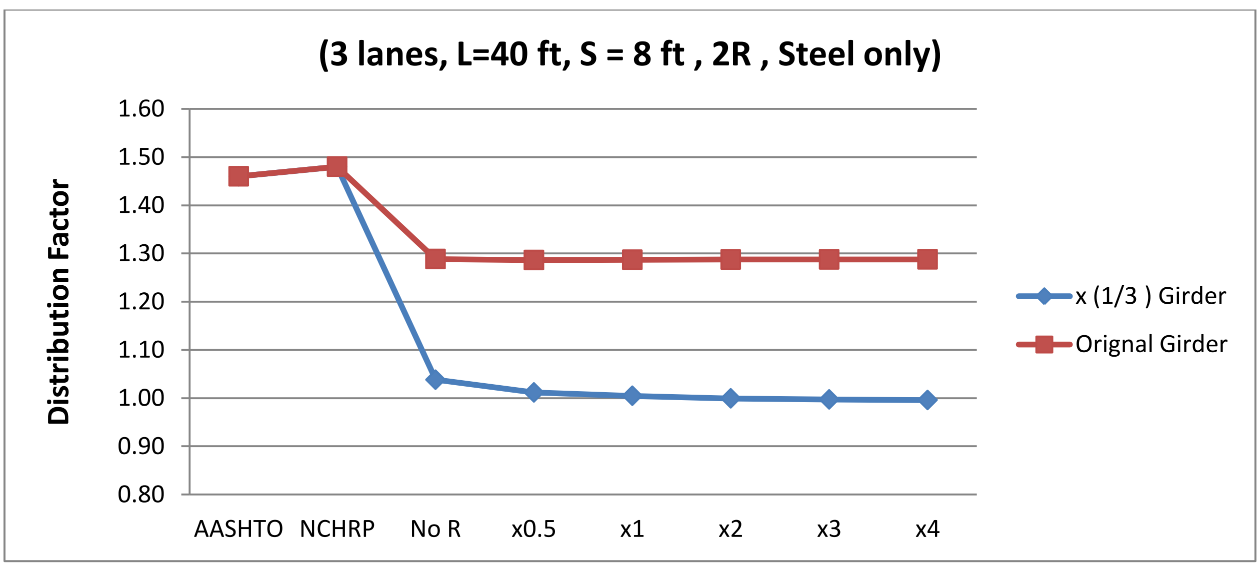


Figure 4.26 : Sensitivity of Distribution Factor to Railing Stiffness in Reduced-Stiffness Girder Bridges (3 lanes, L= 40 ft, S = 8 ft , 2 R , Steel only)

L=80 ft:

In a similar manner to the 40 ft case we extract the results for the 80 ft case (3 lanes , 40 ft span , 8 ft girder spacing , railings on both sides) and the results are presented as before.

Bending Moments (kip.ft) (3 Lanes, L=80 ft, S = 8 ft, 2R, Original Girder)									
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Total	Max
NoR	Girder	464.2	591.4	658.5	636.8	532.7	370.5	3254.0	658.5
	Slab	21.0	50.4	56.2	58.9	38.4	15.6	240.4	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	485.3	641.8	714.7	695.6	571.1	386.0	3494.4	714.7
x0.5	Girder	380.7	538.5	625.4	607.2	491.9	309.9	2953.8	625.4
	Slab	18.0	49.5	56.8	59.6	37.7	13.2	234.9	
	Railling	170.5	0.0	0.0	0.0	0.0	135.3	305.8	
	Total	569.2	588.0	682.2	666.8	529.7	458.5	3494.4	682.2
x1	Girder	325.3	506.1	605.9	589.8	466.3	267.4	2761.0	605.9
	Slab	16.5	49.2	57.5	60.3	37.7	12.2	233.4	
	Railling	277.9	0.0	0.0	0.0	0.0	222.1	500.0	
	Total	619.8	555.4	663.4	650.1	504.0	501.7	3494.4	663.4
x2	Girder	259.1	467.6	582.8	569.0	435.4	215.6	2529.4	582.8
	Slab	14.8	49.0	58.3	61.1	37.6	10.8	231.7	
	Railling	406.2	0.0	0.0	0.0	0.0	327.1	733.4	
	Total	680.1	516.5	641.0	630.1	473.0	553.6	3494.4	641.0
x3	Girder	220.9	445.4	569.4	556.9	417.4	185.4	2395.2	569.4
	Slab	13.8	48.8	58.7	61.6	37.6	10.1	230.7	
	Railling	480.2	0.0	0.0	0.0	0.0	388.4	868.6	
	Total	714.9	494.2	628.1	618.5	455.0	583.8	3494.4	628.1
x4	Girder	196.0	431.0	560.7	549.0	405.6	165.5	2307.7	560.7
	Slab	13.2	48.7	59.0	61.9	37.6	9.6	230.0	
	Railling	528.3	0.0	0.0	0.0	0.0	428.4	956.7	
	Total	737.4	479.7	619.8	610.9	443.1	603.5	3494.4	619.8

Table 4.35. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (3 lanes, Span L=80 ft, Spacing S = 6 ft , 2R , Original Girder)

Table 4.36. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (3 lanes, Span L=80 ft, Spacing S = 6 ft , 2R , x(2/3) Girder)

Bending Moments (kip.ft) (3 Lanes, L=80 ft, S = 8 ft, 2R , x(2/3) Girder)									
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Total	Max
NoR	Girder	481.4	571.3	618.1	598.1	518.9	395.9	3183.8	618.1
	Slab	30.3	63.7	70.1	72.4	50.6	23.6	310.7	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	511.7	635.0	688.2	670.6	569.5	419.5	3494.4	688.2
x0.5	Girder	367.9	494.4	563.9	548.2	456.1	308.5	2739.0	563.9
	Slab	24.6	61.0	70.0	72.5	48.5	19.1	295.6	
	Railling	252.1	0.0	0.0	0.0	0.0	207.7	459.8	
	Total	644.6	555.4	633.9	620.7	504.5	535.4	3494.4	633.9
x1	Girder	302.2	451.5	534.1	520.5	420.1	255.5	2484.0	534.1
	Slab	21.8	59.7	70.2	72.7	47.6	16.9	289.0	
	Railling	394.4	0.0	0.0	0.0	0.0	327.1	721.5	
	Total	718.4	511.3	604.2	593.3	467.7	599.6	3494.4	604.2
x2	Girder	230.3	404.6	501.4	490.1	380.3	196.7	2203.4	501.4
	Slab	18.7	58.4	70.4	73.0	46.7	14.4	281.6	
	Railling	550.0	0.0	0.0	0.0	0.0	459.3	1009.4	
	Total	799.1	463.1	571.8	563.1	427.0	670.4	3494.4	571.8
x3	Girder	191.8	379.5	483.9	473.7	358.8	164.7	2052.5	483.9
	Slab	17.1	57.7	70.5	73.2	46.1	13.1	277.7	
	Railling	633.5	0.0	0.0	0.0	0.0	530.8	1164.3	
	Total	842.3	437.2	554.4	546.9	405.0	708.7	3494.4	554.4
x4	Girder	167.8	363.9	473.0	463.4	345.4	144.7	1958.2	473.0
	Slab	16.1	57.3	70.5	73.3	45.8	12.3	275.2	
	Railling	685.4	0.0	0.0	0.0	0.0	575.6	1261.0	
	Total	869.3	421.1	543.5	536.7	391.2	732.6	3494.4	543.5

a. Steel + Slab :

Table 4.37. Maximum Bending Moments in Interior Steel Girders (3 lanes, L=80 ft, S =8 ft, 2R, Steel + Slab)

(3 Lanes, L=80 ft, S = 8 ft, 2R ,Steel + Slab)							
		Maximum moment (kip-ft)					
	M0	No R	x0.5	x1	x2	x3	x4
x (2/3) Girder	582.4	688.2	633.9	604.2	571.8	554.4	543.5
Orignal Girder	582.4	714.7	682.2	663.4	641.0	628.1	619.8

Table 4.38. Comparison of FEA Distribution Factors in Interior Steel girders (3 lanes, L=80 ft, S=8 ft, 2 R , Steel + slab) with AASHTO (2002) and NCHRP 12-26

DF=0.9*Mmax/M0 (3 lanes, L=80 ft, S = 8 ft, 2R ,Steel + Slab)								
	AASHTO	NCHRP	No R	x0.5	x1	x2	x3	x4
x (2/3) Girder	1.46	1.23	1.06	0.98	0.93	0.88	0.86	0.84
Orignal Girder	1.46	1.23	1.10	1.05	1.03	0.99	0.97	0.96

Table 4.39. Percentage Decrease in Distribution Factor (3 lanes, L=80 ft, S =8 ft , 2R , Steel + Slab)with respect to the No Railing Case

% decrease in DF=[(FEA-NoR)/NoR]x100 (steel + slab) (3 lanes, s=8ft, L=80ft, 2R)					
	x0.5	x1	x2	x3	x4
x (2/3) Girder	-7.9	-12.2	-16.9	-19.4	-21.0
Orignal Girder	-4.5	-7.2	-10.3	-12.1	-13.3

b.<u>Steel only</u> :

Table 4.40. Maximum Bending Moments in All Steel Girders (3 lanes, L=80 ft, S =8 ft, 2R, Steel Only)

(3 Lanes, L=80 ft, S = 8 ft, 2R ,Steel only)							
		Maximum moment (kip-ft)					
	M0	No R	x0.5	x1	x2	x3	x4
x (2/3) Girder	582.4	618.1	563.9	534.1	501.4	483.9	473.0
Orignal Girder	582.4	658.5	625.4	605.9	582.8	569.4	560.7

Table 4.41. Comparison of FEA Distribution Factors in All Steel Girders (3 lanes, L=40 ft, S =8 ft, 2 R , Steel Only) with AASHTO (2002) and NCHRP 12-26

DF=0.9*Mmax/M0 (3 lanes, L=80 ft, S = 8 ft, 2R ,Steel only)								
	AASHTO	NCHRP	No R	x0.5	x1	x2	x3	x4
x (2/3) Girder	1.46	1.23	0.96	0.87	0.83	0.77	0.75	0.73
Orignal Girder	1.46	1.23	1.02	0.97	0.94	0.90	0.88	0.87

Table 4.42. Percentage Decrease in Distribution Factor (3 lanes, L=80 ft, S =8 ft , 2 R , Steel Only) with Respect to the No Railing Case

% decrease in DF=[(FEA-NoR)/NoR]x100 (steel only) (3 lanes, s=8ft, L=80ft, 2R)					
	x0.5	x1	x2	x3	x4
x (2/3) Girder	-8.8	-13.6	-18.9	-21.7	-23.5
Orignal Girder	-5.0	-8.0	-11.5	-13.5	-14.8

Tables 4.39 and 4.40 show the % decrease in the Distribution factor of a 3-lane , 80ft span , 8 ft girder spacing , 2-Railing bridge with respect to the reference case (No railing) when varying the stiffness of railing and the stiffness of girders. For the case of bridge with original girder , the % decrease in distribution factor ranged from 4.5% to13% when considering the effect of slab , and from 5% to 15% when neglecting it. Whereas, when reducing the stiffness of the girder, the % decrease in distribution factor ranged from 8% to 21% when considering the effect of slab , and from 9% to 23.5% when neglecting it.

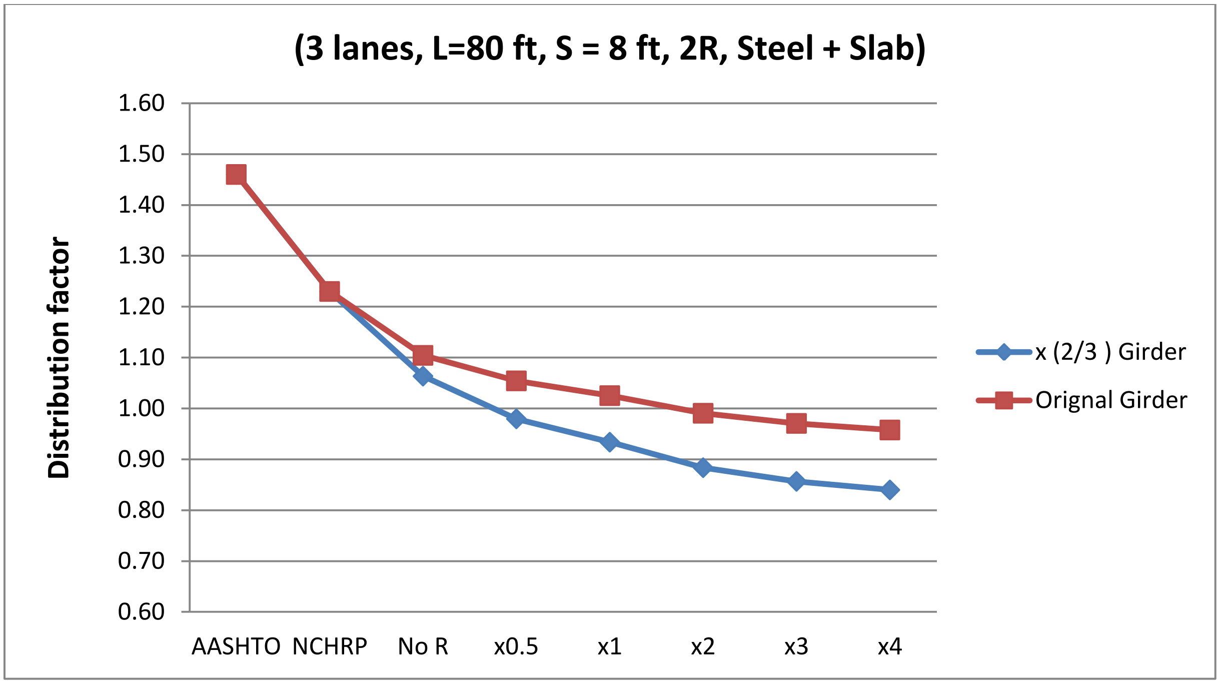


Fig. 4.27 : Sensitivity of Distribution Factor to Railing Stiffness in Reduced-Stiffness Girder Bridges (3 lanes, L= 80 ft, S = 8 ft , 2 R , Steel + Slab)

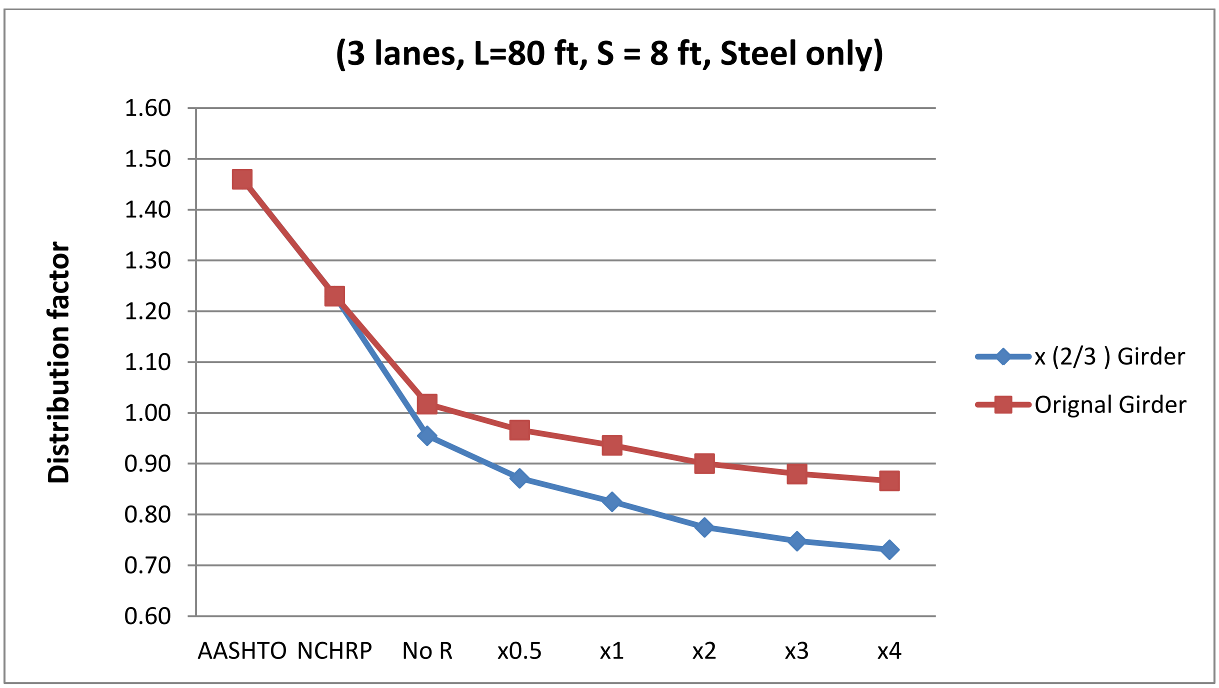


Fig. 4.28 : Sensitivity of Distribution Factor to Railing Stiffness in Reduced-Stiffness Girder Bridges (3 lanes, L= 80 ft, S = 8 ft , 2 R , Steel only)

4.3.3 *Influence on Maximum Live-Load deflections*

We have mentioned earlier how increasing the stiffness of railings alters the load distribution in one span steel girder bridges, by reducing the maximum moment carried by steel girders. In this section we'll present the influence on deflections of steel girder bridges and we'll divide this part into three sections : 2-lane , 3-lane and 4-lane bridges. The tables in each section will show the values of deflection in each case and the percentage difference of each case (span length , number of lanes , girder spacing , railing case , railing stiffness) from the reference case (No R case). The negative sign represents a decrease in deflection , and the positive sign represents an increase (this is a very rare case).

As for moments, deflections decrease (% difference from the No R case increases in absolute value) with the increase of stiffness of railings. This difference (decrease) can be seen more clearly as span length and girder spacing increase , and as the number of lanes decreases, and of course when we introduce railings on both sides of the bridge. In fact the table consists some positive values as % difference from the No R case , indicating that the maximum deflection of the bridge increases when introducing railings. This may appear as "abnormal" at the first glance , as railing shall interfere to decreasing deflections , but in fact it is normal. This is because we are extracting the "maximum" deflection of the nodes of the bridge slab , not the "average" deflection. It is mandatory that the average deflection decreases with the introduction of railings and increasing their stiffness . But it may happen that maximum deflection "slightly" increases in some "rare" cases after introducing railings. In our study, a slight increase on maximum deflection occurred after introducing one railing on the side opposite to that where the loads on the bridge are concentrated (Referred to as RR in "most" bridges cases). And this increase occurred only in some bridge cases.

a.<u>Two-lane bridges:</u>

Tables 4.43 through 4.45 show the influence of stiffness of railings on maximum deflection of two-lane bridges.

	L=40 ft , 2 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.018	-0.018	-0.018	-0.018	-1.2	-0.1	-1.3
Stiffness x1	-0.018	-0.018	-0.018	-0.018	-1.6	0.0	-1.6
Stiffness x2	-0.018	-0.018	-0.018	-0.018	-1.9	0.0	-1.9
Stiffness x3	-0.018	-0.018	-0.018	-0.018	-2.0	0.0	-2.0
Stiffness x4	-0.018	-0.018	-0.018	-0.018	-2.1	0.0	-2.1
	L=60 ft , 2 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.064	-0.059	-0.064	-0.058	-8.4	0.4	-9.3
Stiffness x1	-0.064	-0.057	-0.065	-0.056	-11.1	0.9	-13.1
Stiffness x2	-0.064	-0.055	-0.065	-0.053	-13.5	1.3	-16.6
Stiffness x3	-0.064	-0.055	-0.065	-0.052	-14.4	1.8	-18.3
Stiffness x4	-0.064	-0.054	-0.065	-0.052	-14.9	2.1	-19.4
	L=80 ft , 2 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.167	-0.135	-0.172	-0.132	-19.6	2.7	-21.0
Stiffness x1	-0.167	-0.127	-0.175	-0.120	-24.3	4.5	-28.2
Stiffness x2	-0.167	-0.121	-0.178	-0.107	-27.5	6.4	-36.0
Stiffness x3	-0.167	-0.120	-0.180	-0.101	-28.5	7.4	-40.0
Stiffness x4	-0.167	-0.119	-0.181	-0.096	-29.0	8.0	-42.5
	L=100 ft , 2 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.373	-0.289	-0.383	-0.295	-22.5	2.6	-21.1
Stiffness x1	-0.373	-0.268	-0.390	-0.250	-28.2	4.4	-33.0
Stiffness x2	-0.373	-0.264	-0.398	-0.209	-29.2	6.5	-44.0
Stiffness x3	-0.373	-0.272	-0.402	-0.186	-27.2	7.8	-50.1
Stiffness x4	-0.373	-0.276	-0.405	-0.172	-25.9	8.5	-54.0
	L=120 ft , 2 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.778	-0.615	-0.792	-0.627	-21.0	1.9	-19.3
Stiffness x1	-0.778	-0.556	-0.804	-0.532	-28.5	3.4	-31.7
Stiffness x2	-0.778	-0.575	-0.818	-0.415	-26.0	5.2	-46.7
Stiffness x3	-0.778	-0.588	-0.827	-0.355	-24.4	6.3	-54.3
Stiffness x4	-0.778	-0.596	-0.832	-0.317	-23.3	7.0	-59.3

Table 4.43. Maximum Deflection of 6 ft Girder-Spacing Bridges (2 lanes)

	L=40 ft , 2 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.023	-0.023	-0.023	-0.023	-1.9	-0.2	-2.1
Stiffness x1	-0.023	-0.023	-0.023	-0.023	-2.5	-0.2	-2.7
Stiffness x2	-0.023	-0.023	-0.023	-0.023	-3.0	-0.2	-3.2
Stiffness x3	-0.023	-0.023	-0.023	-0.022	-3.2	-0.2	-3.5
Stiffness x4	-0.023	-0.023	-0.023	-0.022	-3.4	-0.2	-3.6

	L=60 ft , 2 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.079	-0.072	-0.079	-0.070	-8.9	0.0	-10.9
Stiffness x1	-0.079	-0.069	-0.079	-0.067	-11.9	0.2	-15.1
Stiffness x2	-0.079	-0.068	-0.079	-0.063	-14.1	0.4	-19.3
Stiffness x3	-0.079	-0.067	-0.079	-0.062	-15.2	0.6	-21.4
Stiffness x4	-0.079	-0.066	-0.079	-0.061	-15.8	0.7	-22.7

	L=80 ft , 2 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.201	-0.163	-0.207	-0.158	-19.2	2.7	-21.7
Stiffness x1	-0.201	-0.154	-0.210	-0.141	-23.6	4.4	-29.9
Stiffness x2	-0.201	-0.147	-0.214	-0.124	-26.9	6.1	-38.4
Stiffness x3	-0.201	-0.145	-0.215	-0.115	-27.8	7.0	-42.8
Stiffness x4	-0.201	-0.144	-0.217	-0.109	-28.2	7.6	-45.6

	L=100 ft , 2 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.449	-0.347	-0.459	-0.345	-22.7	2.2	-23.3
Stiffness x1	-0.449	-0.324	-0.466	-0.290	-27.8	3.7	-35.5
Stiffness x2	-0.449	-0.328	-0.474	-0.237	-27.0	5.5	-47.2
Stiffness x3	-0.449	-0.335	-0.478	-0.209	-25.5	6.4	-53.5
Stiffness x4	-0.449	-0.339	-0.481	-0.191	-24.6	7.0	-57.5

	L=120 ft , 2 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.938	-0.725	-0.949	-0.734	-22.7	1.2	-21.7
Stiffness x1	-0.938	-0.690	-0.959	-0.609	-26.5	2.2	-35.1
Stiffness x2	-0.938	-0.705	-0.970	-0.466	-24.8	3.3	-50.3
Stiffness x3	-0.938	-0.714	-0.976	-0.393	-23.9	4.0	-58.1
Stiffness x4	-0.938	-0.720	-0.980	-0.348	-23.3	4.4	-63.0

Table 4.44. Maximum Deflection of 8 ft Girder-Spacing Bridges (2 lanes)

	L=40 ft , 2 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.032	-0.031	-0.032	-0.031	-3.2	-0.2	-3.8
Stiffness x1	-0.032	-0.030	-0.032	-0.030	-4.1	-0.3	-4.8
Stiffness x2	-0.032	-0.030	-0.032	-0.030	-4.8	-0.3	-5.7
Stiffness x3	-0.032	-0.030	-0.032	-0.030	-5.2	-0.3	-6.0
Stiffness x4	-0.032	-0.030	-0.032	-0.030	-5.3	-0.3	-6.2
	L=60 ft , 2 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.104	-0.093	-0.102	-0.090	-10.2	-1.3	-13.2
Stiffness x1	-0.104	-0.089	-0.102	-0.084	-13.8	-1.8	-18.8
Stiffness x2	-0.104	-0.087	-0.102	-0.079	-16.5	-1.9	-24.0
Stiffness x3	-0.104	-0.085	-0.102	-0.076	-17.6	-2.0	-26.6
Stiffness x4	-0.104	-0.085	-0.102	-0.075	-18.2	-2.0	-28.1
	L=80 ft , 2 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.254	-0.208	-0.259	-0.197	-17.9	2.1	-22.4
Stiffness x1	-0.254	-0.196	-0.262	-0.173	-22.7	3.4	-32.0
Stiffness x2	-0.254	-0.188	-0.266	-0.148	-25.7	4.7	-41.8
Stiffness x3	-0.254	-0.186	-0.267	-0.135	-26.8	5.3	-46.7
Stiffness x4	-0.254	-0.185	-0.268	-0.128	-27.2	5.7	-49.7
	L=100 ft , 2 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.569	-0.441	-0.575	-0.418	-22.4	1.1	-26.6
Stiffness x1	-0.569	-0.416	-0.579	-0.347	-26.8	1.9	-38.9
Stiffness x2	-0.569	-0.423	-0.584	-0.276	-25.6	2.7	-51.5
Stiffness x3	-0.569	-0.427	-0.587	-0.239	-25.0	3.2	-58.1
Stiffness x4	-0.569	-0.429	-0.589	-0.216	-24.6	3.5	-62.1
	L=120 ft , 2 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-1.193	-0.916	-1.189	-0.890	-23.2	-0.4	-25.4
Stiffness x1	-1.193	-0.896	-1.188	-0.717	-24.9	-0.4	-39.9
Stiffness x2	-1.193	-0.896	-1.188	-0.533	-24.9	-0.5	-55.3
Stiffness x3	-1.193	-0.895	-1.188	-0.442	-25.0	-0.5	-63.0
Stiffness x4	-1.193	-0.895	-1.187	-0.385	-25.0	-0.5	-67.7

Table 4.45. Maximum Deflection of 12 ft Girder-Spacing Bridges (2 lanes)

For 40 ft span bridges no significant decrease in the maximum deflection is observed , even in the ultimate case where the girder spacing is maximum (12 ft) ,the railing stiffness is x4 and railings are introduced on both sides , the % decrease from the No R case is 6% only.

For 60 ft span bridges , the decrease becomes more significant. When having railings on both sides , increasing the railing stiffness resulted in increasing the % difference from 9% to 19% for 6 ft girder spacing , from 11% to 23 % for 8 ft girder spacing , and from 13 % to 28 % for 12 ft girder spacing. These ranges are obtained when varying the stiffness from x0.5 to x4.

For 80 ft span bridges , and for the 2 railing case, varying the stiffness of railing resulted in increasing the % difference of maximum deflection from 21% for x0.5 stiffness to about 45% for x4 stiffness regardless of the girder spacing which didn't have much influence in this case.

For 100 ft span bridges , the case is similar to 80 ft span bridges. Where increasing the stiffness of the 2 railing combined resulted in increasing the % difference of maximum deflection from 23% for x0.5 stiffness to about 58% for x4 stiffness with minor influence from girder spacing.

For 120 ft span bridges , the effect of stiffness of railings has shown to be large in the 2 railing case. While the x0.5 stiffness railing resulted in about 22 % decrease from the case of no railings , this % became about 60 % in the case of x 4 railing , also with minimal effect of girder spacing. When having a railing on one side of a bridge ,if the bridge loads are close to the railing the effect would be significant, but still less than that for case of having 2 railings ; whereas; if the concentration of loads was on the side opposite to that of the railing , the maximum deflection would be slightly greater than the maximum deflection of the No R case.

b.Three-lane bridges:

Tables 4.46 through 4.48 show the influence of stiffness of railings on maximum deflection of three-lane bridges.

	L=40 ft , 3 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.020	-0.021	-0.020	-0.021	0.2	0.0	0.2
Stiffness x1	-0.020	-0.021	-0.020	-0.021	0.4	0.0	0.4
Stiffness x2	-0.020	-0.021	-0.020	-0.021	0.5	0.0	0.5
Stiffness x3	-0.020	-0.021	-0.020	-0.021	0.6	0.0	0.6
Stiffness x4	-0.020	-0.021	-0.020	-0.021	0.6	0.0	0.6
	L=60 ft , 3 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.073	-0.072	-0.073	-0.072	-2.5	0.0	-2.5
Stiffness x1	-0.073	-0.071	-0.073	-0.071	-3.4	0.1	-3.5
Stiffness x2	-0.073	-0.071	-0.074	-0.070	-3.9	0.2	-4.1
Stiffness x3	-0.073	-0.070	-0.074	-0.070	-4.2	0.3	-4.4
Stiffness x4	-0.073	-0.070	-0.074	-0.070	-4.4	0.3	-4.5
	L=80 ft , 3 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.174	-0.161	-0.175	-0.160	-7.8	0.5	-8.2
Stiffness x1	-0.174	-0.155	-0.176	-0.154	-10.7	1.0	-11.7
Stiffness x2	-0.174	-0.151	-0.177	-0.147	-13.1	1.8	-15.3
Stiffness x3	-0.174	-0.149	-0.178	-0.144	-14.2	2.3	-17.1
Stiffness x4	-0.174	-0.148	-0.179	-0.143	-14.8	2.6	-18.1
	L=100 ft , 3 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.406	-0.336	-0.413	-0.335	-17.2	1.9	-17.5
Stiffness x1	-0.406	-0.317	-0.419	-0.309	-21.8	3.2	-23.8
Stiffness x2	-0.406	-0.302	-0.425	-0.283	-25.5	4.7	-30.4
Stiffness x3	-0.406	-0.296	-0.429	-0.268	-27.0	5.6	-34.1
Stiffness x4	-0.406	-0.293	-0.431	-0.258	-27.9	6.2	-36.4
	L=120 ft , 3 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.871	-0.702	-0.888	-0.717	-19.4	2.0	-17.6
Stiffness x1	-0.871	-0.640	-0.901	-0.625	-26.5	3.5	-28.2
Stiffness x2	-0.871	-0.603	-0.918	-0.542	-30.8	5.5	-37.8
Stiffness x3	-0.871	-0.592	-0.928	-0.495	-32.0	6.6	-43.2
Stiffness x4	-0.871	-0.589	-0.935	-0.464	-32.3	7.4	-46.7

Table 4.46. Maximum Deflection of 6 ft Girder-Spacing Bridges (3 lanes)

	L=40 ft , 3 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.027	-0.027	-0.027	-0.027	0.1	0.0	0.0
Stiffness x1	-0.027	-0.027	-0.027	-0.027	0.1	0.0	0.1
Stiffness x2	-0.027	-0.027	-0.027	-0.027	0.2	0.0	0.2
Stiffness x3	-0.027	-0.027	-0.027	-0.027	0.3	0.1	0.3
Stiffness x4	-0.027	-0.027	-0.027	-0.027	0.3	0.1	0.3
	L=60 ft , 3 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.091	-0.090	-0.091	-0.090	-0.9	-0.4	-1.5
Stiffness x1	-0.091	-0.090	-0.091	-0.089	-1.2	-0.5	-2.0
Stiffness x2	-0.091	-0.090	-0.090	-0.089	-1.5	-0.6	-2.6
Stiffness x3	-0.091	-0.089	-0.090	-0.088	-1.7	-0.6	-2.9
Stiffness x4	-0.091	-0.089	-0.090	-0.088	-1.8	-0.7	-3.1
	L=80 ft , 3 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.200	-0.194	-0.197	-0.190	-2.9	-1.5	-5.0
Stiffness x1	-0.200	-0.191	-0.196	-0.184	-4.4	-2.1	-7.9
Stiffness x2	-0.200	-0.189	-0.195	-0.178	-5.8	-2.8	-11.3
Stiffness x3	-0.200	-0.187	-0.194	-0.174	-6.5	-3.1	-13.2
Stiffness x4	-0.200	-0.186	-0.194	-0.171	-7.0	-3.3	-14.5
	L=100 ft , 3 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.412	-0.392	-0.404	-0.377	-4.9	-2.0	-8.4
Stiffness x1	-0.412	-0.382	-0.401	-0.354	-7.3	-2.7	-14.0
Stiffness x2	-0.412	-0.373	-0.398	-0.325	-9.5	-3.3	-21.1
Stiffness x3	-0.412	-0.369	-0.398	-0.308	-10.4	-3.5	-25.3
Stiffness x4	-0.412	-0.367	-0.398	-0.296	-11.0	-3.5	-28.2
	L=120 ft , 3 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.841	-0.786	-0.834	-0.748	-6.5	-0.8	-11.0
Stiffness x1	-0.841	-0.765	-0.848	-0.684	-9.0	0.9	-18.6
Stiffness x2	-0.841	-0.753	-0.866	-0.601	-10.4	3.0	-28.5
Stiffness x3	-0.841	-0.764	-0.877	-0.548	-9.2	4.3	-34.8
Stiffness x4	-0.841	-0.772	-0.885	-0.512	-8.2	5.2	-39.1

Table 4.47. Maximum Deflection of 8 ft Girder-Spacing Bridges (3 lanes)

	L=40 ft , 3 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.039	-0.039	-0.039	-0.038	-0.1	0.0	-0.1
Stiffness x1	-0.039	-0.039	-0.039	-0.039	-0.1	0.0	-0.1
Stiffness x2	-0.039	-0.039	-0.039	-0.039	0.0	0.0	0.0
Stiffness x3	-0.039	-0.039	-0.039	-0.039	0.0	0.0	0.0
Stiffness x4	-0.039	-0.039	-0.039	-0.039	0.0	0.0	0.0
	L=60 ft , 3 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.126	-0.124	-0.123	-0.121	-1.1	-2.0	-3.5
Stiffness x1	-0.126	-0.124	-0.122	-0.119	-1.4	-2.8	-5.0
Stiffness x2	-0.126	-0.123	-0.121	-0.118	-1.8	-3.5	-6.4
Stiffness x3	-0.126	-0.123	-0.121	-0.117	-2.0	-3.9	-7.0
Stiffness x4	-0.126	-0.123	-0.120	-0.116	-2.1	-4.1	-7.4
	L=80 ft , 3 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.280	-0.272	-0.265	-0.254	-2.6	-5.1	-9.1
Stiffness x1	-0.280	-0.270	-0.260	-0.241	-3.5	-7.1	-13.9
Stiffness x2	-0.280	-0.268	-0.254	-0.226	-4.2	-9.1	-19.0
Stiffness x3	-0.280	-0.267	-0.252	-0.219	-4.5	-9.8	-21.7
Stiffness x4	-0.280	-0.266	-0.251	-0.214	-4.7	-10.3	-23.3
	L=100 ft , 3 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.587	-0.574	-0.544	-0.476	-2.1	-7.3	-18.9
Stiffness x1	-0.587	-0.580	-0.528	-0.461	-1.2	-10.0	-21.5
Stiffness x2	-0.587	-0.590	-0.519	-0.408	0.6	-11.6	-30.6
Stiffness x3	-0.587	-0.596	-0.517	-0.378	1.6	-11.9	-35.6
Stiffness x4	-0.587	-0.600	-0.519	-0.360	2.2	-11.5	-38.7
	L=120 ft , 3 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-1.220	-1.247	-1.109	-1.015	2.2	-9.1	-16.8
Stiffness x1	-1.220	-1.264	-1.108	-0.892	3.6	-9.2	-26.9
Stiffness x2	-1.220	-1.284	-1.131	-0.748	5.3	-7.3	-38.7
Stiffness x3	-1.220	-1.295	-1.144	-0.665	6.2	-6.2	-45.5
Stiffness x4	-1.220	-1.302	-1.152	-0.611	6.8	-5.6	-49.9

Table 4.48. Maximum Deflection of 12 ft Girder-Spacing Bridges (3 lanes)

For 40ft span bridges no decrease in maximum deflection is observed when increasing the stiffness of railings.

For 60 ft span bridges , the decrease is still not significant especially for 6 ft and 8 ft girder spacings . For 12 ft girder spacing ,and when having railings on both sides , increasing the railing stiffness from x0.5 to x4 results in increasing the % difference from 3.5% to 7.5% , with respect to the No railing case.

For 80 ft span bridges , the decrease becomes more significant. When having railings on both sides , increasing the railing stiffness resulted in increasing the % difference with respect to the No railing case from 8% to 18% for 6 ft girder spacing , from 5% to 15 % for 8 ft girder spacing , and from 9 % to 23 % for 12 ft girder spacing. These ranges are obtained when varying the stiffness from x0.5 to x4.

For 100 ft span bridges , the case is similar to 80 ft span bridges. Where increasing the stiffness of the 2 railings combined resulted in increasing the % difference of maximum deflection from 18% for x0.5 stiffness to about 36% for x4 stiffness when the girder spacing is 6ft or 12ft. But when the girder spacing is 8 ft , the % difference increases from 8% to 28 %.

For 120 ft span bridges , the effect of stiffness of railing has shown to be large in the 2 railing case. While the x0.5 stiffness railing resulted in about 17 % decrease from the case of no railings , this % became about 50 % in the case of x 4 railing for 6ft and 12 ft girder spacing , while the % difference increased from 11 % to 39 % for 8 ft girder spacing.

When having a railing on one side of a bridge ,if the bridge loads are close to the railing the effect would be significant, but still less than that for case of having 2 railings ; whereas; if the concentration of loads was on the side opposite to that of the railing , the maximum deflection would be slightly greater than the maximum deflection of the No R case.

c.<u>Four-lane bridges:</u>

Tables 4.49 through 4.51 show the influence of stiffness of railings on maximum deflection of four-lane bridges.

	L=40 ft , 4 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.021	-0.021	-0.021	-0.021	0.5	0.0	0.5
Stiffness x1	-0.021	-0.021	-0.021	-0.021	0.7	0.0	0.7
Stiffness x2	-0.021	-0.022	-0.021	-0.021	0.8	0.0	0.8
Stiffness x3	-0.021	-0.022	-0.021	-0.022	0.9	0.0	0.9
Stiffness x4	-0.021	-0.022	-0.021	-0.022	1.0	0.0	0.9
	L=60 ft , 4 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.080	-0.080	-0.080	-0.080	-0.1	0.0	-0.1
Stiffness x1	-0.080	-0.080	-0.080	-0.080	0.0	0.1	0.0
Stiffness x2	-0.080	-0.080	-0.080	-0.080	0.0	0.1	0.1
Stiffness x3	-0.080	-0.080	-0.080	-0.080	0.0	0.1	0.1
Stiffness x4	-0.080	-0.080	-0.080	-0.080	0.0	0.1	0.1
	L=80 ft , 4 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.187	-0.182	-0.187	-0.182	-2.9	0.1	-2.9
Stiffness x1	-0.187	-0.180	-0.187	-0.179	-3.9	0.2	-4.0
Stiffness x2	-0.187	-0.178	-0.188	-0.177	-5.0	0.3	-5.1
Stiffness x3	-0.187	-0.177	-0.188	-0.176	-5.5	0.4	-5.7
Stiffness x4	-0.187	-0.176	-0.188	-0.176	-5.7	0.4	-6.1
	L=100 ft , 4 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.407	-0.375	-0.411	-0.374	-8.0	0.9	-8.2
Stiffness x1	-0.407	-0.363	-0.413	-0.361	-10.8	1.6	-11.4
Stiffness x2	-0.407	-0.353	-0.417	-0.347	-13.4	2.3	-14.9
Stiffness x3	-0.407	-0.348	-0.418	-0.339	-14.6	2.8	-16.7
Stiffness x4	-0.407	-0.344	-0.420	-0.334	-15.4	3.1	-17.9
	L=120 ft , 4 lanes ,6 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.903	-0.761	-0.915	-0.762	-15.7	1.4	-15.6
Stiffness x1	-0.903	-0.720	-0.924	-0.711	-20.2	2.4	-21.2
Stiffness x2	-0.903	-0.685	-0.936	-0.658	-24.1	3.7	-27.1
Stiffness x3	-0.903	-0.670	-0.943	-0.628	-25.8	4.5	-30.5
Stiffness x4	-0.903	-0.661	-0.948	-0.608	-26.8	5.0	-32.7

Table 4.49. Maximum Deflection of 6 ft Girder-Spacing Bridges (4 lanes)

	L=40 ft , 4 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.028	-0.028	-0.028	-0.028	0.4	0.0	0.4
Stiffness x1	-0.028	-0.028	-0.028	-0.028	0.6	0.0	0.5
Stiffness x2	-0.028	-0.028	-0.028	-0.028	0.7	0.0	0.7
Stiffness x3	-0.028	-0.028	-0.028	-0.028	0.7	0.0	0.7
Stiffness x4	-0.028	-0.028	-0.028	-0.028	0.8	0.0	0.8
	L=60 ft , 4 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.102	-0.102	-0.102	-0.102	-0.5	0.0	-0.5
Stiffness x1	-0.102	-0.102	-0.102	-0.102	-0.7	0.1	-0.7
Stiffness x2	-0.102	-0.101	-0.103	-0.102	-0.9	0.1	-0.8
Stiffness x3	-0.102	-0.101	-0.103	-0.102	-1.0	0.1	-0.9
Stiffness x4	-0.102	-0.101	-0.103	-0.101	-1.1	0.1	-0.9
	L=80 ft , 4 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.237	-0.228	-0.237	-0.228	-3.6	0.0	-3.9
Stiffness x1	-0.237	-0.225	-0.237	-0.224	-5.1	0.1	-5.4
Stiffness x2	-0.237	-0.221	-0.237	-0.220	-6.5	0.2	-7.1
Stiffness x3	-0.237	-0.220	-0.237	-0.218	-7.1	0.3	-8.0
Stiffness x4	-0.237	-0.219	-0.238	-0.217	-7.4	0.3	-8.4
	L=100 ft , 4 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.509	-0.466	-0.515	-0.463	-8.5	1.1	-9.0
Stiffness x1	-0.509	-0.450	-0.519	-0.443	-11.6	1.9	-13.0
Stiffness x2	-0.509	-0.435	-0.524	-0.421	-14.6	2.9	-17.3
Stiffness x3	-0.509	-0.428	-0.526	-0.409	-16.0	3.4	-19.7
Stiffness x4	-0.509	-0.424	-0.528	-0.401	-16.7	3.7	-21.2
	L=120 ft , 4 lanes ,8 ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-1.125	-0.940	-1.143	-0.935	-16.4	1.6	-16.8
Stiffness x1	-1.125	-0.887	-1.155	-0.864	-21.2	2.7	-23.2
Stiffness x2	-1.125	-0.843	-1.171	-0.786	-25.1	4.1	-30.1
Stiffness x3	-1.125	-0.823	-1.180	-0.742	-26.8	4.9	-34.0
Stiffness x4	-1.125	-0.813	-1.187	-0.713	-27.7	5.5	-36.6

Table 4.50. Maximum Deflection of 8 ft Girder-Spacing Bridges (4 lanes)

	L=40 ft , 4 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.041	-0.041	-0.041	-0.041	0.0	0.0	0.0
Stiffness x1	-0.041	-0.041	-0.041	-0.041	0.0	0.0	0.0
Stiffness x2	-0.041	-0.041	-0.041	-0.041	0.1	0.0	0.1
Stiffness x3	-0.041	-0.041	-0.041	-0.041	0.1	0.0	0.1
Stiffness x4	-0.041	-0.041	-0.041	-0.041	0.1	0.0	0.1
	L=60 ft , 4 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.144	-0.142	-0.144	-0.141	-1.5	0.0	-1.6
Stiffness x1	-0.144	-0.141	-0.144	-0.141	-2.1	0.0	-2.1
Stiffness x2	-0.144	-0.140	-0.144	-0.140	-2.6	0.1	-2.6
Stiffness x3	-0.144	-0.140	-0.144	-0.140	-2.8	0.1	-2.9
Stiffness x4	-0.144	-0.139	-0.144	-0.139	-3.0	0.1	-3.0
	L=80 ft , 4 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.325	-0.309	-0.324	-0.307	-4.9	-0.3	-5.6
Stiffness x1	-0.325	-0.303	-0.324	-0.299	-7.0	-0.4	-8.1
Stiffness x2	-0.325	-0.297	-0.324	-0.291	-8.8	-0.5	-10.7
Stiffness x3	-0.325	-0.294	-0.324	-0.286	-9.6	-0.5	-12.0
Stiffness x4	-0.325	-0.292	-0.323	-0.284	-10.2	-0.6	-12.7
	L=100 ft , 4 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-0.688	-0.623	-0.689	-0.613	-9.5	0.2	-10.9
Stiffness x1	-0.688	-0.597	-0.694	-0.577	-13.2	0.8	-16.2
Stiffness x2	-0.688	-0.574	-0.701	-0.538	-16.5	1.8	-21.8
Stiffness x3	-0.688	-0.564	-0.704	-0.517	-18.0	2.4	-24.9
Stiffness x4	-0.688	-0.559	-0.707	-0.504	-18.8	2.7	-26.8
	L=120 ft , 4 lanes ,12ft						
	Deflection (ft)				% difference (from No R)		
	No R	RL	RR	2R	RL	RR	2R
Stiffness x0.5	-1.495	-1.242	-1.519	-1.220	-16.9	1.6	-18.4
Stiffness x1	-1.495	-1.169	-1.536	-1.106	-21.8	2.7	-26.0
Stiffness x2	-1.495	-1.109	-1.555	-0.980	-25.8	4.0	-34.4
Stiffness x3	-1.495	-1.084	-1.566	-0.911	-27.5	4.8	-39.1
Stiffness x4	-1.495	-1.071	-1.573	-0.866	-28.3	5.3	-42.1

Table 4.51. Maximum Deflection of 12 ft Girder-Spacing Bridges (4 lanes)

For 40 ft and 60 ft span bridges no decrease in deflection is observed , even in the ultimate case where the girder spacing is maximum (12 ft) ,the railing stiffness is x4 and railings are introduced on both sides.

For 80 ft span bridges and when having railings on both sides , increasing the railing stiffness resulted in increasing the % difference from 3% to 6% for 6 ft girder spacing , from 4% to 8 % for 8 ft girder spacing , and from 6 % to 12 % for 12 ft girder spacing. These ranges are obtained when varying the stiffness from x0.5 to x4.

For 100 ft span bridges , the effect becomes more significant. Increasing the stiffness of the 2 railings combined resulted in increasing the % difference from 8% to 18% for 6 ft girder spacing , from 9% to 21 % for 8 ft girder spacing , and from 11 % to 27 % for 12 ft girder spacing. These ranges are obtained when varying the stiffness from x0.5 to x4.

For 120 ft span bridges the x0.5 stiffness railing resulted a decrease of about 18 % from the case of no railings for all girder spacings , this % became about 32 % , 36 % and 42 % for 6 ft , 8 ft and 12 ft girder spacings respectively in the case of x4 stiffness railing.

When having a railing on one side of a bridge ,if the bridge loads are close to the railing the effect would be significant, but still less than that for case of having 2 railings ; whereas; if the concentration of loads was on the side opposite to that of the railing , the maximum deflection would be slightly greater than the maximum deflection of the No R case.

CHAPTER 5

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1. Summary

In this thesis, the influence of stiffness of railings on the load carrying capacity and maximum deflection of one-span, multi-lane, steel girder bridges was investigated. Railings with several sizes (stiffnesses) 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. A total of 720 bridge cases were analyzed using Finite Element Software (SAP 2013). 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 stiffness of railings cast integrally with the slab deck on the lateral load distribution in the bridge, which is mainly assessed by comparing bridge cases with railings of several sizes with the reference case which is the bridge case without railings.

First, the influence of railings on one-span bridges has been studied by introducing railings of standard size (8"x30") {referred to as x1 railing}on either or both sides of a bridge , without varying the stiffness of railings. Since at this stage we aimed at showing that railings do have a significant influence on steel girder bridges. This issue has been subject to research by Nuwayhid (2015) , who studied ; in addition to railings ; the influence of sidewalks on one-span steel girder bridges. FEA results were compared to AASHTO Standard Specifications and LRFD procedures and to the reference case (No R) and results have shown an increase in the load carrying capacity of bridges.

Then we moved to varying the stiffness of railings where we considered 5 stiffness factors and presented the results of each. Comparison between bridges with railings of several sizes were made , and results were shown in table and chart form. Increasing the stiffness of railings has shown significant increase in the load carrying capacity of one-span bridges , and a significant decrease in the maximum deflection . This influence increases with the increase of span length , with the decrease in number of lanes , with the increase of railing stiffness , and with the introduction of railings on both sides of a bridge.

A small section of this research was assigned to figure out whether the influence of stiffness of railings is altered by the stiffness of girders. 2 bridge cases were studied with and without reducing the stiffness of steel girders and their results were tabulated.

5.2. Conclusion

5.2.1. Conclusion on Distribution Factors

For 2 lane-bridges , and for spans up to 40 ft, the addition of railings has a negligible effect on the distribution factor for all stiffness factors. Otherwise, the average reductions in FEA distribution factors when introducing railing on one side for spans between 60 and 80 ft were between 5% and 11% for stiffness factors x0.5 through x4, and between 8 % to 20 % for stiffness factors x0.5 through x4 for spans between 80 and 120 ft compared to the base case (no railings).

The average reductions in FEA distribution factors when introducing railings on both sides for spans between 60 and 80 ft were between 12% and 28% for stiffness factors x0.5 through x4, and between 16 % to 50 % for stiffness factors x0.5 through x4 for spans between 80 and 120 ft compared to the base case (no railings).

For 3-lane bridges, and for spans up to 40 ft, the addition or railings has a negligible effect on the distribution factor. Otherwise, the average reductions in FEA distribution factors when compared to the base case were : 2 % for spans between 40 and 80 ft (12 and 18 m) (almost same for all railing stiffnesses), 4 % to 6% (depending on stiffness of railing) for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on one side, and 3 % to 6 % (depending on railing stiffness) for spans between 40 and 80 ft (12 and 24 m) and 11% to 30% (depending on railing stiffness) for spans between 80 and 120 ft (24 and 36 m) when introducing railings on both sides.

For 4-lane bridges, and for spans up to 60 ft, the addition of railings has a negligible effect on the distribution factor .Otherwise, the average reductions in FEA distribution factors when compared to the base case were; 0 % when introducing railings on one side; 4% to 10% (depending on railing stiffness) for spans between 60 and 80 ft (18 and 24 m) and 8 % to 20 %

(depending on railing stiffness) for spans between 80 and 120 ft (24 m and 36 m) when introducing railings on both sides.

The finite-element results show the effects of stiffness of 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 , though ; bridges with higher girder spacing are "slightly" more influenced by the increase of stiffness of railings.In reality, all the reduction discussed in this section implies an increase in the load-carrying capacity due to the increasing the stiffness of railings in a bridge superstructure.

5.2.2. Conclusion on Maximum Live-Load Deflections

For 2-lane bridges, and for spans up to 40 span bridges no significant decrease in maximum deflection is observed when increasing the stiffness of railings. For 60 ft span bridges, and when having railings on both sides, increasing the railing stiffness resulted in increasing the % difference ;on average; from 11% to 23 % when varying the stiffness from x0.5 to x4. For 80 ft span bridges , and for the 2 railing case, varying the stiffness of railing resulted in increasing the % difference of maximum deflection from 21% for x0.5 stiffness to about 45% for x4 stiffness. For 100 ft span bridges, increasing the stiffness of the 2 railings combined resulted in increasing the % difference of maximum deflection from 23% for x0.5 stiffness to about 58% for x4. For 120 ft span bridges , and for the 2 railing case, while the x0.5 stiffness railing resulted in about 22 % decrease from the case of no railings , this % became about 60 % in the case of x 4 railing , also with minimal effect of girder spacing.

For 3-lane bridges, and for 40 span bridges no decrease in maximum deflection is observed when increasing the stiffness of railings. For 60 ft span bridges with 12 ft girder spacing ,and when having railings on both sides , increasing the railing stiffness from x0.5 to x4 results in increasing the % difference from 3.5% to 7.5%.For 80 ft span bridges, and when having railings on both sides , increasing the railing stiffness from x0.5 to x4 resulted in increasing the % difference ; on average; from 8% to 18%. For 100 ft span bridges, increasing the stiffness of the 2 railings combined resulted in increasing the % difference of maximum deflection from 15% for x0.5 stiffness to about 35% for x4 stiffness. For 120 ft span bridges , and for the 2-railing case, while the x0.5 stiffness railing resulted in about 15 % decrease in maximum deflection from the case of no railings , this % became about 50 % in the case of x 4 railing.

For 4-lane bridges , and for bridges with spans up to 60ft, no decrease in maximum deflection is observed when increasing the stiffness of railings. For 80 ft span bridges and when having railings on both sides , increasing the railing stiffness from x0.5 to x4 resulted in increasing the % difference from 4% to 9 % (with respect to the No Railing case).For 100 ft span bridges , increasing the stiffness of the 2 railings combined from x1 to x4 resulted in increasing the % difference from 10% to 22%. For 120 ft span bridges the x0.5 stiffness railing resulted a decrease of about 18 % from the case of no railings, this % became about 36 % in the case of x4 stiffness railing.

For all span bridges , girder spacing has shown little influence on decreasing the maximum deflection , thought ; maximum deflections in bridges with a larger girder spacing are slightly more influenced by increasing the stiffness of railings. Moreover; when having a railing on one side of a bridge ,if the bridge loads are close to the railing (railing and loads are on the same side of bridge) the effect would be significant (depending on the bridge case) , but still less than that for case of having 2 railings ; whereas; if the concentration of loads was on the side opposite to that of the railing , the maximum deflection would be slightly greater than the maximum deflection of the No R case . Consequently , we can't rely on a decrease in maximum deflection when having bridge cases with one railing only , since we don't know the real positions of live loads on bridges. So, we can only talk about a decrease in maximum deflection of bridges when we have 2 railings on a bridge deck.

5.2.3. Conclusion on Bridges with Reduced-Stiffness Girders

For the 2 bridge-cases studied (3 lanes , 40 ft span length ,8 ft girder spacing , 2 R) and (3 lanes, 40 ft span length ,8 ft girder spacing , 2 R) we conclude the following:

When reducing the stiffness of the girder of the 40 ft bridge by a factor of 1/3 , the %decrease in distribution factor with respect to the no railing bridge case ranged from 2% for x0.5 railing stiffness to 3% for x4 stiffness. Whereas; for the same bridge with non-reduced stiffness girders, there is no decrease in the distribution factor for any railing stiffness.

When reducing the stiffness of the girder of the 80 ft bridge by a factor of 2/3 , the %decrease in distribution factor with respect to the no-railing bridge-case ranged from 8% for x0.5 railing stiffness to 21% for x4 stiffness. Whereas; for the same bridge with non-reduced stiffness girders the % decrease in distribution factor with respect to the no railing bridge case ranged from 4.5% for x0.5 railing stiffness to 13% for x4 railing stiffness.

This implies that reducing the stiffness of girders makes the bridge more ifluenced by the variation of stiffness of railings.

5.3. Recommendations

It is recommended not to rely on an increase in load carrying capacity or a decrease in the maximum deflection of bridges of spans up to 60 ft when increasing the stiffness of railings. Experimental work is recommended to assess the accuracy of the finite element analysis obtained in this investigation. Moreover, since most of the bridges are cast with railings, it would be recommended that they are casted integrally with the bridge deck in order to ensure an increase in the moment capacity of the bridge.

APPENDIX

Table A.1. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 2 lanes, Span L=80 ft, Spacing S = 6 ft)

(Stiffness x1, 2 Lanes, L=80 ft, S = 6 ft)								
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Total	Max
no R	Girder	514.8	503.3	472.9	405.4	311.5	2208.0	514.8
	Slab	28.1	30.4	28.2	20.9	14.0	121.6	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	542.9	533.7	501.1	426.4	325.5	2329.6	533.7
RL	Girder	348.3	407.9	429.0	398.8	336.2	1920.1	429.0
	Slab	22.4	29.4	28.7	21.9	15.5	118.0	
	Railling	291.5	0.0	0.0	0.0	0.0	291.5	
	Total	662.1	437.4	457.7	420.7	351.7	2329.6	457.7
RR	Girder	526.6	498.2	447.6	353.9	227.9	2054.1	526.6
	Slab	28.9	30.8	28.3	20.2	10.6	118.7	
	Railling	0.0	0.0	0.0	0.0	156.7	156.7	
	Total	555.4	529.0	475.9	374.1	395.2	2329.6	529.0
2R	Girder	353.6	398.1	398.6	339.0	238.0	1727.3	398.6
	Slab	23.1	30.0	29.0	21.2	11.8	115.2	
	Railling	305.5	0.0	0.0	0.0	181.6	487.1	
	Total	682.2	428.2	427.6	360.3	431.4	2329.6	428.2

Table A.2. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 2 lanes, Span L=80 ft, Spacing S = 8 ft)

(Stiffness x1, 2 Lanes, L=80 ft, S = 8 ft)							
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Total	Max
no R	Girder	622.9	614.6	532.7	404.2	2174.3	622.9
	Slab	39.9	47.8	45.1	22.5	155.3	
	Railling	0.0	0.0	0.0	0.0	0.0	
	Total	662.8	662.4	577.8	426.7	2329.6	662.4
RL	Girder	408.7	509.3	500.2	426.4	1844.7	509.3
	Slab	31.6	46.1	45.8	24.2	147.8	
	Railling	337.1	0.0	0.0	0.0	0.0	
	Total	777.5	555.4	546.0	450.7	1992.5	555.4
RR	Girder	634.0	593.4	469.5	285.3	1982.2	634.0
	Slab	40.8	48.0	43.8	17.3	149.8	
	Railling	0.0	0.0	0.0	197.6	197.6	
	Total	674.8	641.4	513.2	500.2	2329.6	641.4
2R	Girder	411.2	482.1	428.7	290.6	1612.6	482.1
	Slab	32.4	46.5	44.6	18.6	142.2	
	Railling	352.2	0.0	0.0	222.6	574.8	
	Total	795.8	528.6	473.2	531.9	2329.6	528.6

Table A.3. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 2 lanes, Span L=80 ft, Spacing S = 12 ft)

(Stiffness x1, 2 Lanes, L=80 ft, S = 12 ft)						
Case	Zone	Girder 1	Girder 2	Girder 3	Total	Max
no R	Girder	795.0	768.0	560.1	2123.1	795.0
	Slab	65.9	93.9	46.7	206.5	
	Railling	0.0	0.0	0.0	0.0	
	Total	861.0	861.9	606.7	2329.6	861.9
RL	Girder	503.9	661.1	568.3	1733.4	661.1
	Slab	52.5	90.6	47.8	190.9	
	Railling	405.3	0.0	0.0	405.3	
	Total	961.7	751.7	616.2	2329.6	751.7
RR	Girder	797.8	695.3	378.0	1871.1	797.8
	Slab	66.5	91.3	37.3	195.1	
	Railling	0.0	0.0	263.4	263.4	
	Total	864.3	786.6	678.7	2329.6	786.6
2R	Girder	497.3	581.9	369.9	1449.0	581.9
	Slab	53.0	88.2	38.2	179.4	
	Railling	417.9	0.0	283.3	701.2	
	Total	968.2	670.1	691.4	2329.6	670.1

Table A.4. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 3 lanes, Span L=80 ft, Spacing S = 6 ft)

(Stiffness x1 , 3 Lanes, L=80 ft, S = 6 ft)										
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Total	Max
no R	Girder	548.2	564.3	564.6	533.0	464.6	369.1	260.4	3304.2	564.6
	Slab	29.3	32.6	32.9	32.6	31.0	20.9	11.0	190.3	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	577.5	597.0	597.5	565.5	495.6	390.1	271.4	3494.4	597.5
RL	Girder	381.7	466.2	513.3	511.2	462.2	379.9	281.0	2995.6	513.3
	Slab	23.6	32.0	33.8	33.9	32.2	22.0	12.1	189.6	
	Railling	309.2	0.0	0.0	0.0	0.0	0.0	0.0	309.2	
	Total	714.4	498.2	547.2	545.1	494.5	401.8	293.1	3494.4	547.2
RR	Girder	556.1	568.0	562.7	523.0	442.2	329.3	199.2	3180.5	568.0
	Slab	29.7	33.0	33.3	33.0	31.2	20.4	8.3	188.9	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	125.0	125.0	
	Total	585.7	601.0	596.1	556.0	473.4	349.6	332.6	3494.4	601.0
2R	Girder	386.8	468.4	510.4	499.8	437.4	335.3	211.5	2849.7	510.4
	Slab	23.9	32.5	34.4	34.5	32.5	21.4	9.2	188.5	
	Railling	315.8	0.0	0.0	0.0	0.0	0.0	140.5	456.3	
	Total	726.5	500.8	544.8	534.3	470.0	356.8	361.1	3494.4	544.8

Table A.5. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 3 lanes, Span L=80 ft, Spacing S = 8 ft)

(Stiffness x1 , 3 Lanes, L=80 ft, S =8 ft)									
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Total	Max
no R	Girder	464.2	591.4	658.5	636.8	532.7	370.5	3254.0	658.5
	Slab	21.0	50.4	56.2	58.9	38.4	15.6	240.4	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	485.3	641.8	714.7	695.6	571.1	386.0	3494.4	714.7
RL	Girder	318.2	506.0	618.0	624.0	536.4	385.7	2988.4	624.0
	Slab	16.1	48.5	56.6	59.9	39.3	16.2	236.7	
	Railling	269.4	0.0	0.0	0.0	0.0	0.0	269.4	
	Total	603.8	554.5	674.6	683.9	575.8	401.9	3494.4	683.9
RR	Girder	475.8	593.9	648.0	604.4	465.8	258.3	3046.2	648.0
	Slab	21.5	51.1	56.9	59.1	36.7	11.6	237.1	
	Railling	0.0	0.0	0.0	0.0	0.0	211.2	211.2	
	Total	497.3	645.0	704.9	663.5	502.5	481.1	3494.4	704.9
2R	Girder	325.3	506.1	605.9	589.8	466.3	267.4	2761.0	605.9
	Slab	16.5	49.2	57.5	60.3	37.7	12.2	233.4	
	Railling	277.9	0.0	0.0	0.0	0.0	222.1	500.0	
	Total	619.8	555.4	663.4	650.1	504.0	501.7	3494.4	663.4

Table A.6. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 3 lanes, Span L=80 ft, Spacing S = 12 ft)

(Stiffness x1 , 3 Lanes, L=80 ft, S = 12 ft)							
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Total	Max
no R	Girder	633.6	865.6	907.2	756.1	3162.5	907.2
	Slab	51.6	106.0	110.0	64.3	331.9	
	Railling	0.0	0.0	0.0	0.0	0.0	
	Total	685.2	971.6	1017.2	820.4	3494.4	1017.2
RL	Girder	430.9	778.0	885.7	773.7	2868.3	885.7
	Slab	41.5	103.7	111.0	65.7	322.1	
	Railling	304.1	0.0	0.0	0.0	304.1	
	Total	776.5	881.8	996.7	839.5	3494.4	996.7
RR	Girder	655.9	840.0	800.7	502.1	2798.7	840.0
	Slab	53.4	107.4	107.4	52.3	320.5	
	Railling	0.0	0.0	0.0	375.2	375.2	
	Total	709.3	947.5	908.2	929.5	3494.4	947.5
2R	Girder	440.1	747.1	774.5	509.4	2471.1	774.5
	Slab	43.0	105.3	108.6	53.4	310.3	
	Railling	322.7	0.0	0.0	390.4	713.1	
	Total	805.8	852.3	883.1	953.2	3494.4	883.1

Table A.7. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 4 lanes, Span L=80 ft, Spacing S = 6 ft)

(Stiffness x1, 4 Lanes, L=80 ft, S = 6 ft)												
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Girder 8	Girder 9	Total	Max
no R	Girder	543.1	582.3	609.2	609.5	578.1	530.5	436.9	314.4	199.3	4403.0	609.5
	Slab	28.7	32.6	33.7	35.1	36.8	36.4	28.9	16.0	7.9	256.2	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	571.8	614.9	642.9	644.6	614.8	566.9	465.8	330.4	207.2	4659.2	644.6
RL	Girder	384.6	488.5	559.8	587.4	572.5	533.7	444.0	322.9	208.5	4101.8	587.4
	Slab	23.2	32.1	34.8	36.6	38.2	37.5	29.7	16.5	8.3	256.9	
	Railling	300.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	300.5	
	Total	708.3	520.5	594.5	624.1	610.7	571.2	473.7	339.4	216.8	4659.2	624.1
RR	Girder	545.6	584.5	610.7	609.7	575.5	522.9	421.1	287.4	160.6	4318.2	610.7
	Slab	28.9	32.8	33.9	35.4	37.1	36.7	29.0	15.5	5.9	255.3	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	85.8	85.8	
	Total	574.5	617.3	644.7	645.1	612.7	559.7	450.1	302.9	252.4	4659.2	645.1
2R	Girder	386.4	490.3	561.2	587.6	569.8	525.8	427.3	294.3	166.9	4009.6	587.6
	Slab	23.3	32.2	35.0	36.9	38.6	37.9	29.8	16.0	6.3	256.0	
	Railling	302.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	91.4	393.6	
	Total	711.9	522.5	596.2	624.5	608.4	563.7	457.2	310.3	264.5	4659.2	624.5

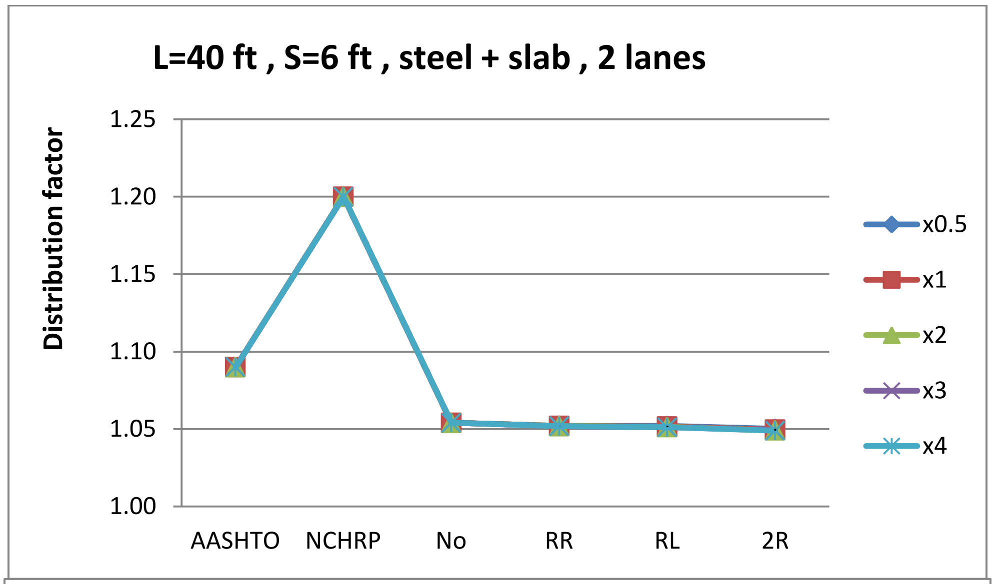
Table A.8. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 4 lanes, Span L=80 ft, Spacing S = 8 ft)

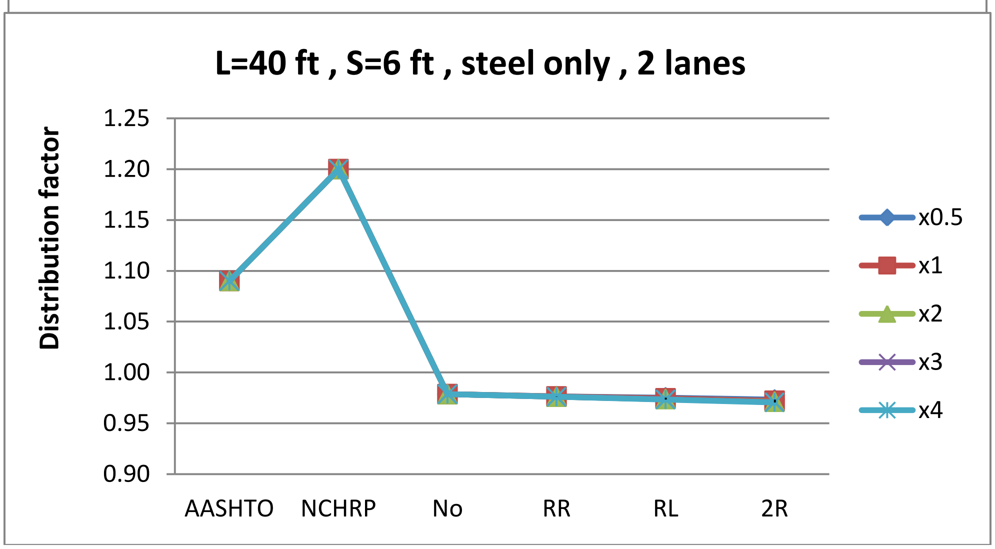
(Stiffness x1, 4 Lanes, L=80 ft, S = 8 ft)										
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Girder 6	Girder 7	Total	Max
no R	Girder	680.7	753.3	767.5	735.4	645.9	471.1	282.1	4336.1	767.5
	Slab	42.2	54.4	62.7	62.7	52.8	34.8	13.6	323.2	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	722.9	807.7	830.2	798.1	698.7	505.9	295.7	4659.2	830.2
RL	Girder	467.6	642.4	718.7	721.0	648.9	481.6	296.4	3976.6	721.0
	Slab	33.9	53.3	64.3	64.7	54.5	35.9	14.4	320.9	
	Railling	361.8	0.0	0.0	0.0	0.0	0.0	0.0	361.8	
	Total	863.3	695.7	783.0	785.7	703.3	517.5	310.8	4659.2	785.7
RR	Girder	685.4	756.4	767.6	729.2	627.2	431.8	217.0	4214.7	767.6
	Slab	42.5	54.8	63.3	63.2	53.1	34.0	10.2	321.0	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	123.5	123.5	
	Total	727.9	811.2	830.9	792.4	680.3	465.7	350.8	4659.2	830.9
2R	Girder	470.7	644.8	718.5	714.4	629.0	439.7	225.9	3843.0	718.5
	Slab	34.1	53.7	64.8	65.3	54.8	35.1	10.8	318.8	
	Railling	365.0	0.0	0.0	0.0	0.0	0.0	132.4	497.5	
	Total	869.8	698.5	783.4	779.7	683.9	474.7	369.2	4659.2	783.4

Table A.9. Calculated Bending Moments (kip-ft) at Critical Section Based on FEA Results (Stiffness x1, 4 lanes, Span L=80 ft, Spacing S = 12 ft)

(Stiffness x1, 4 Lanes, L=80 ft, S = 12 ft)								
Case	Zone	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5	Total	Max
no R	Girder	914.4	1047.5	1013.2	793.6	441.7	4210.3	1047.5
	Slab	73.4	120.0	128.5	96.3	30.7	448.9	
	Railling	0.0	0.0	0.0	0.0	0.0	0.0	
	Total	987.8	1167.5	1141.7	889.9	472.4	4659.3	1167.5
RL	Girder	605.5	919.7	975.9	797.0	463.1	3761.2	975.9
	Slab	59.1	117.6	130.9	98.8	32.2	438.6	
	Railling	459.5	0.0	0.0	0.0	0.0	459.5	
	Total	1124.0	1037.3	1106.8	895.8	495.3	4659.3	1106.8
RR	Girder	922.9	1047.8	995.5	736.6	320.9	4023.7	1047.8
	Slab	74.1	121.0	129.3	94.7	23.9	442.9	
	Railling	0.0	0.0	0.0	0.0	192.7	192.7	
	Total	997.0	1168.8	1124.7	831.2	537.5	4659.3	1168.8
2R	Girder	610.4	918.6	956.8	736.4	332.7	3555.0	956.8
	Slab	59.6	118.7	131.8	97.2	25.1	432.4	
	Railling	465.5	0.0	0.0	0.0	206.4	671.9	
	Total	1135.5	1037.2	1088.6	833.6	564.3	4659.3	1088.6

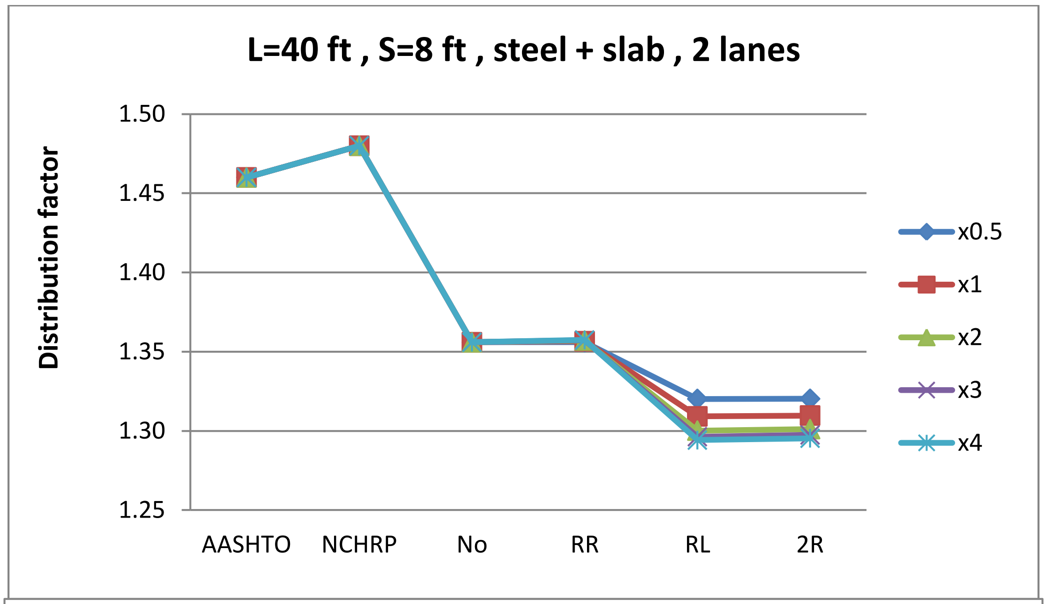
	DF=Mmax/M0									
	L=40 ft , S=6ft , steel + slab 2 lanes					L=40 ft , S=6ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
No	1.05	1.05	1.05	1.05	1.05	0.98	0.98	0.98	0.98	0.98
RR	1.05	1.05	1.05	1.05	1.05	0.98	0.98	0.98	0.98	0.98
RL	1.05	1.05	1.05	1.05	1.05	0.98	0.97	0.97	0.97	0.97
2R	1.05	1.05	1.05	1.05	1.05	0.97	0.97	0.97	0.97	0.97

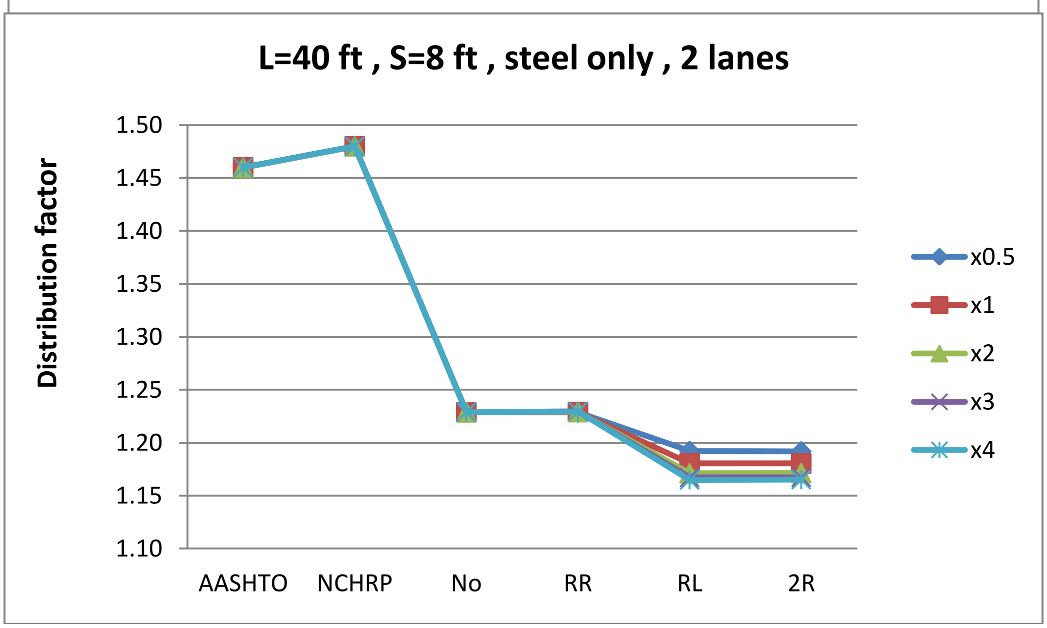




A.13. Sensitivity of Distribution Factor to Railing Stiffness (L=40ft , s=6ft , 2 lanes)

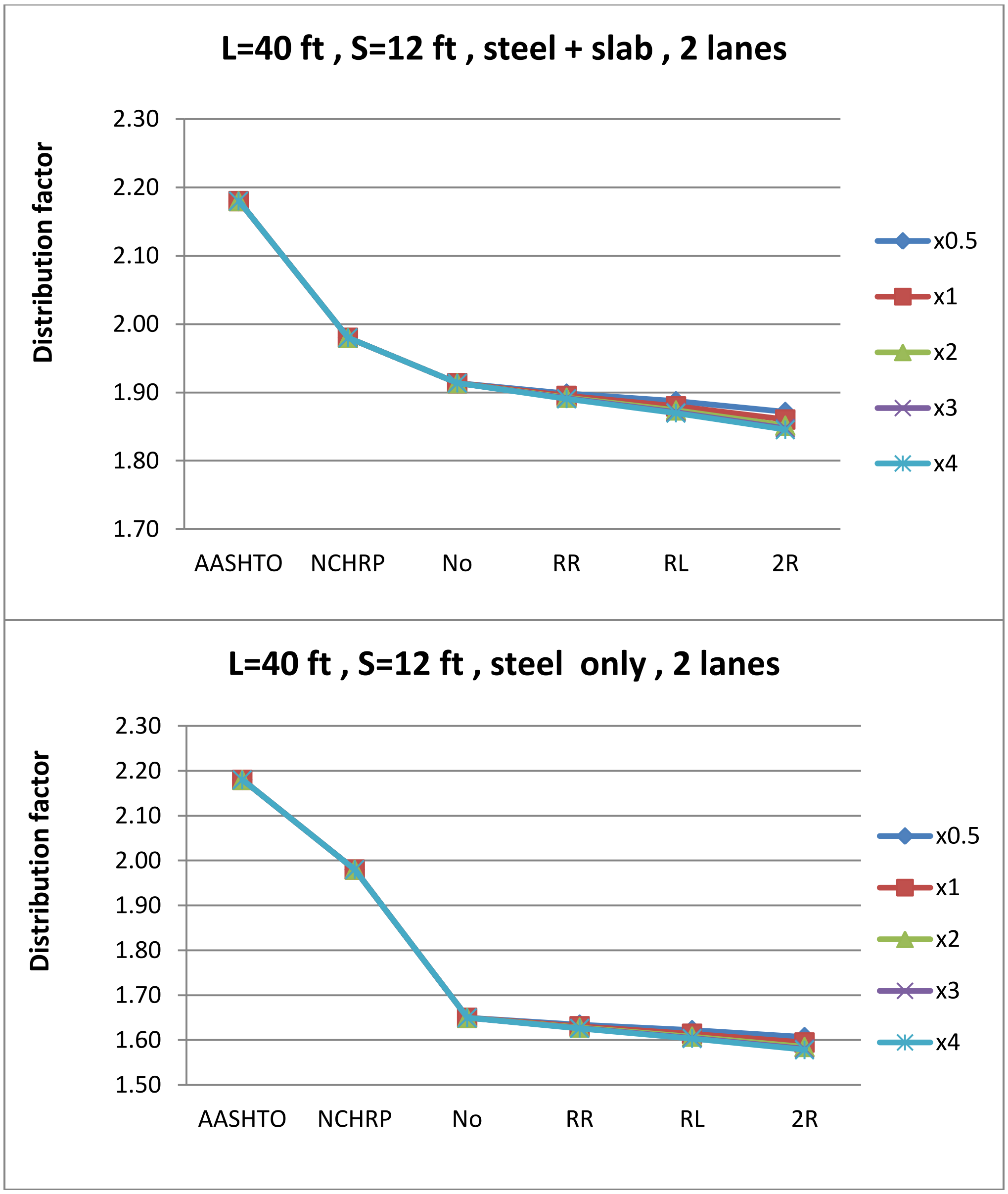
	DF=Mmax/M0									
	L=40 ft , S=8ft , steel + slab , 2 lanes					L=40 ft , S=8ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48
No	1.36	1.36	1.36	1.36	1.36	1.23	1.23	1.23	1.23	1.23
RR	1.36	1.36	1.36	1.36	1.36	1.23	1.23	1.23	1.23	1.23
RL	1.32	1.31	1.30	1.30	1.29	1.19	1.18	1.17	1.17	1.16
2R	1.32	1.31	1.30	1.30	1.30	1.19	1.18	1.17	1.17	1.17





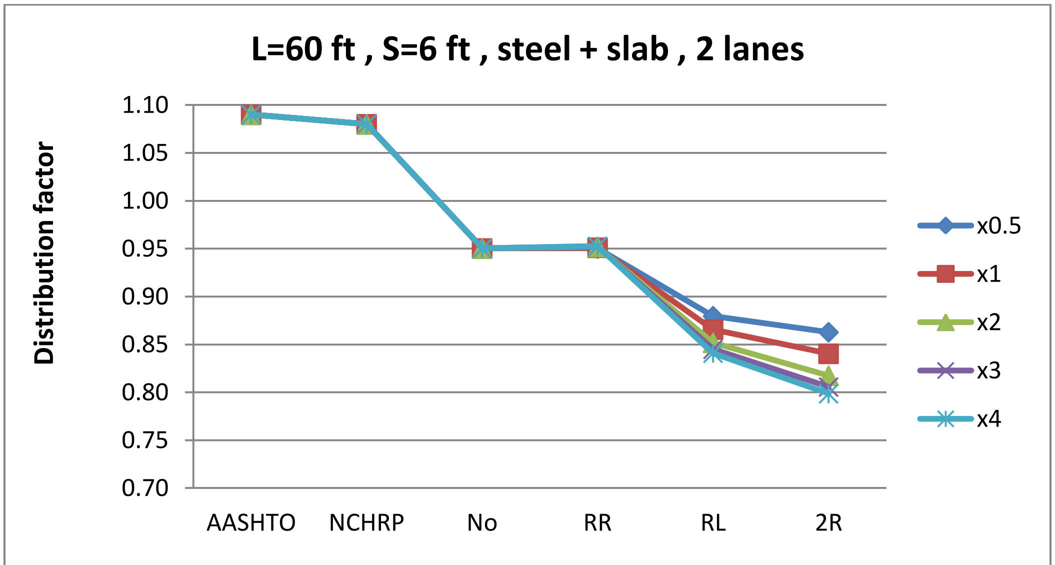
A.11. Sensitivity of Distribution Factor to Railing Stiffness (L=40ft , s=8ft , 2 lanes)

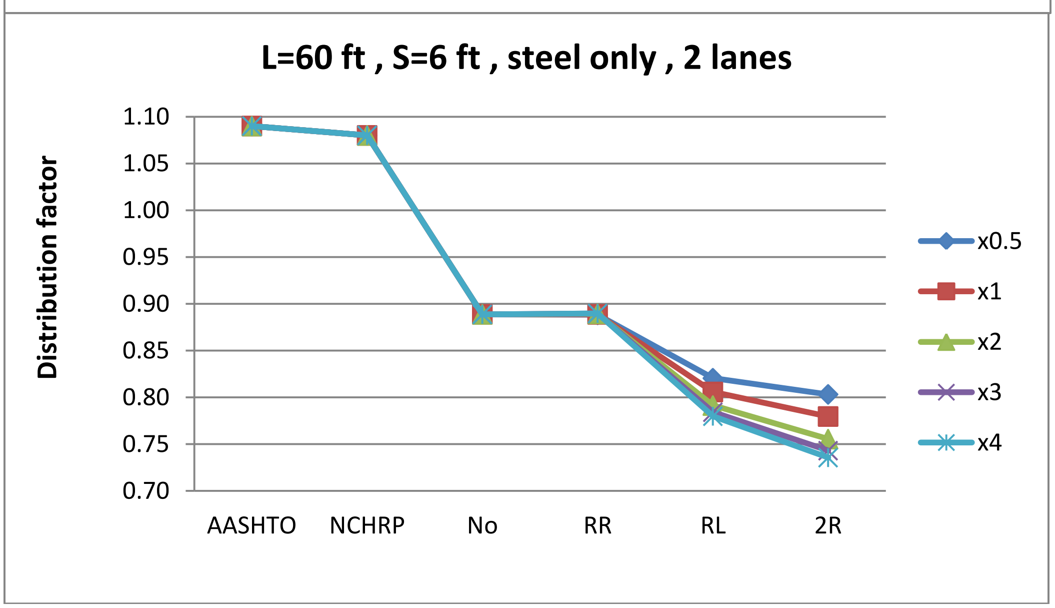
	DF=Mmax/M0									
	L=40 ft , S=12ft , steel + slab , 2 lanes					L=40 ft , S=12ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98
No	1.91	1.91	1.91	1.91	1.91	1.65	1.65	1.65	1.65	1.65
RR	1.90	1.90	1.89	1.89	1.89	1.63	1.63	1.63	1.63	1.63
RL	1.89	1.88	1.87	1.87	1.87	1.62	1.61	1.61	1.60	1.60
2R	1.87	1.86	1.85	1.85	1.85	1.61	1.59	1.58	1.58	1.58



A.12. Sensitivity of Distribution Factor to Railing Stiffness (L=40ft , s=12ft , 2 lanes)

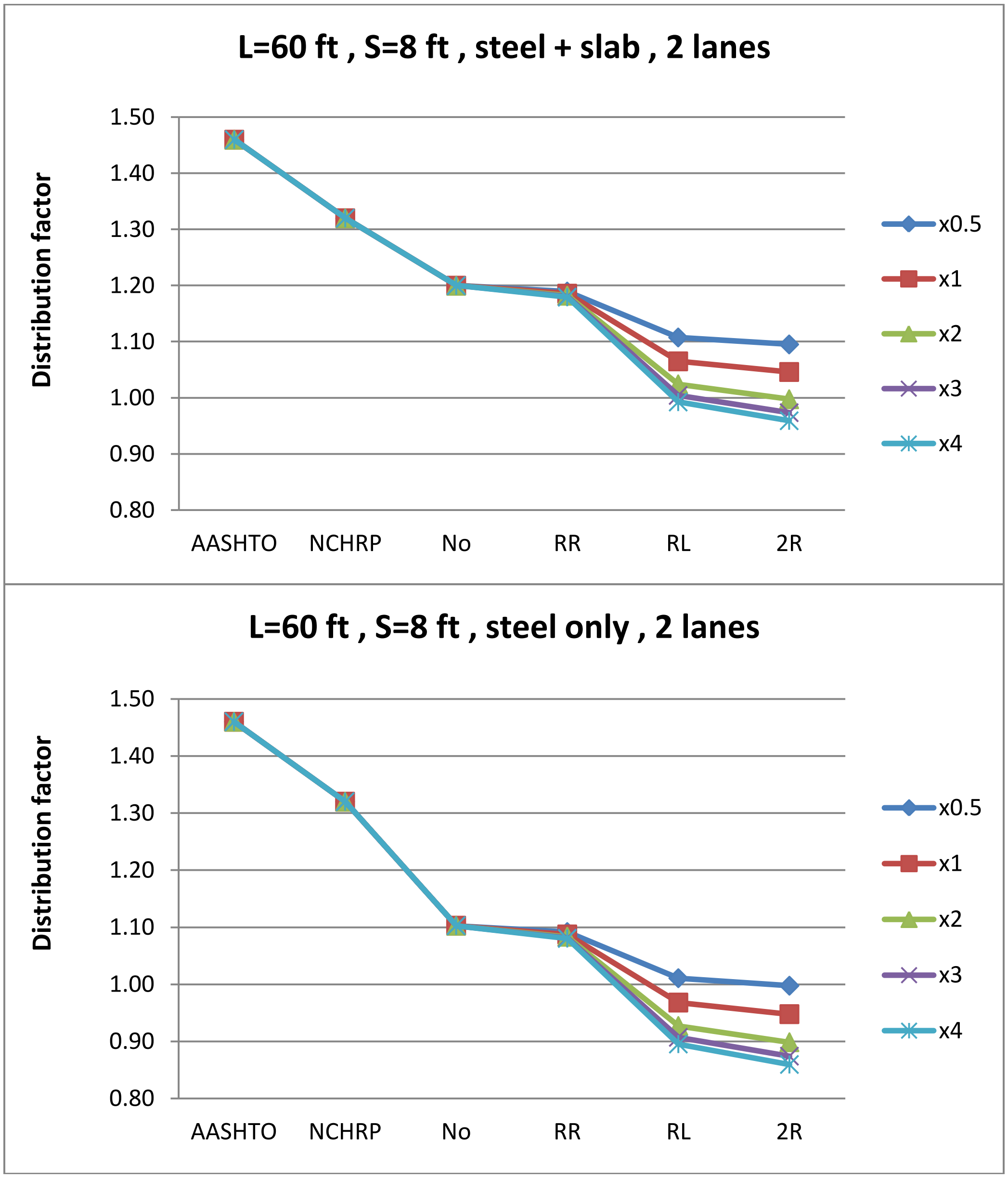
	DF=Mmax/M0									
	L=60 ft , S=6ft , steel + slab , 2 lanes					L=60 ft , S=6ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08
No	0.95	0.95	0.95	0.95	0.95	0.89	0.89	0.89	0.89	0.89
RR	0.95	0.95	0.95	0.95	0.95	0.89	0.89	0.89	0.89	0.89
RL	0.88	0.87	0.85	0.85	0.84	0.82	0.81	0.79	0.78	0.78
2R	0.86	0.84	0.82	0.81	0.80	0.80	0.78	0.76	0.74	0.74





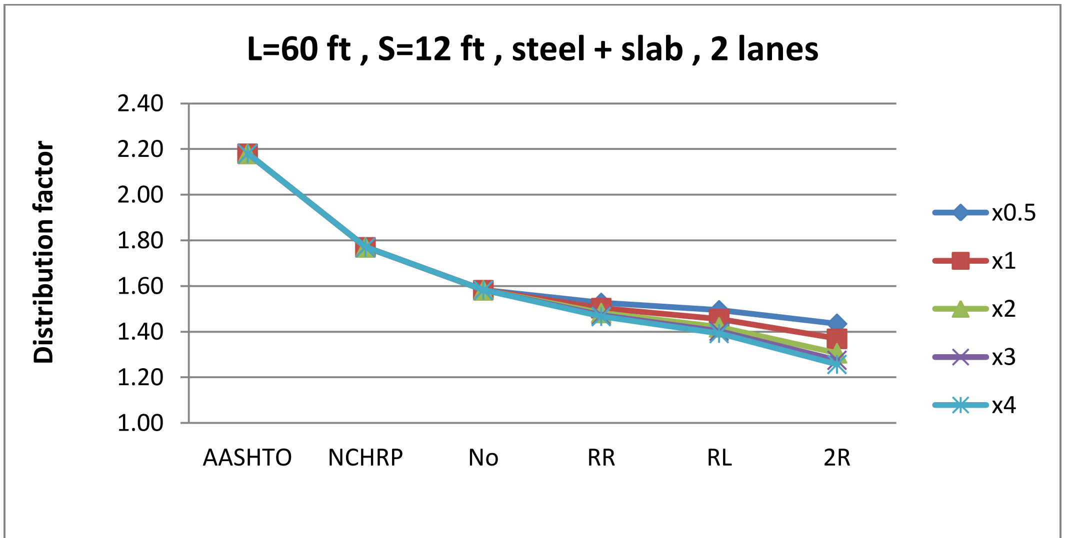
A.13. Sensitivity of Distribution Factor to Railing Stiffness (L=60ft , s=6ft , 2 lanes)

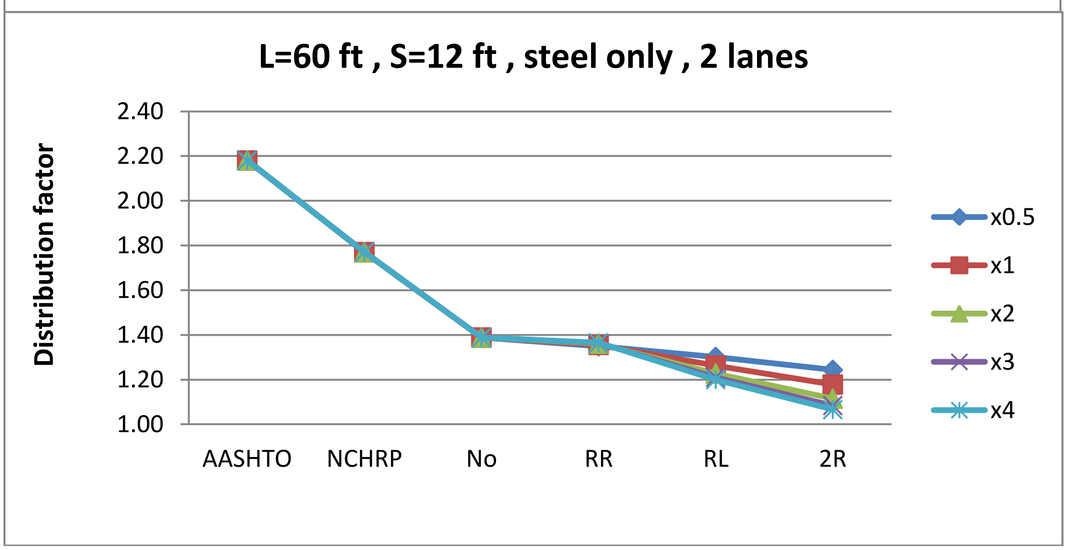
	DF=Mmax/M0									
	L=60 ft , S=8ft , steel + slab , 2 lanes					L=60 ft , S=8ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32
No	1.20	1.20	1.20	1.20	1.20	1.10	1.10	1.10	1.10	1.10
RR	1.19	1.19	1.18	1.18	1.18	1.09	1.09	1.08	1.08	1.08
RL	1.11	1.07	1.02	1.00	0.99	1.01	0.97	0.93	0.91	0.90
2R	1.10	1.05	1.00	0.97	0.96	1.00	0.95	0.90	0.87	0.86



A.14. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=8ft , 2 lanes)

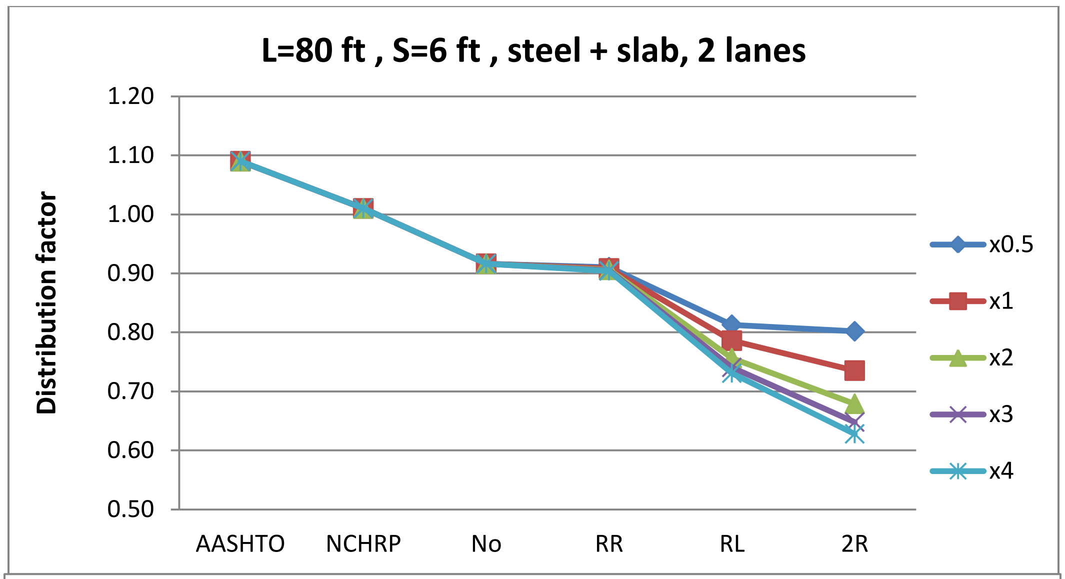
	DF=Mmax/M0									
	L=60 ft , S=12ft , steel + slab , 2 lanes					L=60 ft , S=12ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77
No	1.58	1.58	1.58	1.58	1.58	1.39	1.39	1.39	1.39	1.39
RR	1.53	1.50	1.48	1.47	1.47	1.35	1.36	1.36	1.36	1.36
RL	1.49	1.46	1.42	1.40	1.39	1.30	1.26	1.23	1.21	1.20
2R	1.44	1.37	1.31	1.28	1.26	1.24	1.18	1.11	1.08	1.07

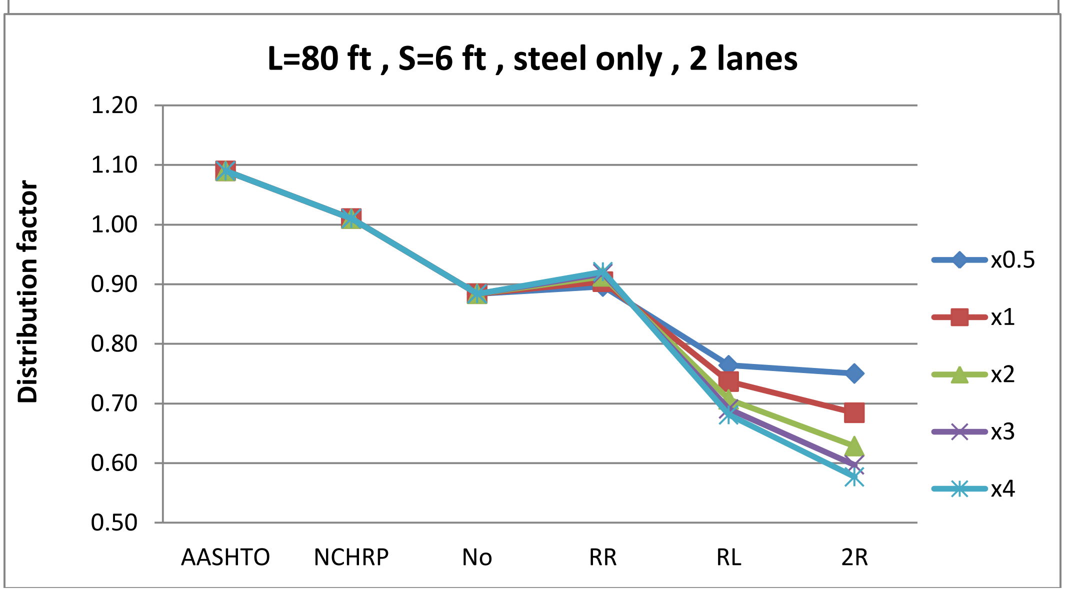




A.14. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=12ft , 2 lanes)

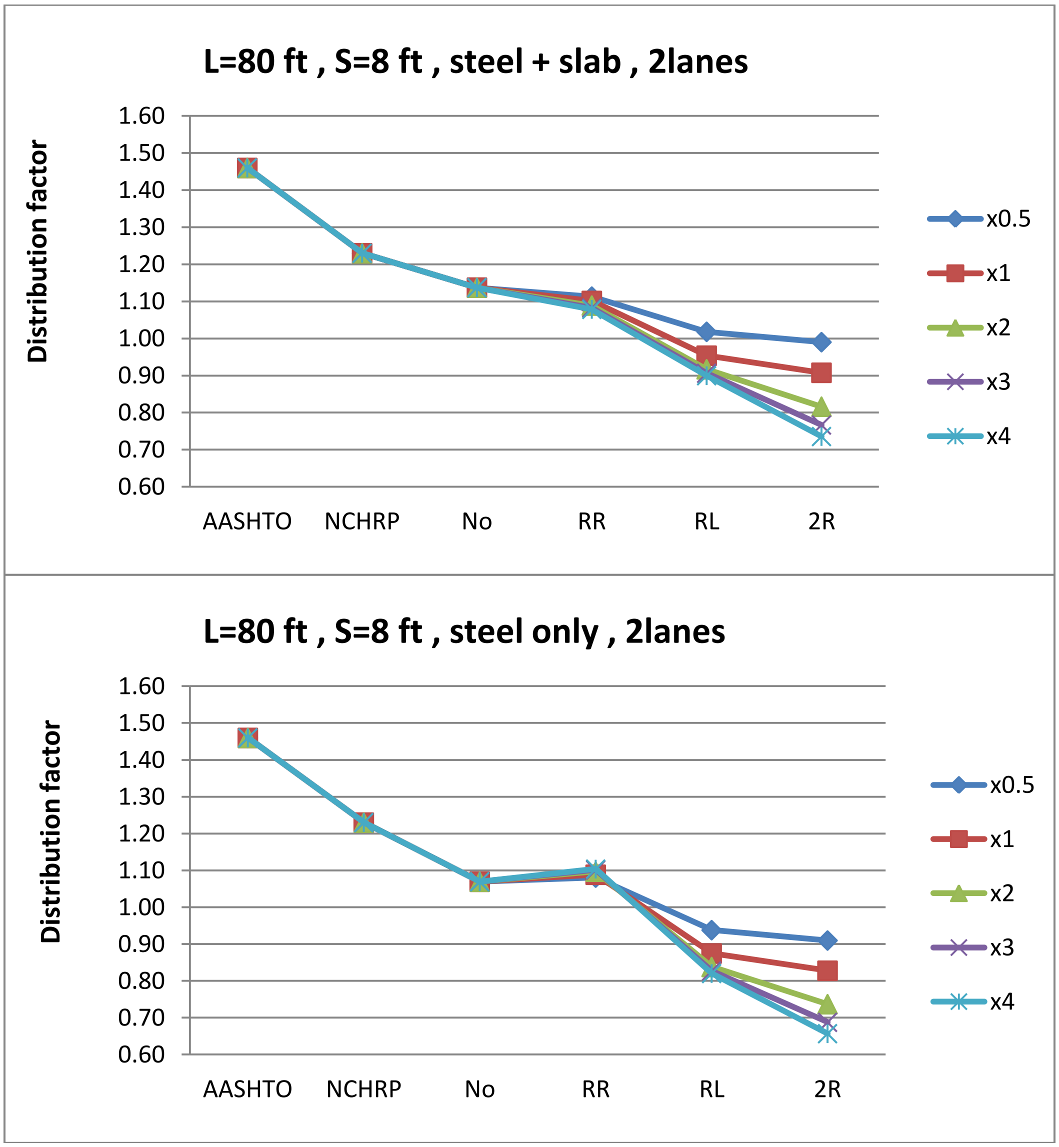
	DF=Mmax/M0									
	L=80 ft , S=6ft , steel + slab , 2 lanes					L=80 ft , S=6ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
No	0.92	0.92	0.92	0.92	0.92	0.88	0.88	0.88	0.88	0.88
RR	0.91	0.91	0.91	0.90	0.90	0.90	0.90	0.91	0.92	0.92
RL	0.81	0.79	0.76	0.74	0.73	0.76	0.74	0.71	0.69	0.68
2R	0.80	0.74	0.68	0.65	0.63	0.75	0.68	0.63	0.60	0.58





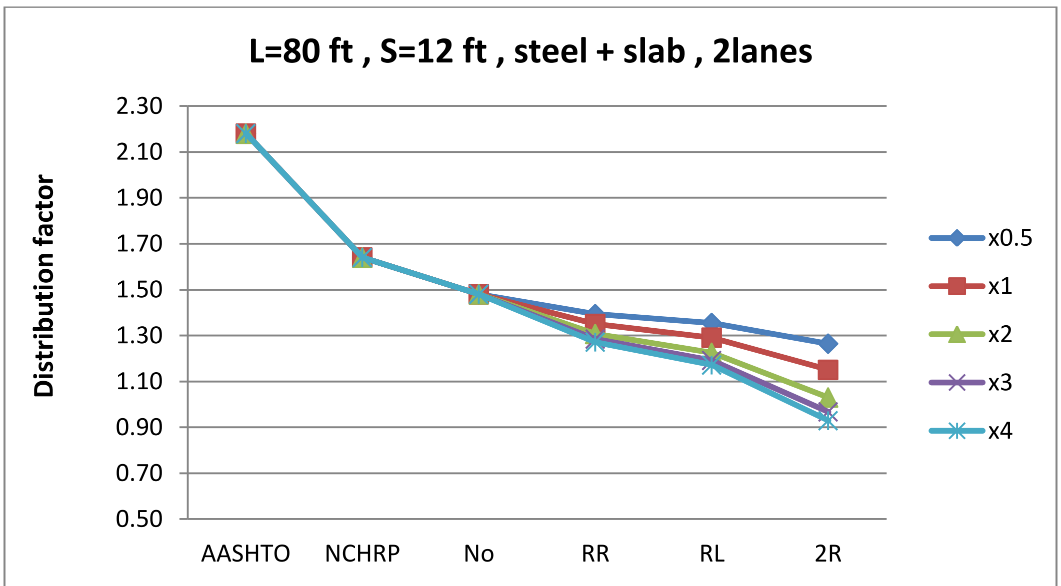
A.15. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=6ft , 2 lanes)

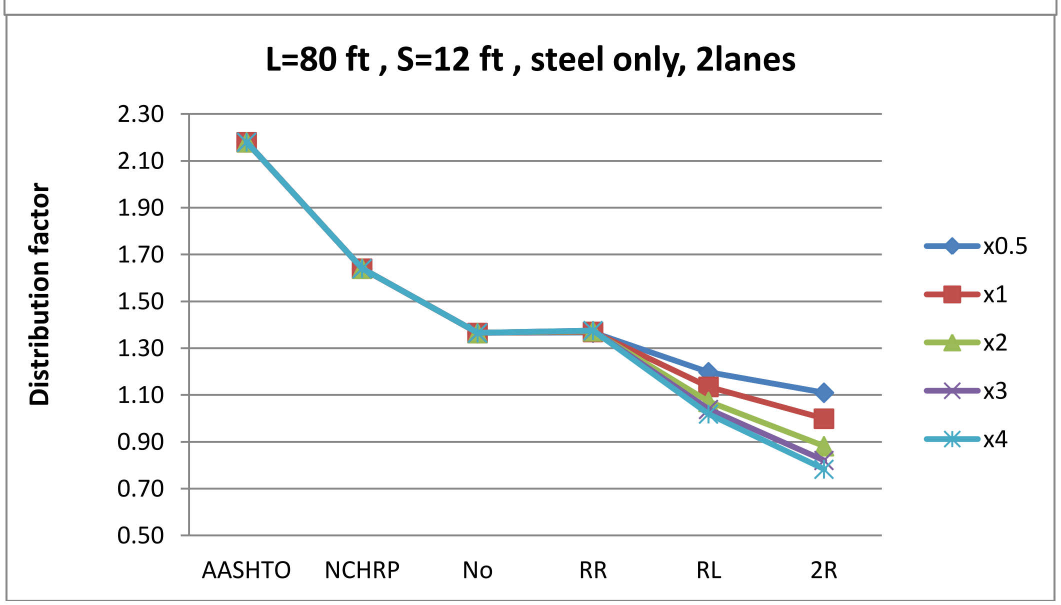
	DF=Mmax/M0									
	L=80 ft , S=8ft , steel + slab , 2 lanes					L=80 ft , S=8ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23
No	1.14	1.14	1.14	1.14	1.14	1.07	1.07	1.07	1.07	1.07
RR	1.11	1.10	1.09	1.08	1.08	1.08	1.09	1.10	1.10	1.10
RL	1.02	0.95	0.92	0.91	0.90	0.94	0.87	0.84	0.83	0.82
2R	0.99	0.91	0.82	0.77	0.74	0.91	0.83	0.74	0.69	0.66



A.16. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=8ft , 2 lanes)

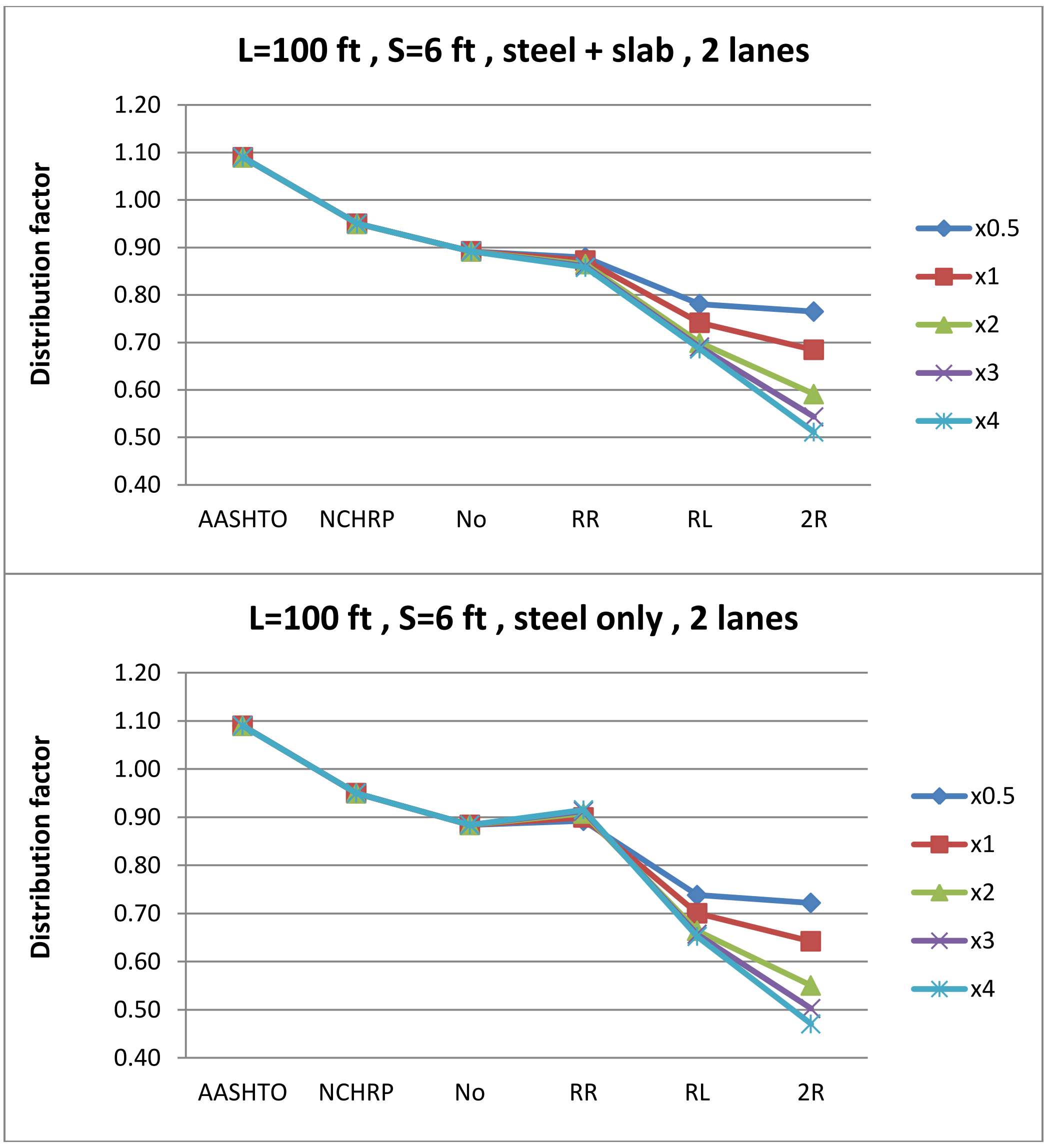
	DF=Mmax/M0									
	L=80 ft , S=12ft , steel + slab , 2 lanes					L=80 ft , S=12ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
No	1.48	1.48	1.48	1.48	1.48	1.37	1.37	1.37	1.37	1.37
RR	1.39	1.35	1.31	1.28	1.27	1.37	1.37	1.37	1.37	1.38
RL	1.35	1.29	1.22	1.19	1.17	1.20	1.14	1.07	1.04	1.02
2R	1.26	1.15	1.03	0.97	0.93	1.11	1.00	0.88	0.82	0.78





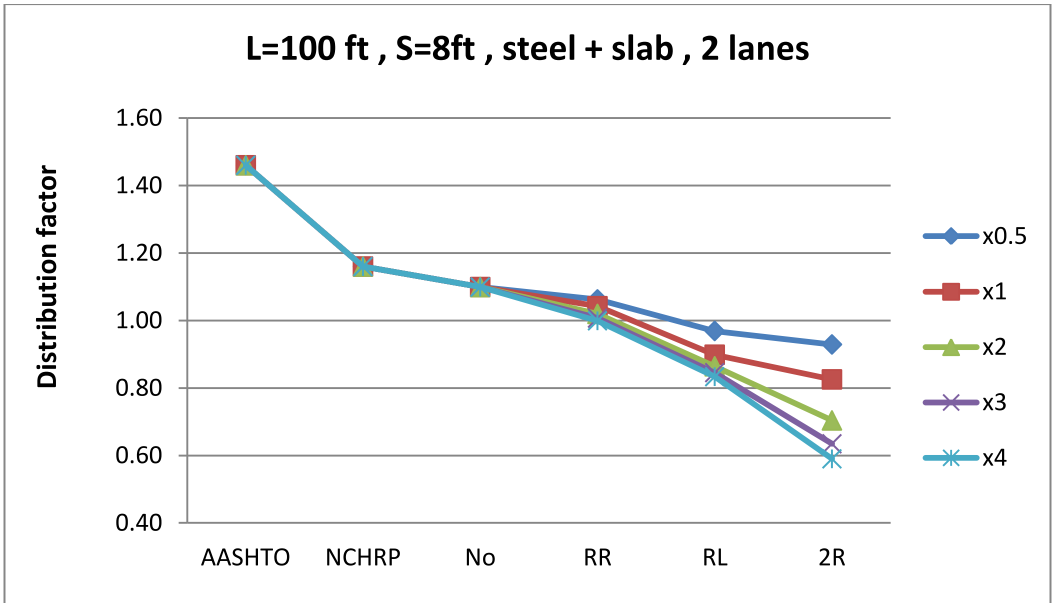
A.17. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=12ft , 2 lanes)

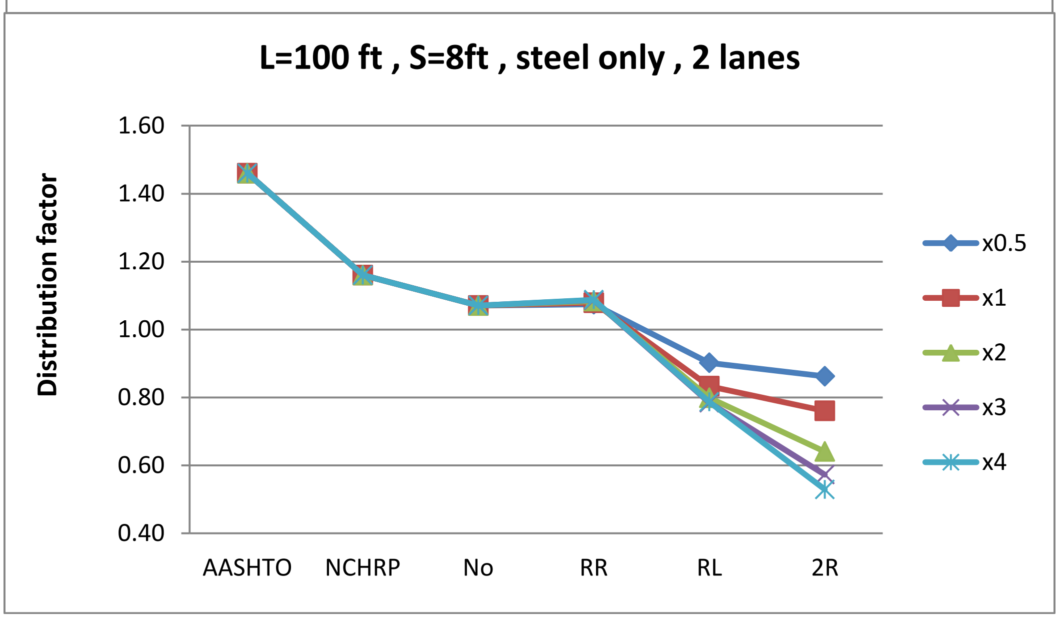
	DF=Mmax/M0									
	L=100 ft , S=6ft , steel + slab , 2 lanes					L=100 ft , S=6ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
No	0.89	0.89	0.89	0.89	0.89	0.88	0.88	0.88	0.88	0.88
RR	0.88	0.87	0.86	0.86	0.86	0.89	0.90	0.91	0.91	0.92
RL	0.78	0.74	0.70	0.69	0.69	0.74	0.70	0.66	0.66	0.65
2R	0.76	0.68	0.59	0.54	0.51	0.72	0.64	0.55	0.50	0.47



A.18. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=6ft , 2 lanes)

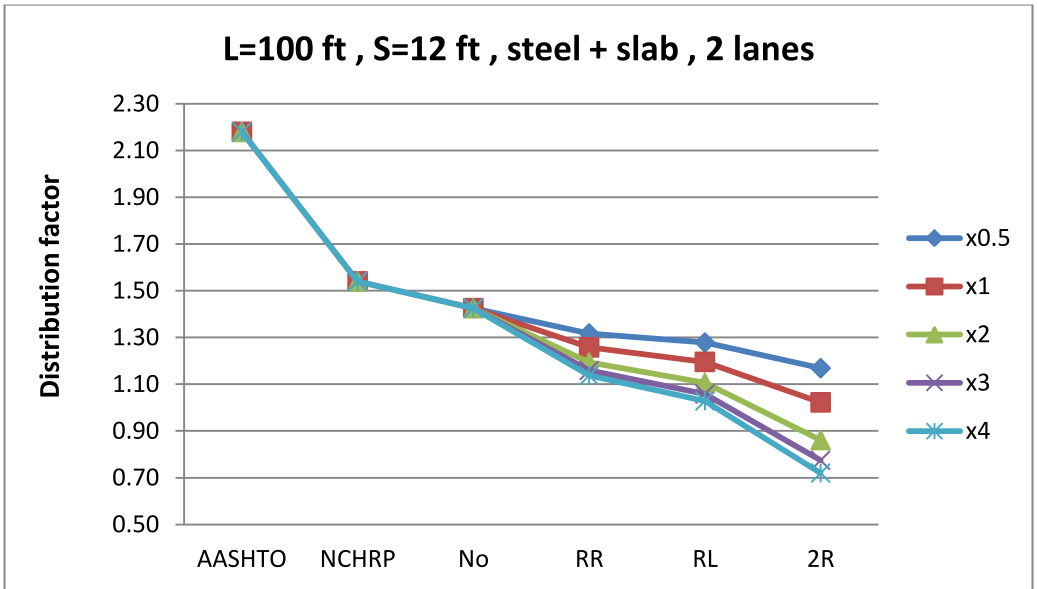
	DF=Mmax/M0									
	L=100 ft , S=8ft , steel + slab , 2 lanes					L=100 ft , S=8ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16
No	1.10	1.10	1.10	1.10	1.10	1.07	1.07	1.07	1.07	1.07
RR	1.06	1.04	1.02	1.01	1.00	1.07	1.08	1.08	1.09	1.09
RL	0.97	0.90	0.86	0.85	0.83	0.90	0.83	0.80	0.78	0.79
2R	0.93	0.83	0.70	0.63	0.59	0.86	0.76	0.64	0.57	0.53

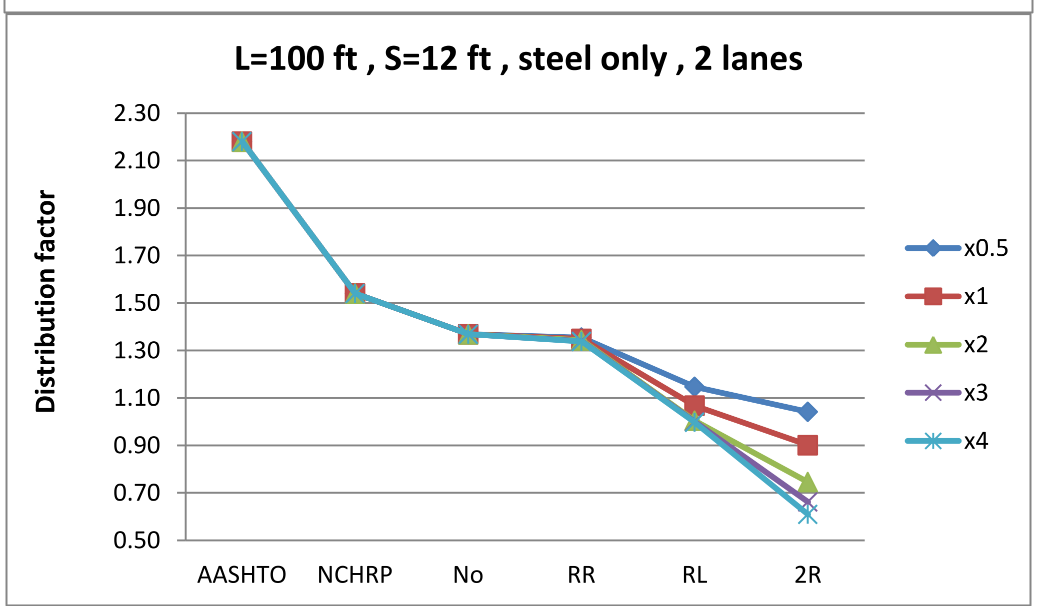




A.19. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=8ft , 2 lanes)

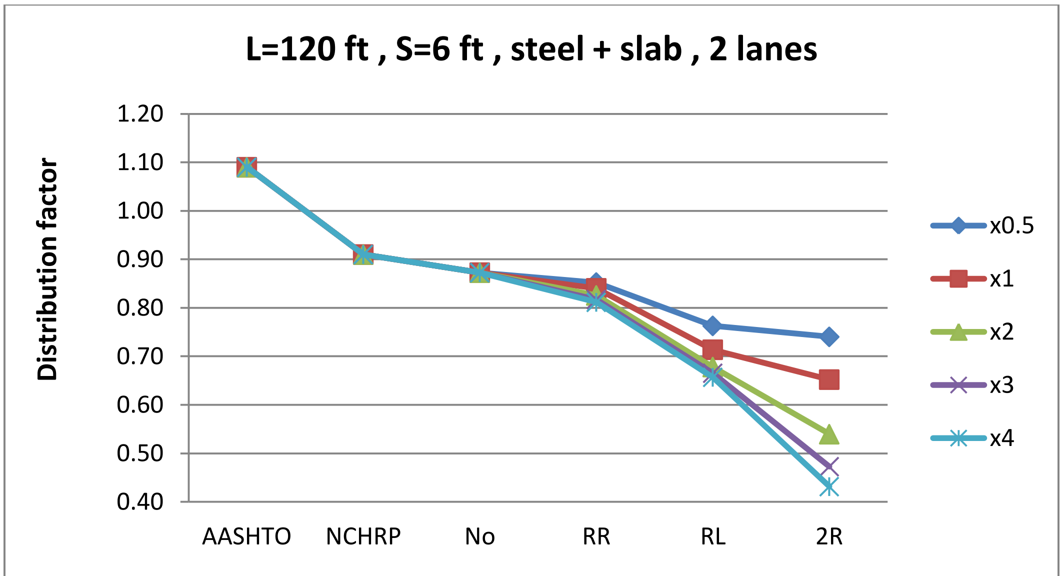
	DF=Mmax/M0									
	L=100 ft , S=12ft , steel + slab , 2 lanes					L=100 ft , S=12ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54
No	1.43	1.43	1.43	1.43	1.43	1.37	1.37	1.37	1.37	1.37
RR	1.32	1.26	1.19	1.16	1.14	1.35	1.35	1.34	1.34	1.34
RL	1.28	1.20	1.11	1.06	1.03	1.15	1.07	1.00	1.00	1.00
2R	1.17	1.02	0.86	0.77	0.72	1.04	0.90	0.75	0.66	0.61

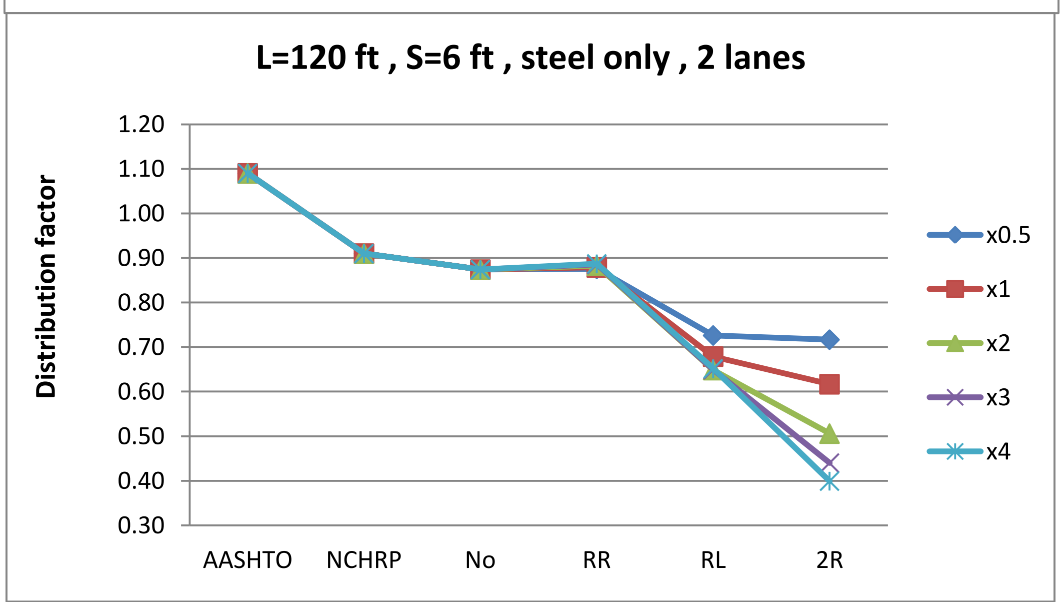




A.21. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=12ft , 2 lanes)

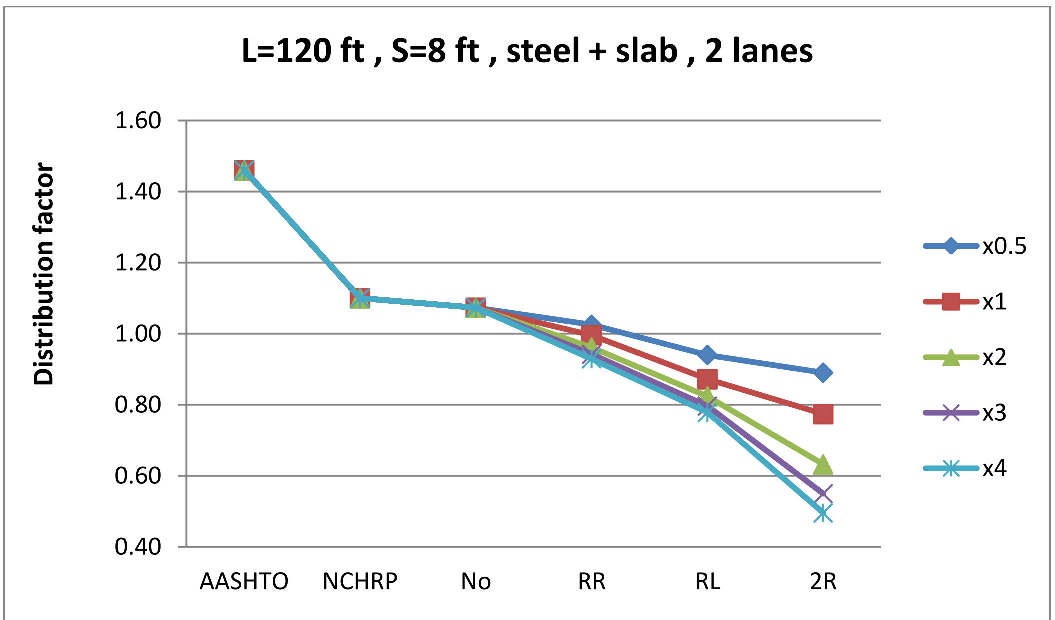
	DF=Mmax/M0									
	L=120 ft , S=6ft , steel + slab , 2 lanes					L=120 ft , S=6ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
No	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87
RR	0.85	0.84	0.83	0.82	0.81	0.88	0.88	0.88	0.89	0.89
RL	0.76	0.71	0.68	0.67	0.66	0.73	0.68	0.65	0.65	0.65
2R	0.74	0.65	0.54	0.47	0.43	0.72	0.62	0.51	0.44	0.40

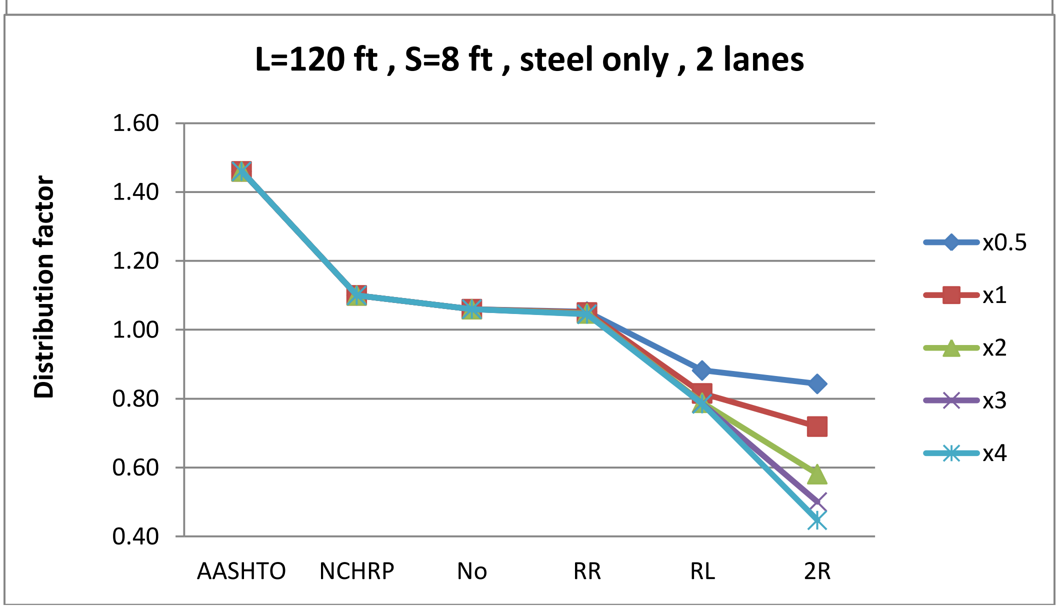




A.22. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=6ft , 2 lanes)

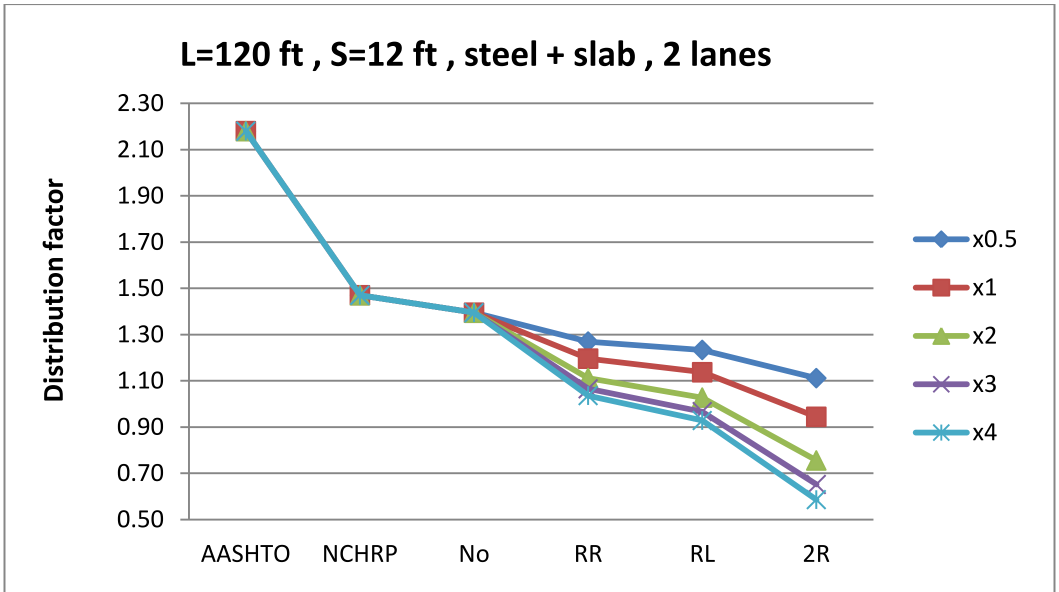
	DF=Mmax/M0									
	L=120 ft , S=8ft , steel + slab , 2 lanes					L=120 ft , S=8ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
No	1.07	1.07	1.07	1.07	1.07	1.06	1.06	1.06	1.06	1.06
RR	1.02	1.00	0.96	0.94	0.93	1.05	1.05	1.05	1.05	1.05
RL	0.94	0.87	0.82	0.80	0.78	0.88	0.82	0.79	0.79	0.78
2R	0.89	0.77	0.63	0.55	0.50	0.84	0.72	0.58	0.50	0.45

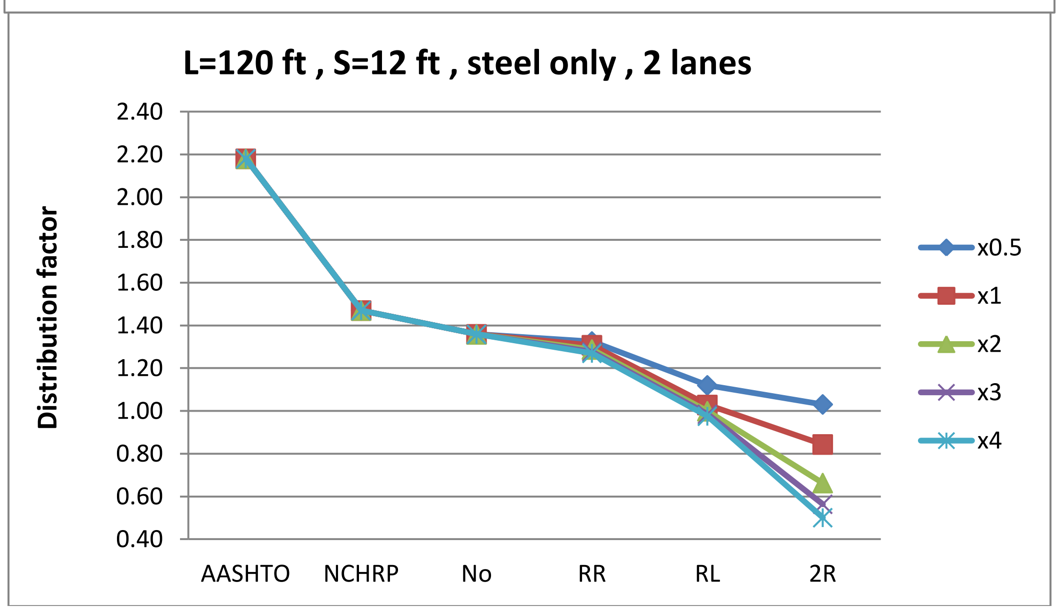




A.23. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=8ft , 2 lanes)

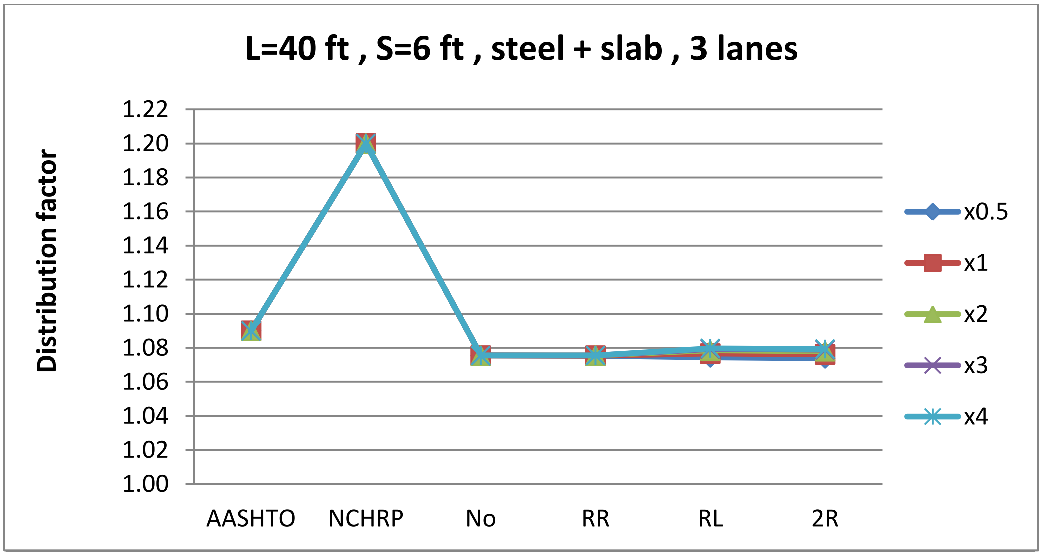
	DF=Mmax/M0									
	L=120 ft , S=12ft , steel + slab , 2 lanes					L=120 ft , S=12ft , steel only , 2 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47
No	1.39	1.39	1.39	1.39	1.39	1.36	1.36	1.36	1.36	1.36
RR	1.27	1.20	1.11	1.07	1.04	1.33	1.31	1.29	1.28	1.27
RL	1.23	1.14	1.03	0.97	0.93	1.12	1.03	1.00	0.98	0.98
2R	1.11	0.94	0.76	0.65	0.59	1.03	0.84	0.66	0.56	0.50

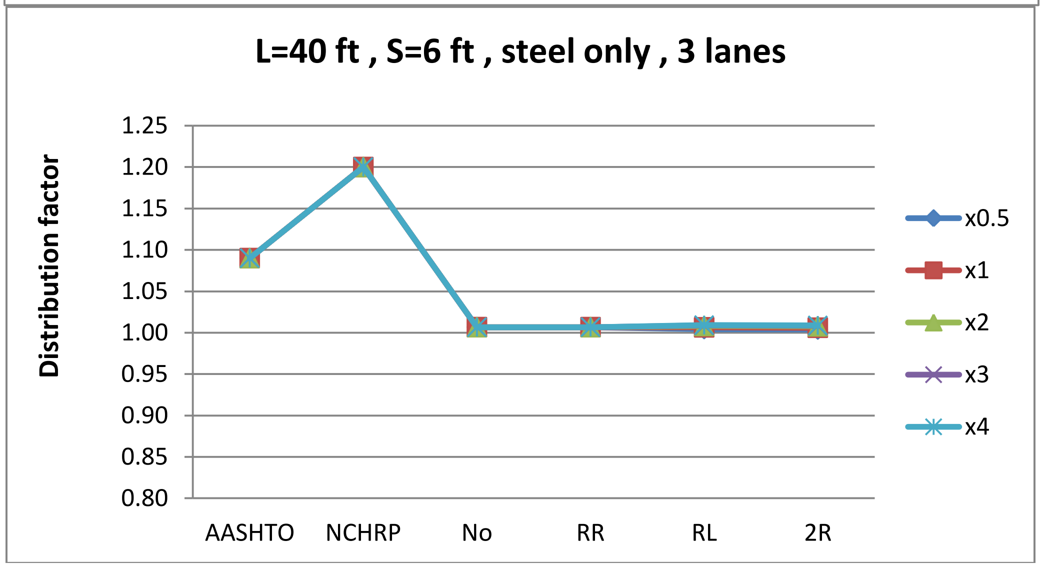




A.24. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=12ft , 2 lanes)

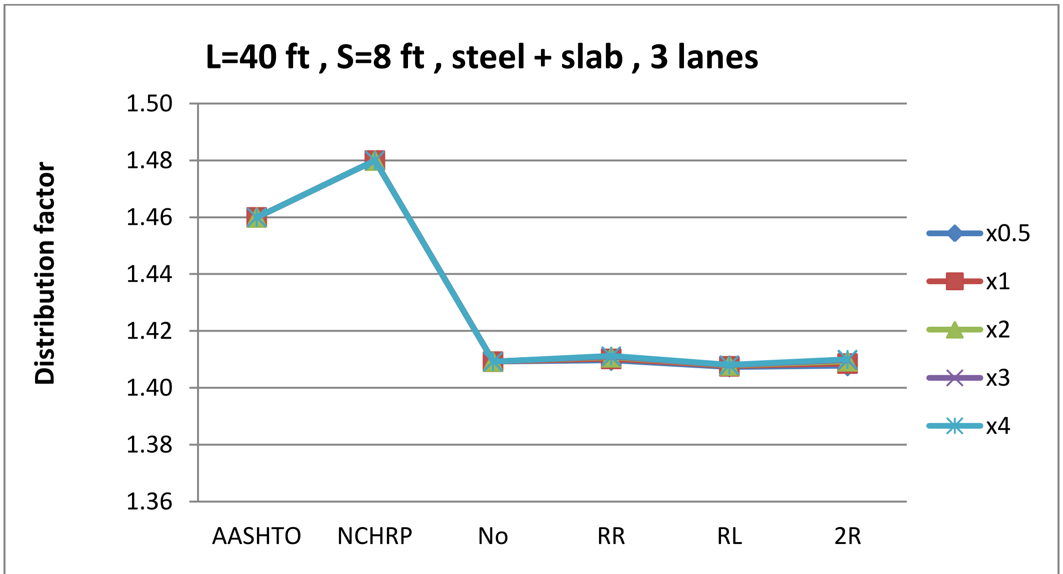
	DF=0.9*Mmax/M0									
	L=40 ft , S=6ft , steel + slab , 3 lanes					L=40 ft , S=6ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
No	1.08	1.08	1.08	1.08	1.08	1.01	1.01	1.01	1.01	1.01
RR	1.08	1.08	1.08	1.08	1.08	1.01	1.01	1.01	1.01	1.01
RL	1.07	1.08	1.08	1.08	1.08	1.00	1.01	1.01	1.01	1.01
2R	1.07	1.08	1.08	1.08	1.08	1.00	1.01	1.01	1.01	1.01

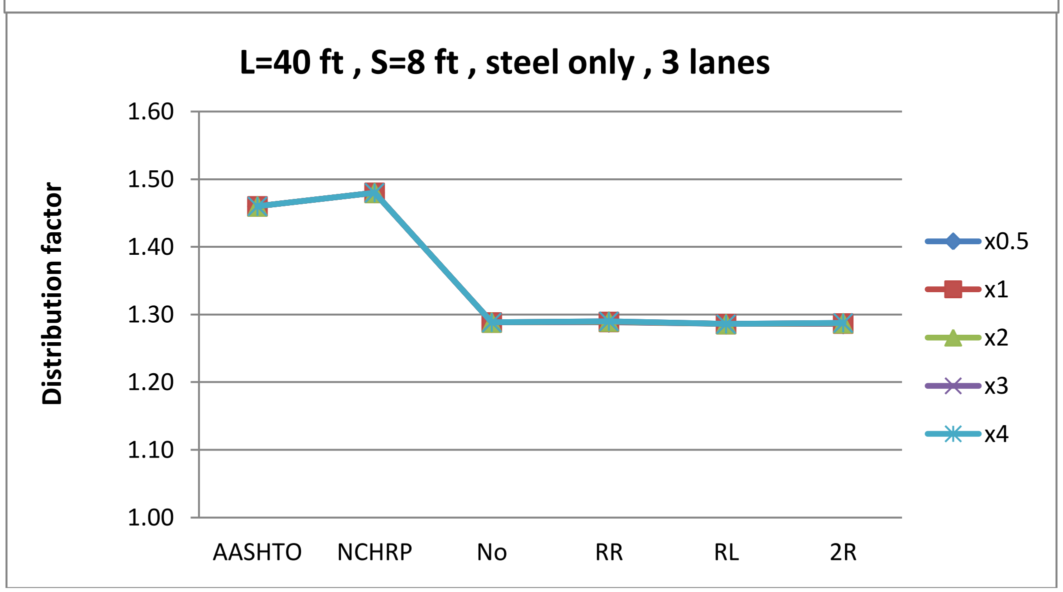




A.25. Sensitivity of Distribution Factor to railing stiffness (L=40ft , s=6ft , 3 lanes)

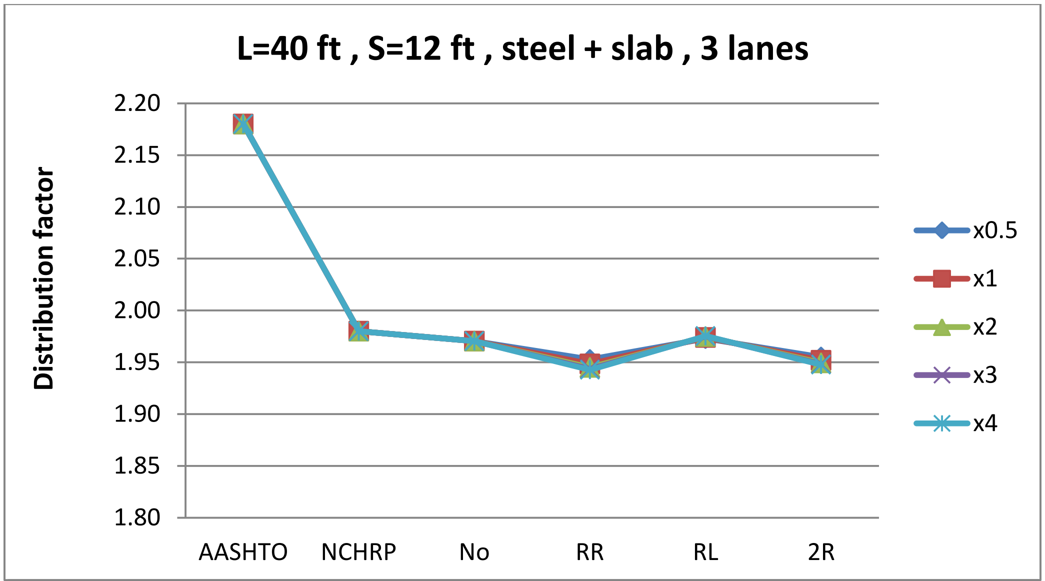
	DF=0.9*Mmax/M0									
	L=40 ft , S=8ft , steel + slab , 3 lanes					L=40 ft , S=8ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48
No	1.41	1.41	1.41	1.41	1.41	1.29	1.29	1.29	1.29	1.29
RR	1.41	1.41	1.41	1.41	1.41	1.29	1.29	1.29	1.29	1.29
RL	1.41	1.41	1.41	1.41	1.41	1.29	1.29	1.29	1.29	1.29
2R	1.41	1.41	1.41	1.41	1.41	1.29	1.29	1.29	1.29	1.29

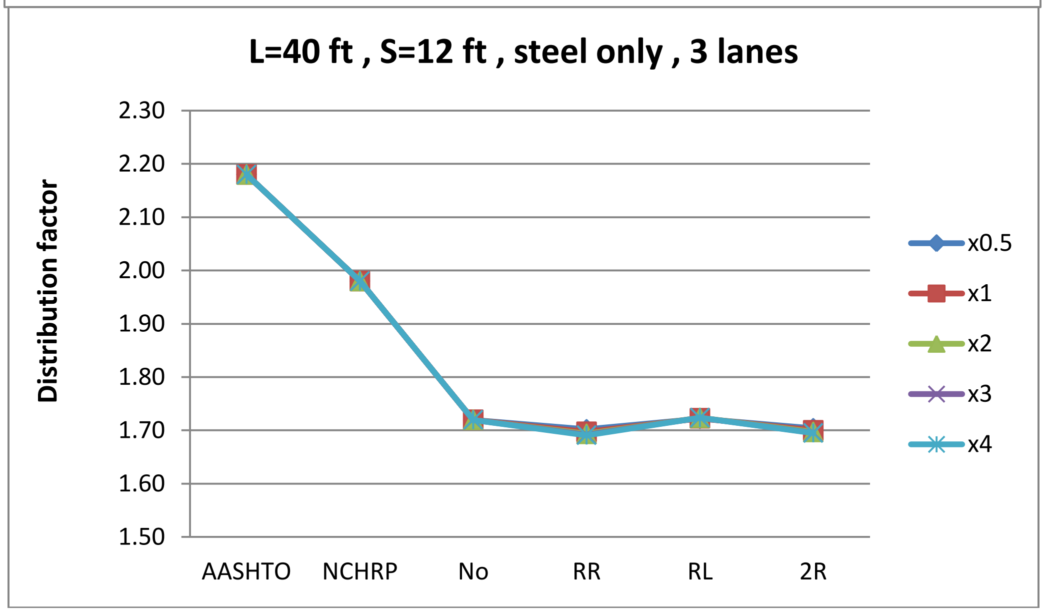




A.26. Sensitivity of Distribution Factor to railing stiffness (L=40ft , s=8ft , 3 lanes)

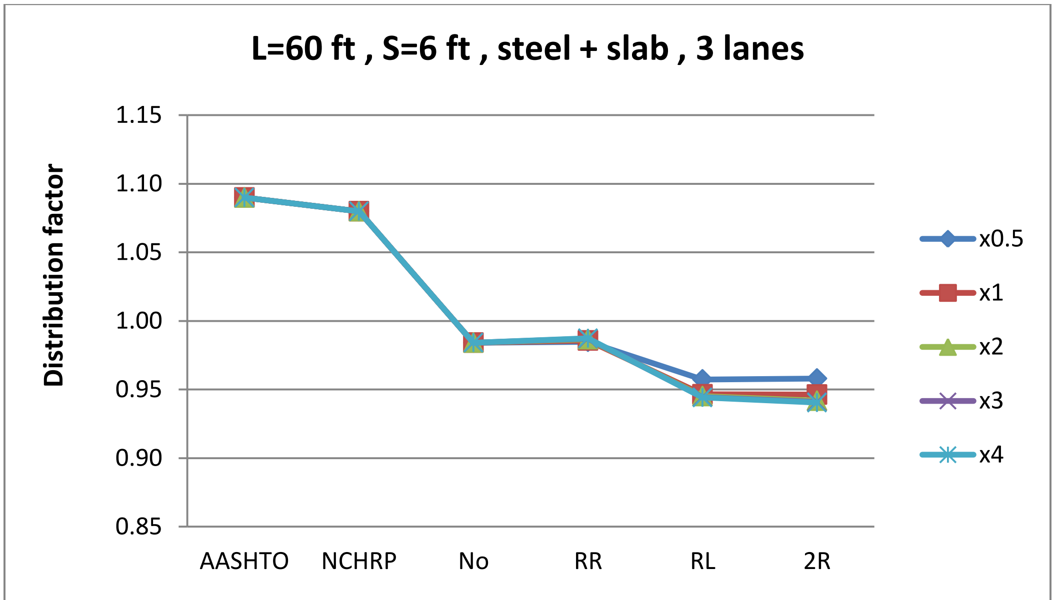
	DF=0.9*Mmax/M0									
	L=40 ft , S=12ft , steel + slab , 3 lanes					L=40 ft , S=12ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98
No	1.97	1.97	1.97	1.97	1.97	1.72	1.72	1.72	1.72	1.72
RR	1.95	1.95	1.94	1.94	1.94	1.70	1.70	1.69	1.69	1.69
RL	1.97	1.97	1.97	1.98	1.98	1.72	1.72	1.72	1.72	1.72
2R	1.96	1.95	1.95	1.95	1.95	1.70	1.70	1.70	1.70	1.69

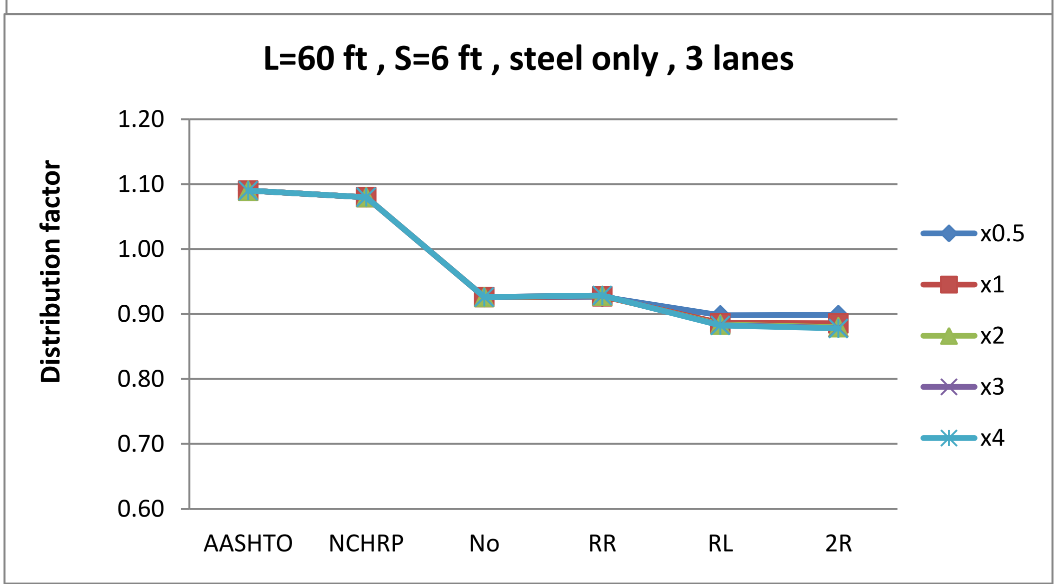




A.27. Sensitivity of Distribution Factor to railing stiffness (L=40ft , s=12ft , 3 lanes)

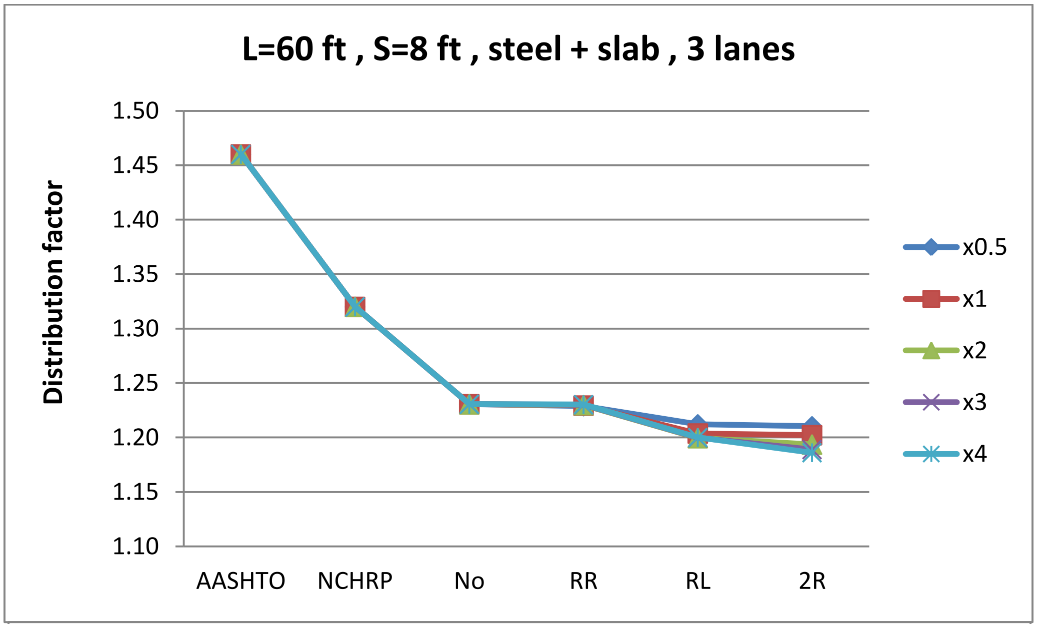
	DF=0.9*Mmax/M0									
	L=60 ft , S=6ft , steel + slab , 3 lanes					L=60 ft , S=6ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08
No	0.98	0.98	0.98	0.98	0.98	0.93	0.93	0.93	0.93	0.93
RR	0.98	0.99	0.99	0.99	0.99	0.93	0.93	0.93	0.93	0.93
RL	0.96	0.95	0.95	0.94	0.94	0.90	0.89	0.88	0.88	0.88
2R	0.96	0.95	0.94	0.94	0.94	0.90	0.89	0.88	0.88	0.88

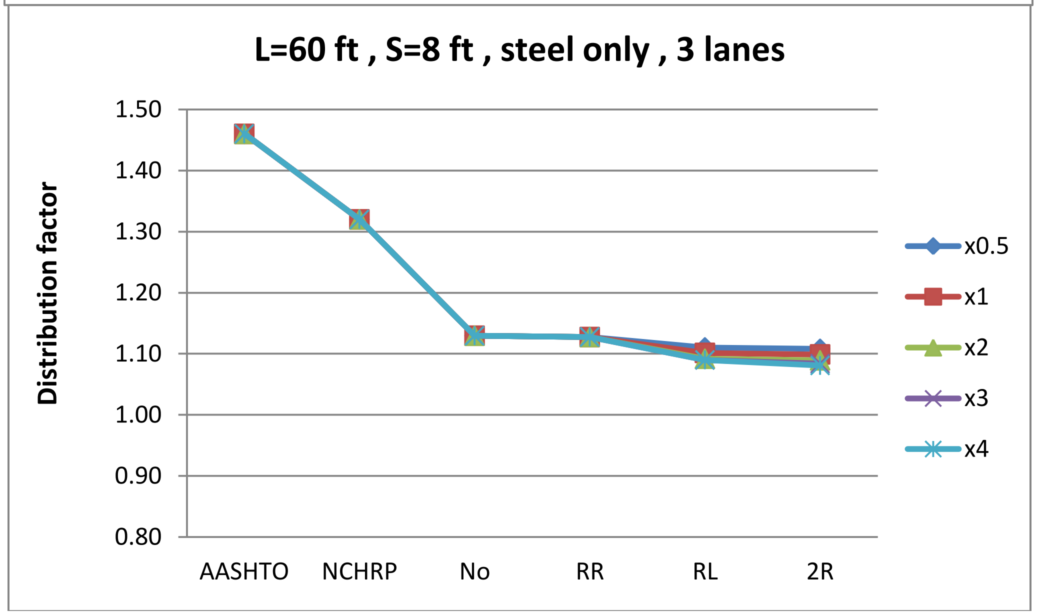




A.28. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=6ft , 3 lanes)

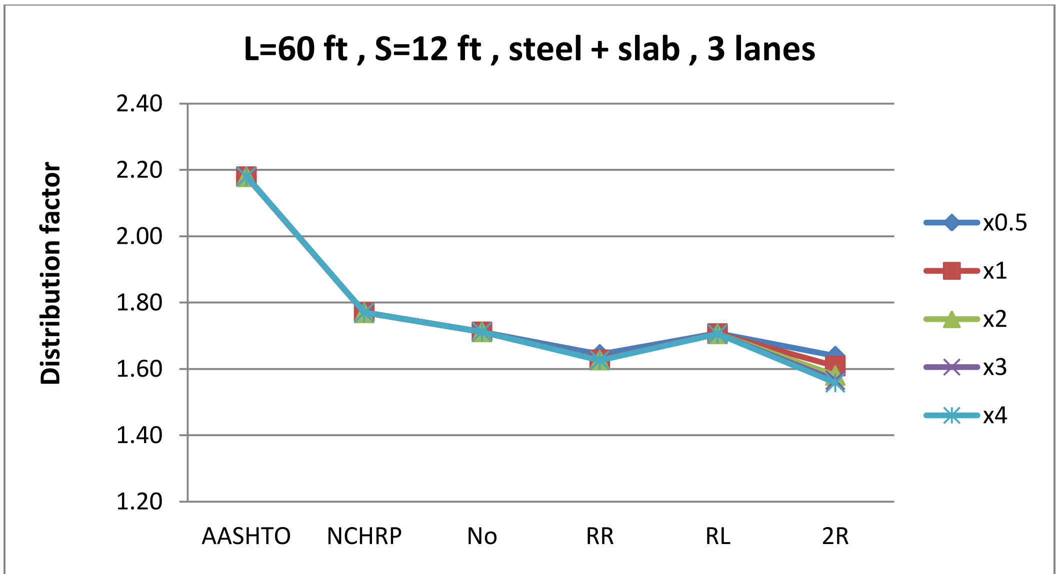
	DF=0.9*Mmax/M0									
	L=60 ft , S=8ft , steel + slab , 3 lanes					L=60 ft , S=8ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32
No	1.23	1.23	1.23	1.23	1.23	1.13	1.13	1.13	1.13	1.13
RR	1.23	1.23	1.23	1.23	1.23	1.13	1.13	1.13	1.13	1.13
RL	1.21	1.20	1.20	1.20	1.20	1.11	1.10	1.09	1.09	1.09
2R	1.21	1.20	1.19	1.19	1.19	1.11	1.10	1.09	1.08	1.08

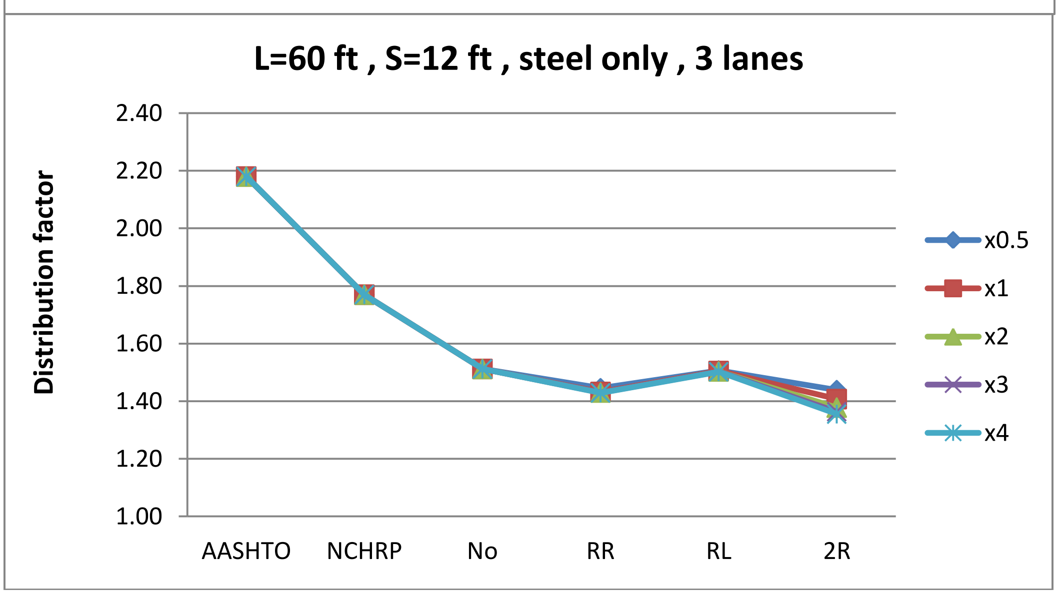




A.29. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=8ft , 3 lanes)

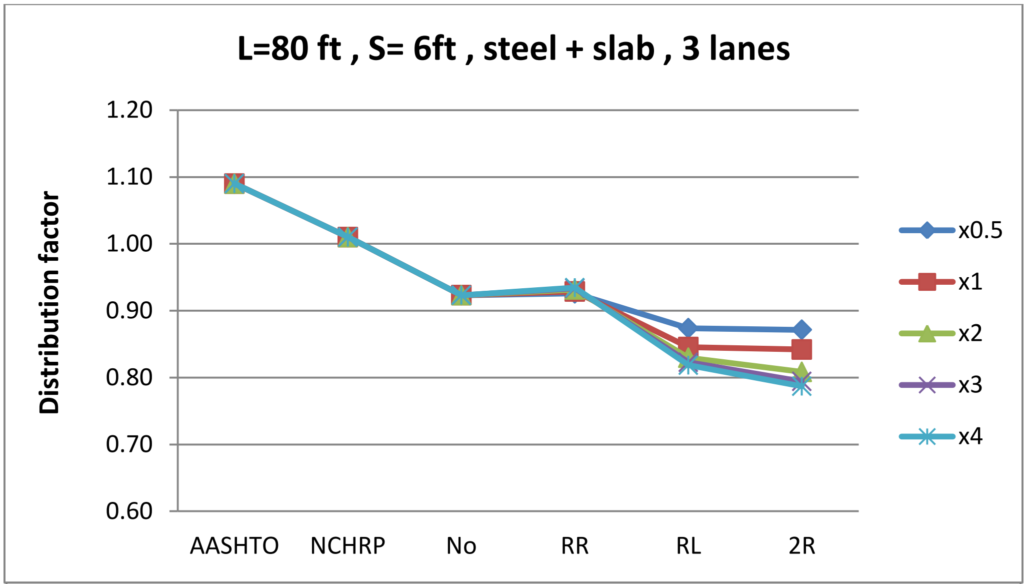
	DF=0.9*Mmax/M0									
	L=60 ft , S=12ft , steel + slab , 3 lanes					L=60 ft , S=12ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77
No	1.71	1.71	1.71	1.71	1.71	1.51	1.51	1.51	1.51	1.51
RR	1.64	1.63	1.63	1.63	1.63	1.44	1.43	1.43	1.43	1.43
RL	1.71	1.71	1.71	1.71	1.71	1.51	1.50	1.50	1.50	1.50
2R	1.64	1.61	1.58	1.57	1.56	1.44	1.41	1.38	1.36	1.36

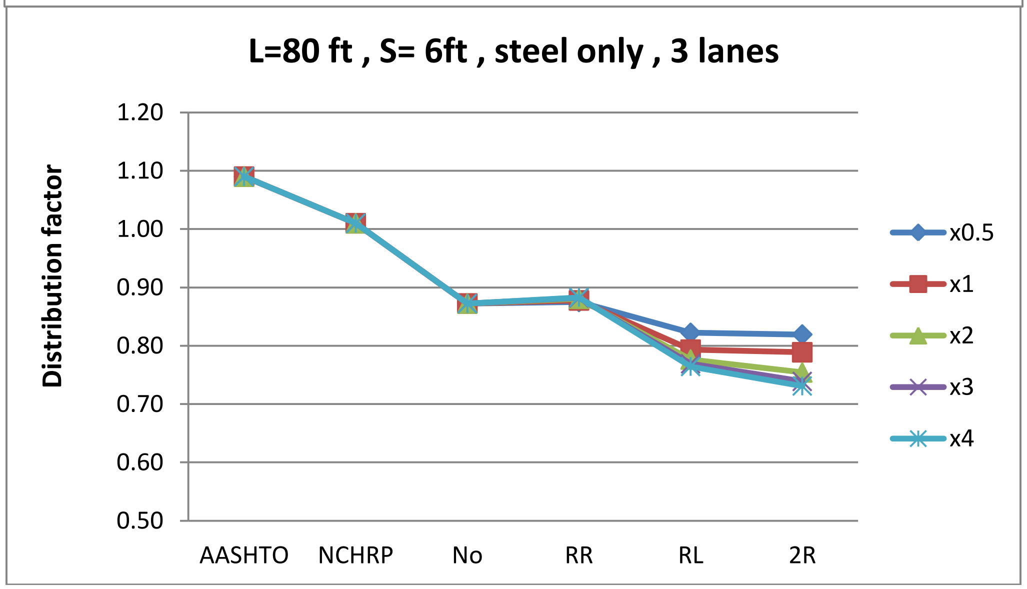




A.20. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=12ft , 3 lanes)

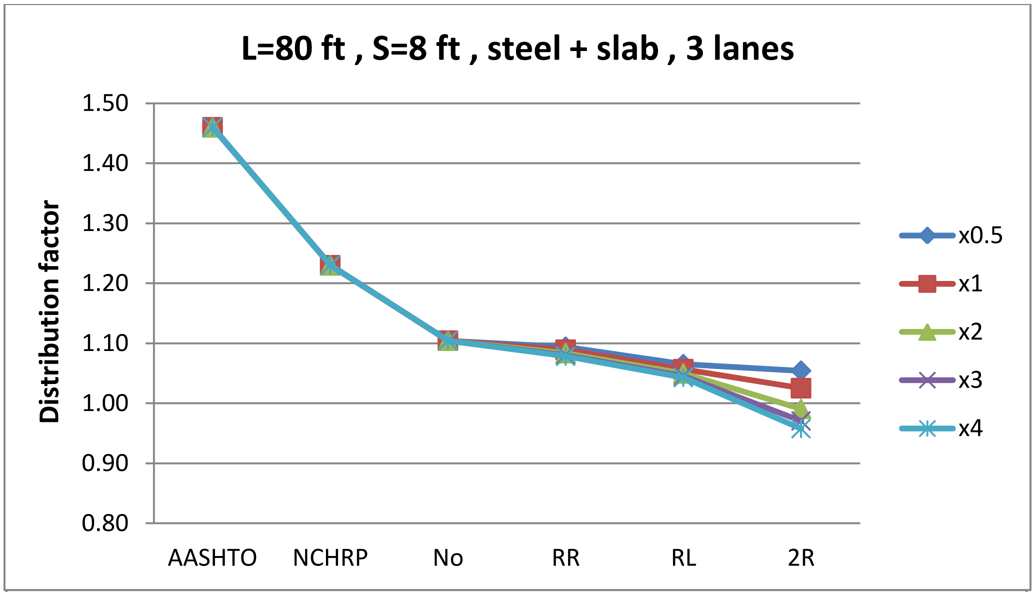
	DF=0.9*Mmax/M0									
	L=80 ft , S=6ft , steel + slab , 3 lanes					L=80 ft , S=6ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
No	0.92	0.92	0.92	0.92	0.92	0.87	0.87	0.87	0.87	0.87
RR	0.93	0.93	0.93	0.93	0.93	0.88	0.88	0.88	0.88	0.88
RL	0.87	0.85	0.83	0.82	0.82	0.82	0.79	0.78	0.77	0.76
2R	0.87	0.84	0.81	0.79	0.79	0.82	0.79	0.75	0.74	0.73

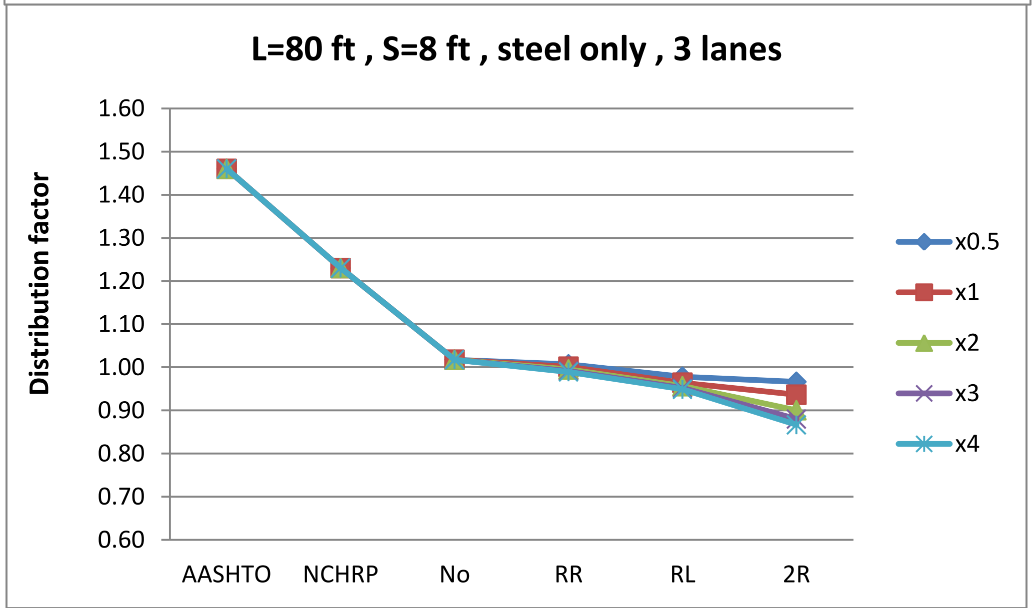




A.31. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=6ft , 3 lanes)

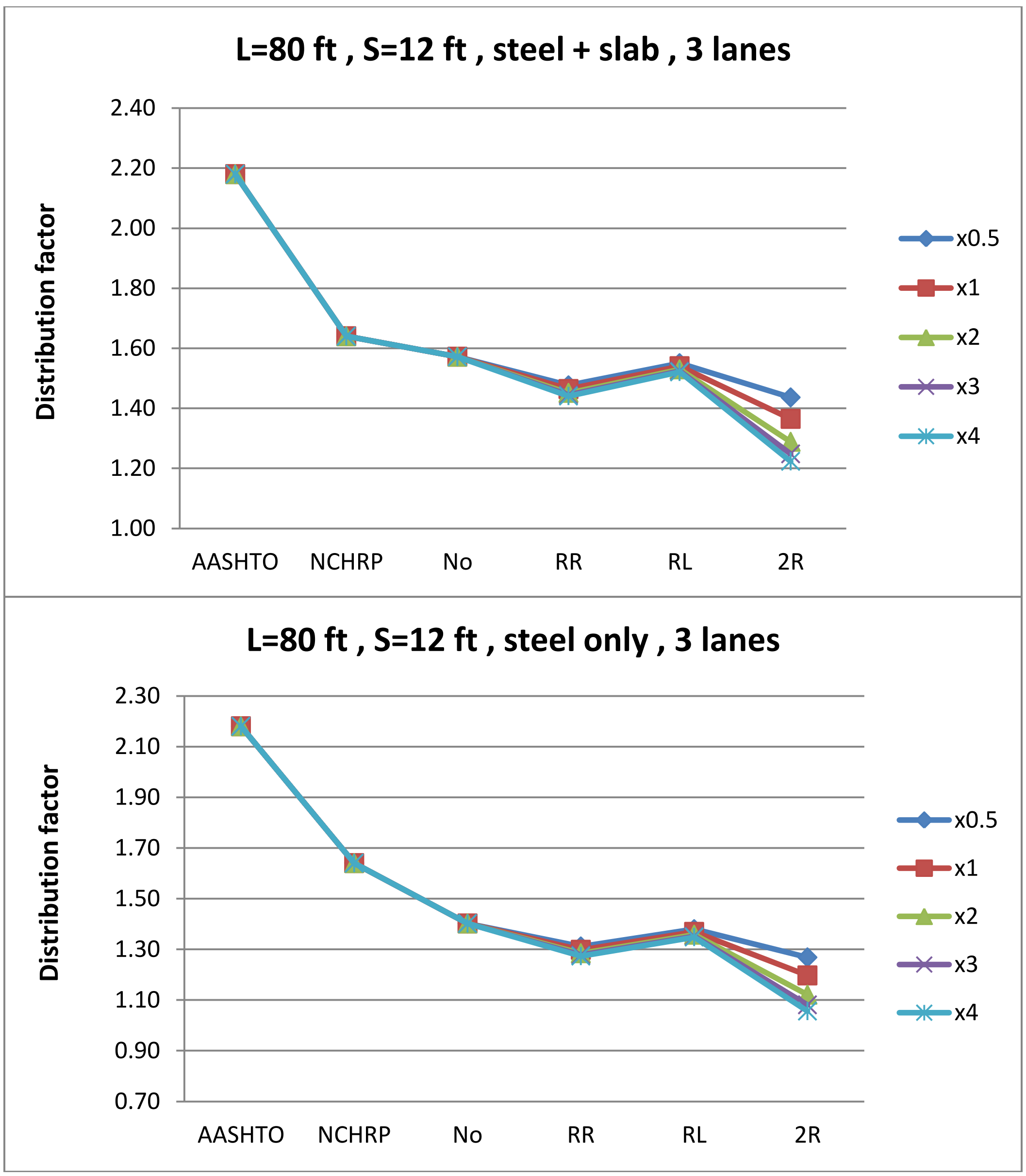
	DF=0.9*Mmax/M0									
	L=80 ft , S=8ft , steel + slab , 3 lanes					L=80 ft , S=8ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23
No	1.10	1.10	1.10	1.10	1.10	1.02	1.02	1.02	1.02	1.02
RR	1.09	1.09	1.08	1.08	1.08	1.01	1.00	1.00	0.99	0.99
RL	1.07	1.06	1.05	1.05	1.04	0.98	0.96	0.96	0.95	0.95
2R	1.05	1.03	0.99	0.97	0.96	0.97	0.94	0.90	0.88	0.87





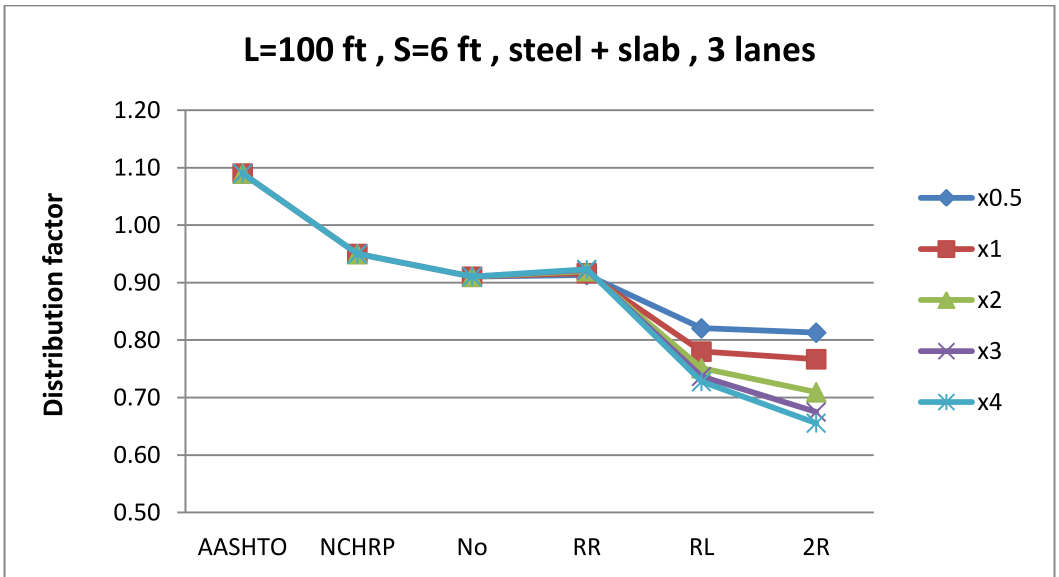
A.32. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=8ft , 3 lanes)

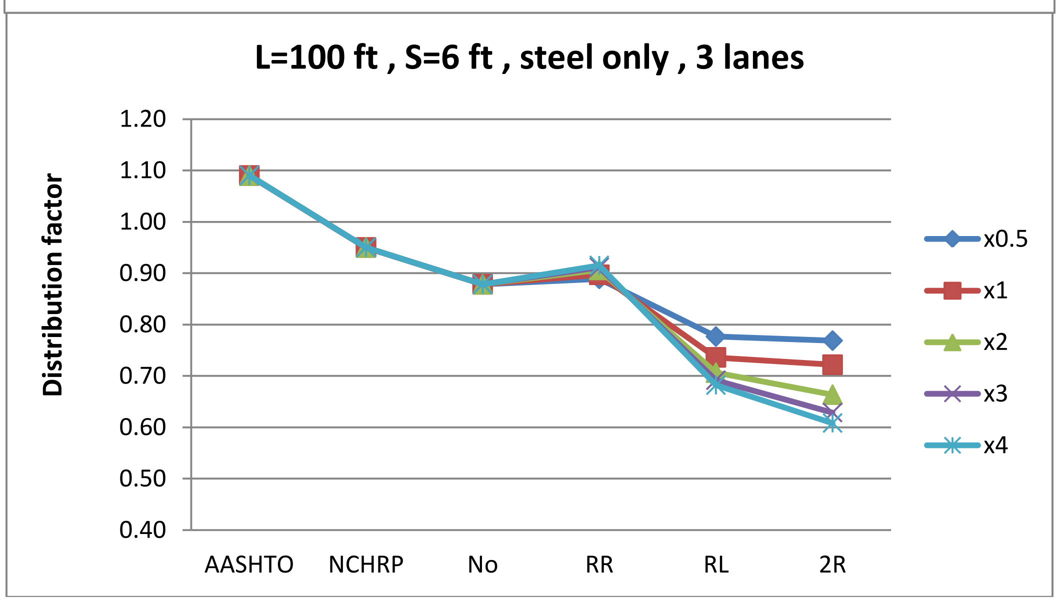
	DF=0.9*Mmax/M0									
	L=80 ft , S=12ft , steel + slab , 3 lanes					L=80 ft , S=12ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
No	1.57	1.57	1.57	1.57	1.57	1.40	1.40	1.40	1.40	1.40
RR	1.48	1.46	1.45	1.44	1.44	1.31	1.30	1.28	1.28	1.27
RL	1.55	1.54	1.53	1.52	1.52	1.38	1.37	1.36	1.35	1.35
2R	1.44	1.36	1.29	1.25	1.22	1.27	1.20	1.12	1.08	1.06



A.33. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=12ft , 3 lanes)

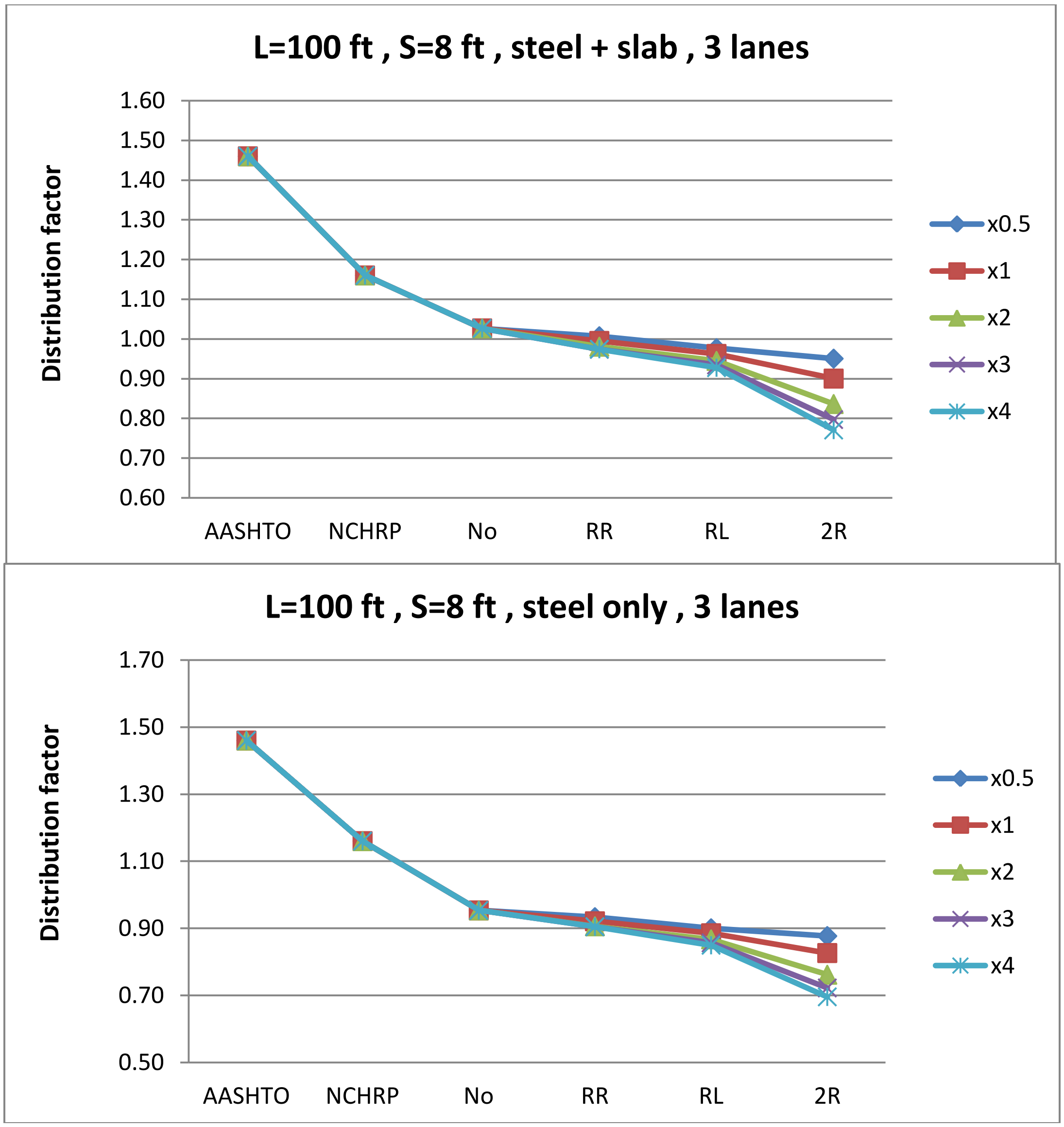
	DF=0.9*Mmax/M0									
	L=100 ft , S=6ft , steel + slab , 3 lanes					L=100 ft , S=6ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
No	0.91	0.91	0.91	0.91	0.91	0.88	0.88	0.88	0.88	0.88
RR	0.91	0.92	0.92	0.92	0.92	0.89	0.90	0.91	0.91	0.92
RL	0.82	0.78	0.75	0.74	0.73	0.78	0.74	0.71	0.69	0.68
2R	0.81	0.77	0.71	0.68	0.66	0.77	0.72	0.66	0.63	0.61





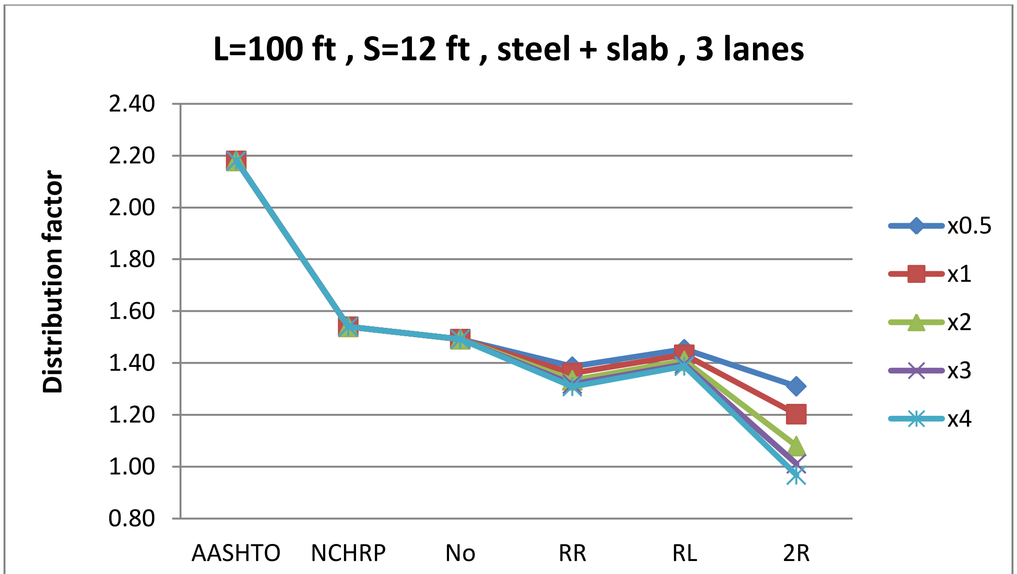
A.34. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=6ft , 3 lanes)

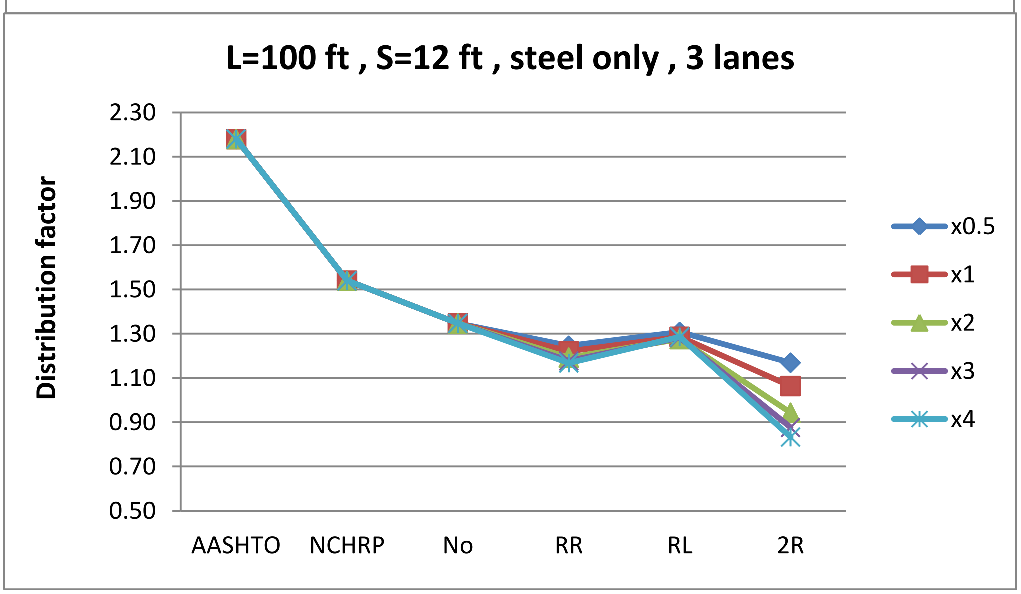
	DF=0.9*Mmax/M0									
	L=100 ft , S=8ft , steel + slab , 3 lanes					L=100 ft , S=8ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16
No	1.03	1.03	1.03	1.03	1.03	0.95	0.95	0.95	0.95	0.95
RR	1.01	1.00	0.98	0.97	0.97	0.93	0.92	0.91	0.90	0.90
RL	0.98	0.96	0.94	0.93	0.93	0.90	0.89	0.87	0.86	0.85
2R	0.95	0.90	0.84	0.80	0.77	0.88	0.83	0.76	0.72	0.70



A.35. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=8ft , 3 lanes)

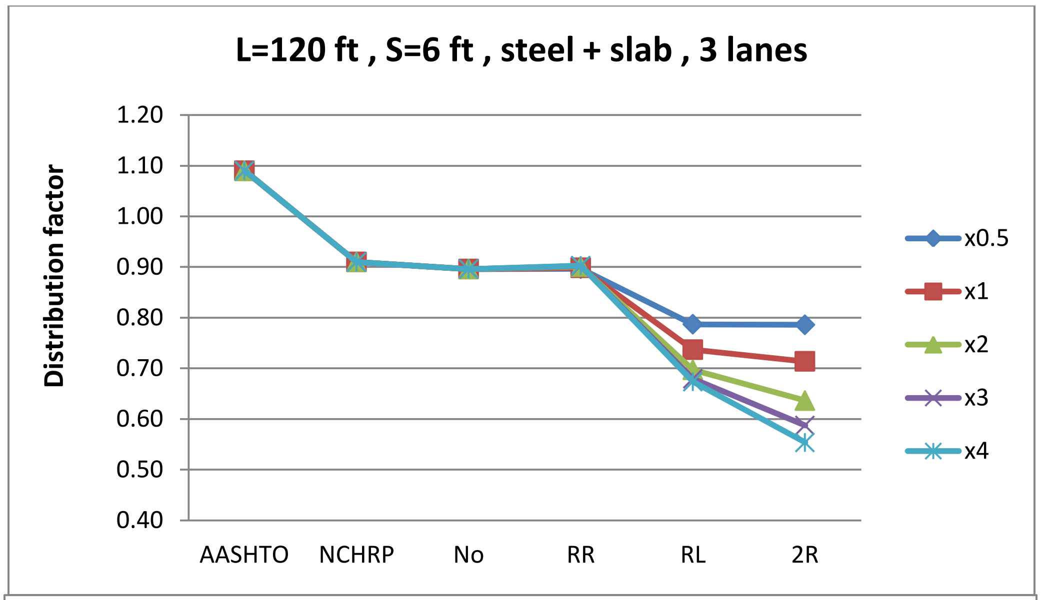
	DF=0.9*Mmax/M0									
	L=100 ft , S=12ft , steel + slab , 3 lanes					L=100 ft , S=12ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54
No	1.49	1.49	1.49	1.49	1.49	1.35	1.35	1.35	1.35	1.35
RR	1.39	1.36	1.33	1.32	1.31	1.25	1.22	1.19	1.18	1.17
RL	1.45	1.43	1.41	1.40	1.39	1.31	1.29	1.28	1.28	1.29
2R	1.31	1.20	1.08	1.01	0.97	1.17	1.06	0.94	0.88	0.83

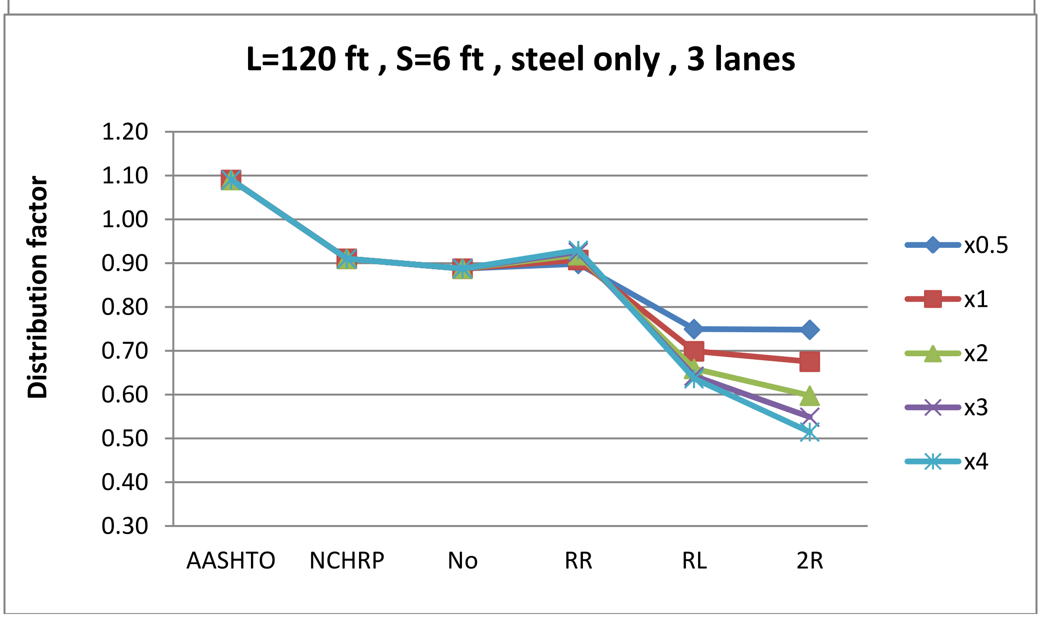




A.36. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=12ft , 3 lanes)

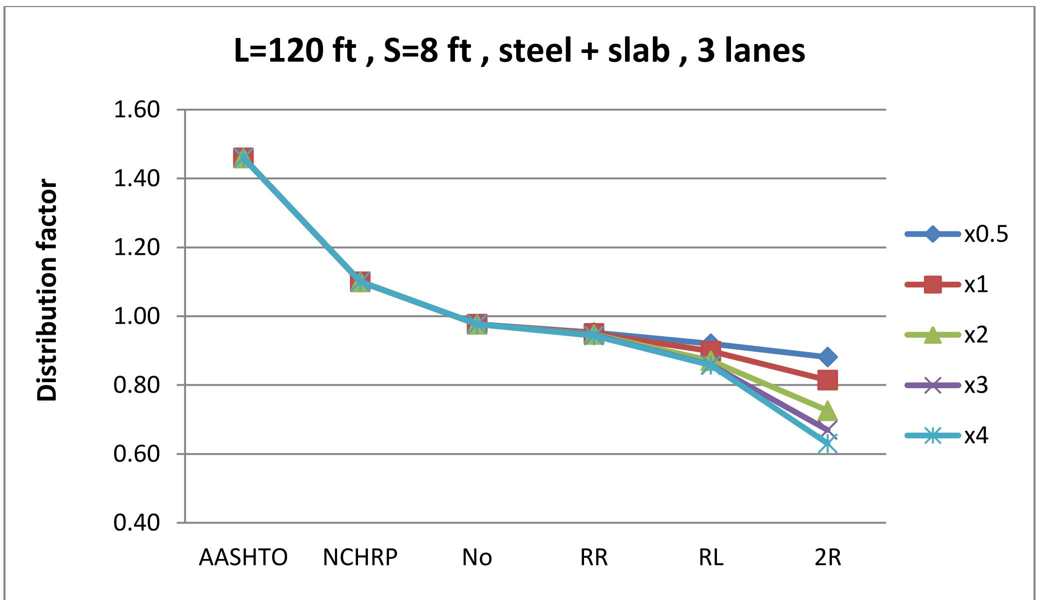
	DF=0.9*Mmax/M0									
	L=120 ft , S=6ft , steel + slab , 3 lanes					L=120 ft , S=6ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
No	0.90	0.90	0.90	0.90	0.90	0.89	0.89	0.89	0.89	0.89
RR	0.90	0.90	0.90	0.90	0.90	0.90	0.91	0.92	0.93	0.93
RL	0.79	0.74	0.70	0.68	0.67	0.75	0.70	0.66	0.64	0.64
2R	0.79	0.71	0.64	0.59	0.55	0.75	0.68	0.60	0.55	0.51

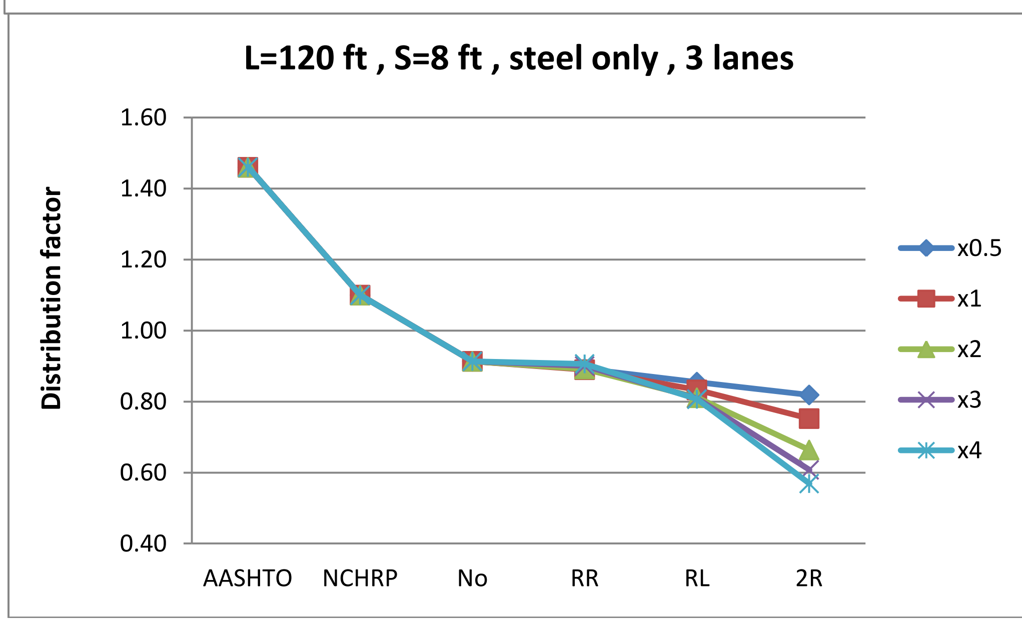




A.37. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=6ft , 3 lanes)

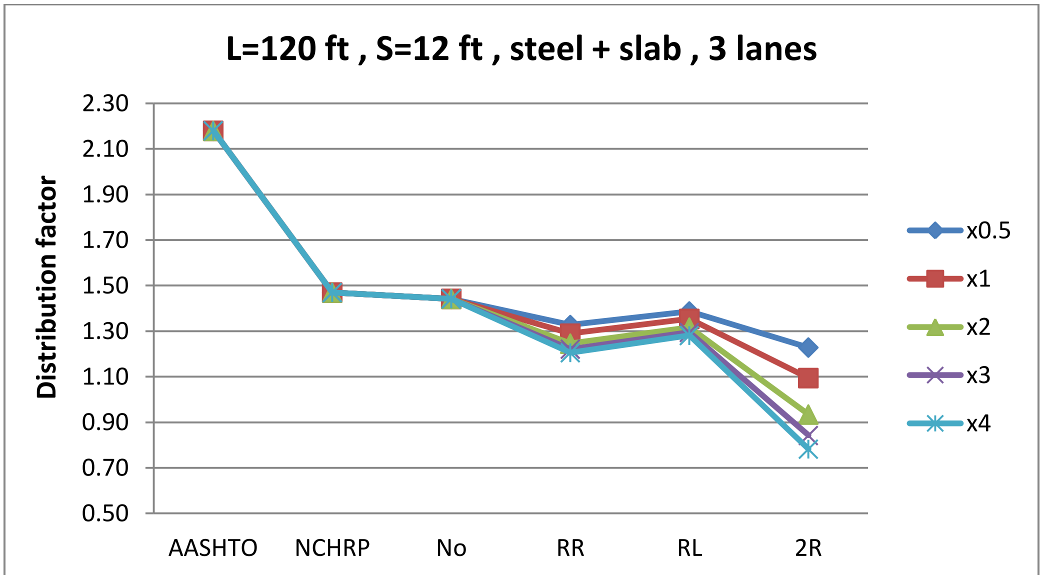
	DF=0.9*Mmax/M0									
	L=120 ft , S=8ft , steel + slab , 3 lanes					L=120 ft , S=8ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
No	0.98	0.98	0.98	0.98	0.98	0.91	0.91	0.91	0.91	0.91
RR	0.95	0.95	0.95	0.94	0.94	0.89	0.89	0.89	0.90	0.91
RL	0.92	0.90	0.87	0.86	0.86	0.85	0.83	0.81	0.81	0.81
2R	0.88	0.81	0.73	0.67	0.63	0.82	0.75	0.66	0.61	0.57

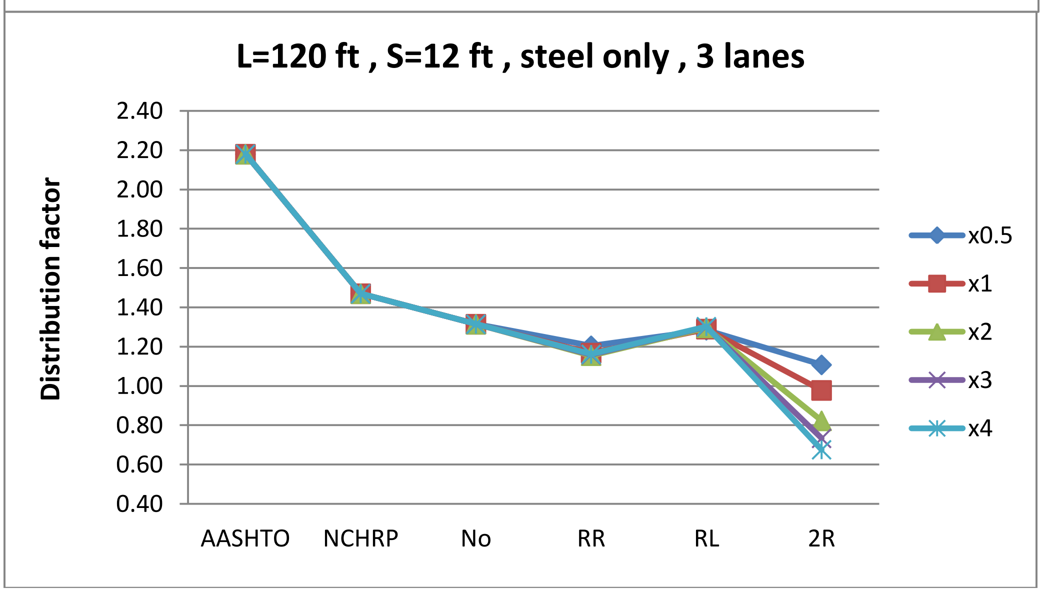




A.38. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=8ft , 3 lanes)

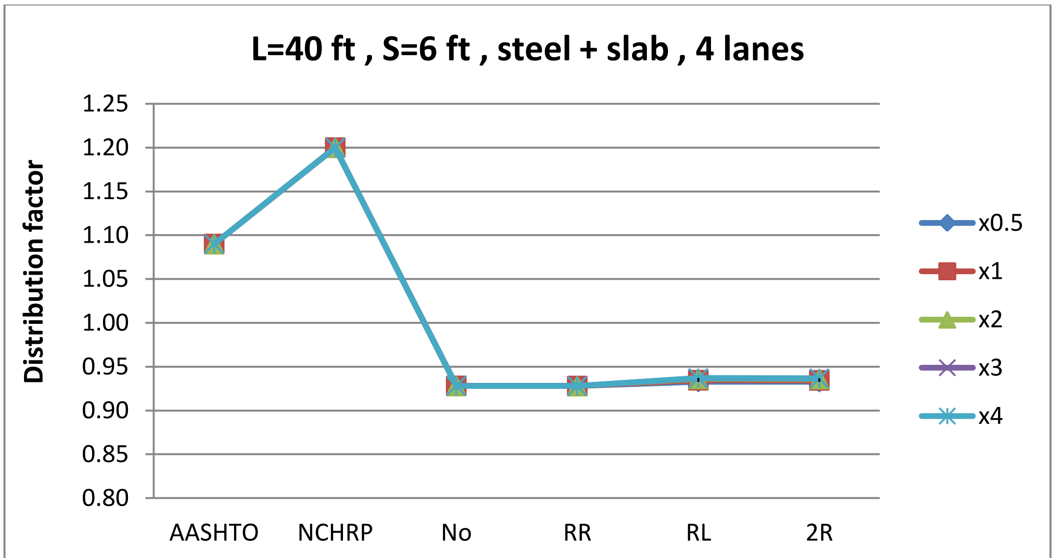
	DF=0.9*Mmax/M0									
	L=120 ft , S=12ft , steel + slab , 3 lanes					L=120 ft , S=12ft , steel only , 3 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47
No	1.44	1.44	1.44	1.44	1.44	1.31	1.31	1.31	1.31	1.31
RR	1.33	1.29	1.25	1.22	1.21	1.20	1.17	1.15	1.16	1.16
RL	1.39	1.35	1.32	1.29	1.28	1.28	1.29	1.30	1.30	1.30
2R	1.23	1.09	0.93	0.84	0.78	1.11	0.98	0.82	0.73	0.68

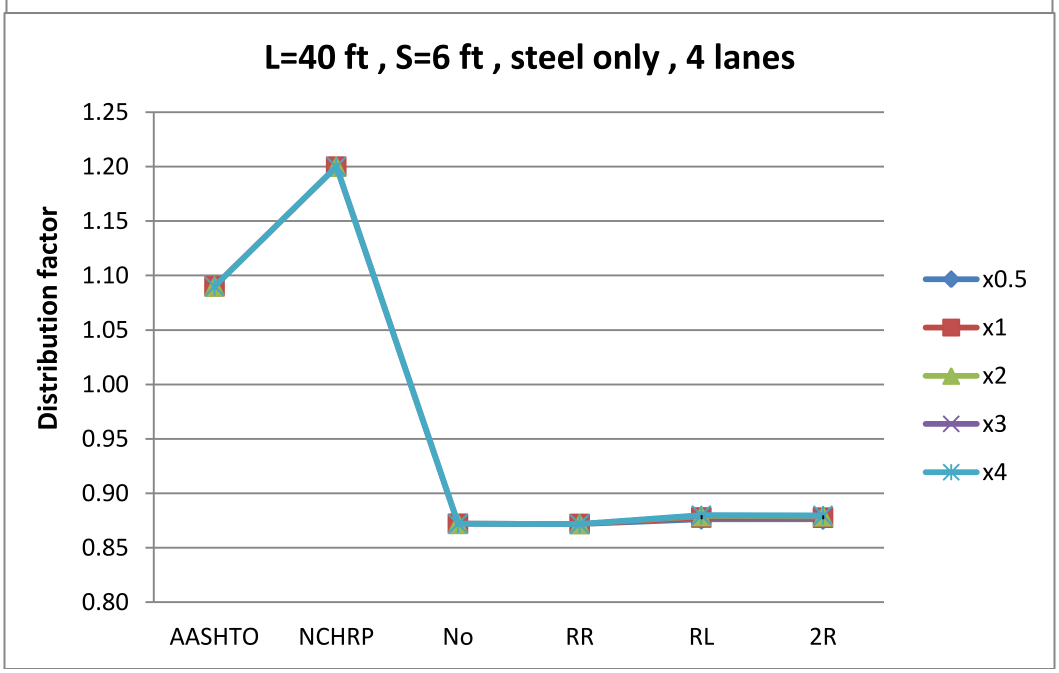




A.39. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=12ft , 3 lanes)

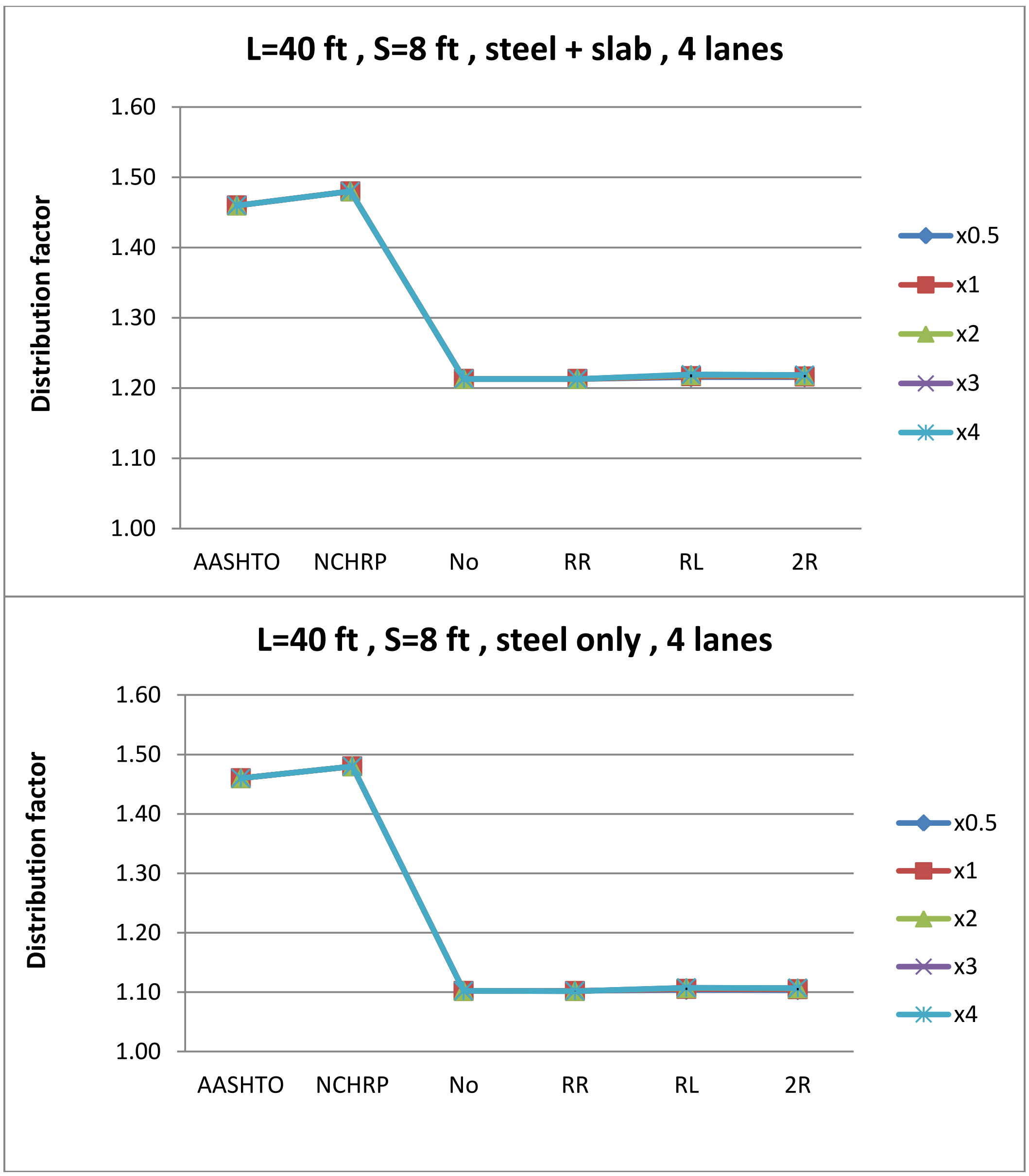
	DF=0.75*Mmax/M0									
	L=40 ft , S=6ft , steel + slab , 4 lanes					L=40 ft , S=6ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
No	0.93	0.93	0.93	0.93	0.93	0.87	0.87	0.87	0.87	0.87
RR	0.93	0.93	0.93	0.93	0.93	0.87	0.87	0.87	0.87	0.87
RL	0.93	0.93	0.94	0.94	0.94	0.88	0.88	0.88	0.88	0.88
2R	0.93	0.93	0.94	0.94	0.94	0.88	0.88	0.88	0.88	0.88





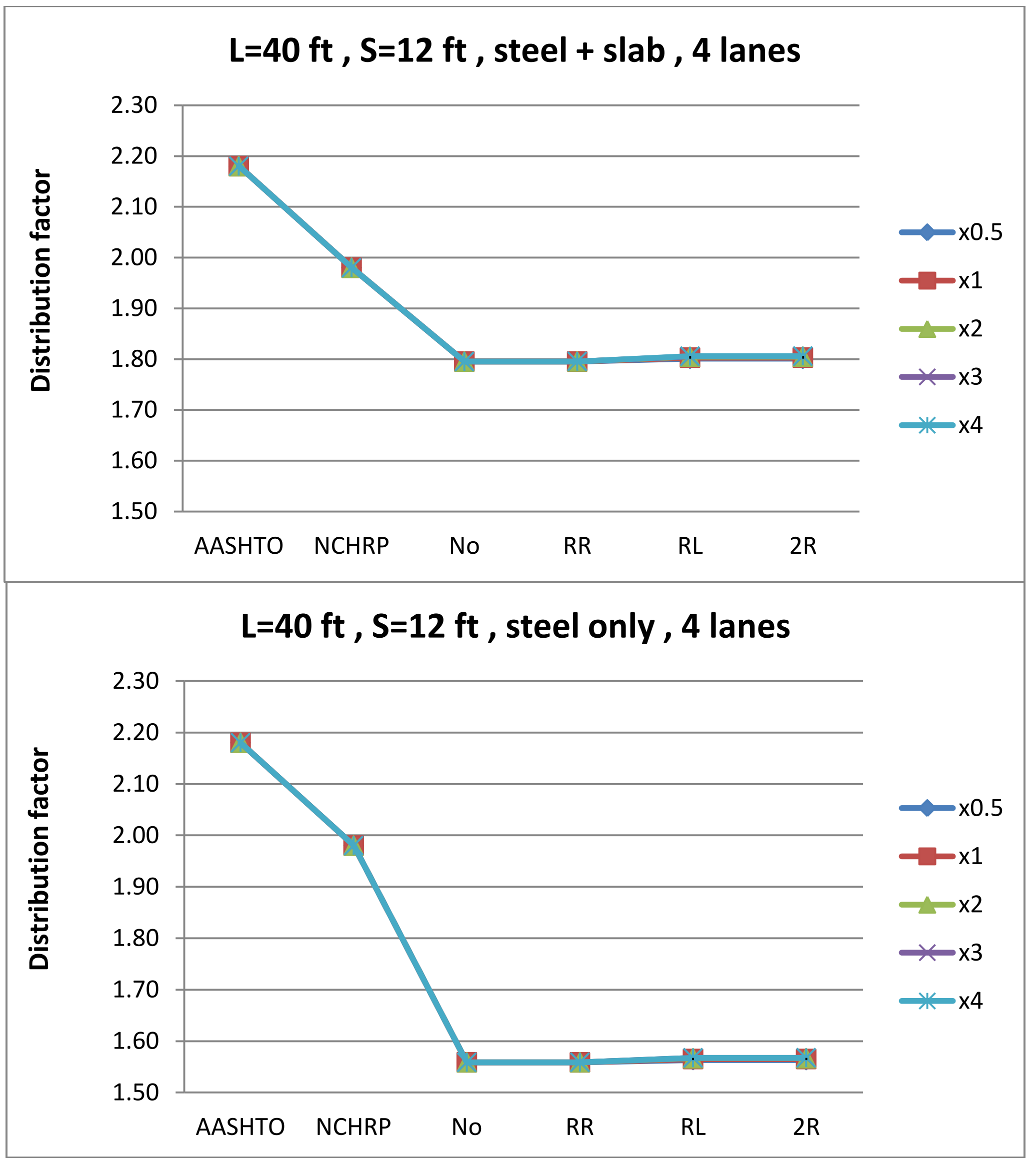
A.21. Sensitivity of Distribution Factor to railing stiffness (L=40ft , s=6ft , 4 lanes)

	DF=0.75*Mmax/M0									
	L=40 ft , S=8ft , steel + slab , 4 lanes					L=40 ft , S=8ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48	1.48
No	1.21	1.21	1.21	1.21	1.21	1.10	1.10	1.10	1.10	1.10
RR	1.21	1.21	1.21	1.21	1.21	1.10	1.10	1.10	1.10	1.10
RL	1.22	1.22	1.22	1.22	1.22	1.10	1.11	1.11	1.11	1.11
2R	1.22	1.22	1.22	1.22	1.22	1.10	1.10	1.11	1.11	1.11



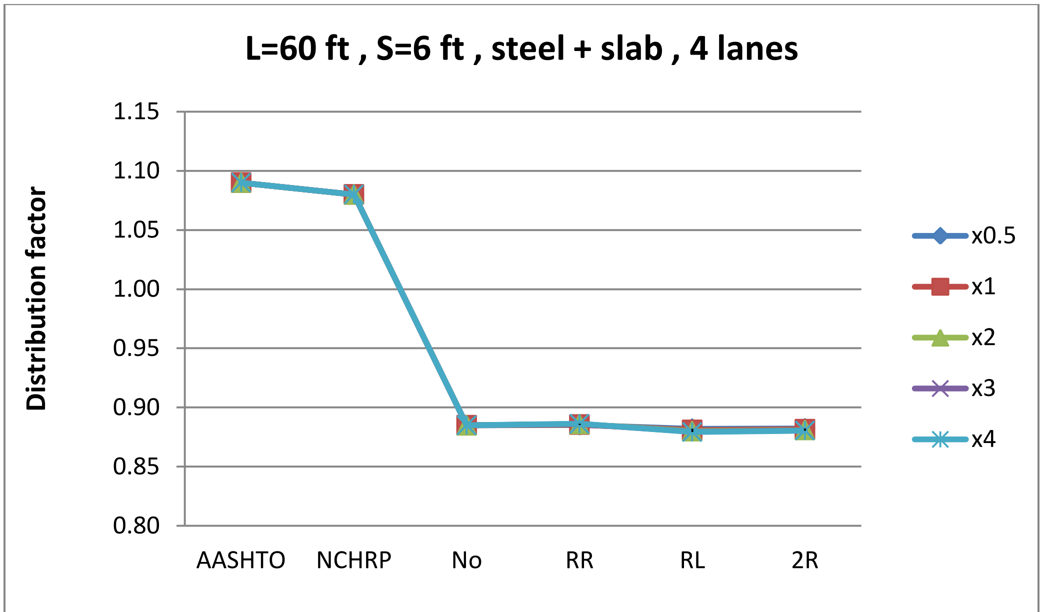
A.41. Sensitivity of Distribution Factor to railing stiffness (L=40ft , s=8ft , 4 lanes)

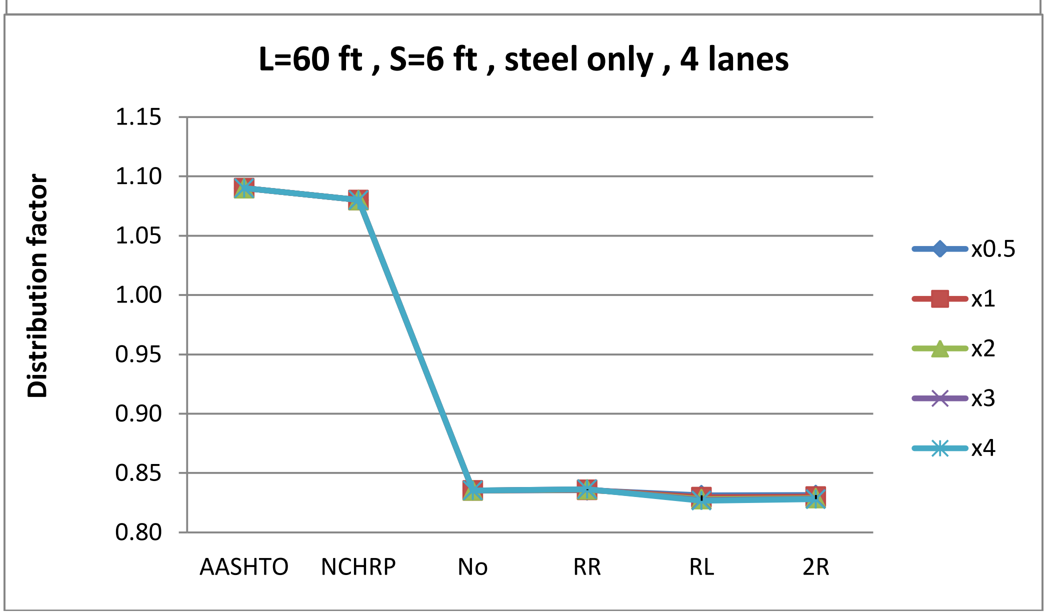
	DF=0.75*Mmax/M0									
	L=40 ft , S=12ft , steel + slab , 4 lanes					L=40 ft , S=12ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98
No	1.80	1.80	1.80	1.80	1.80	1.56	1.56	1.56	1.56	1.56
RR	1.80	1.80	1.80	1.80	1.80	1.56	1.56	1.56	1.56	1.56
RL	1.80	1.80	1.80	1.81	1.81	1.56	1.57	1.57	1.57	1.57
2R	1.80	1.80	1.80	1.81	1.81	1.56	1.57	1.57	1.57	1.57



A.42. Sensitivity of Distribution Factor to railing stiffness (L=40ft , s=12ft , 4 lanes)

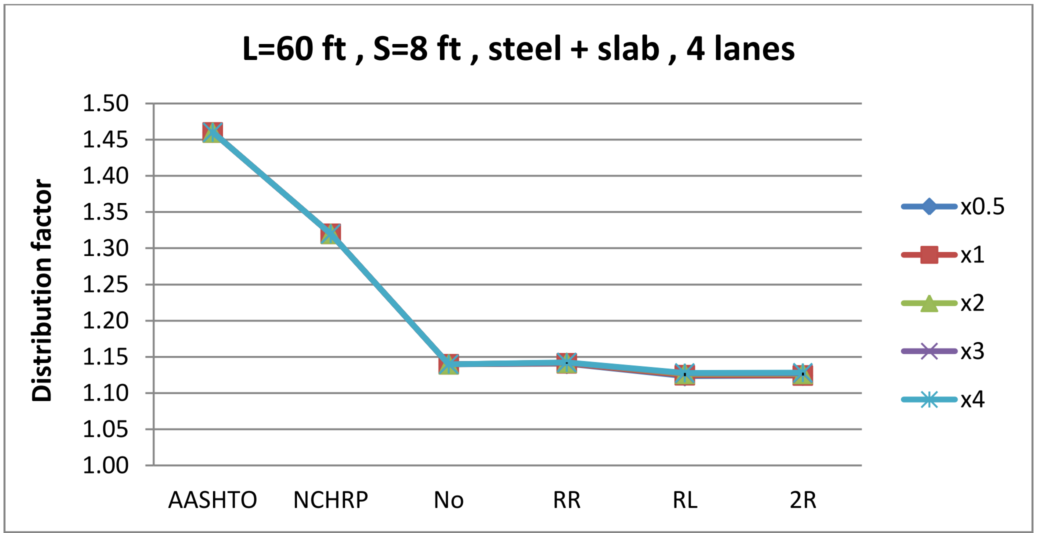
	DF=0.75*Mmax/M0									
	L=60 ft , S=6ft , steel + slab , 4 lanes					L=60 ft , S=6ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08	1.08
No	0.88	0.88	0.88	0.88	0.88	0.84	0.84	0.84	0.84	0.84
RR	0.89	0.89	0.89	0.89	0.89	0.84	0.84	0.84	0.84	0.84
RL	0.88	0.88	0.88	0.88	0.88	0.83	0.83	0.83	0.83	0.83
2R	0.88	0.88	0.88	0.88	0.88	0.83	0.83	0.83	0.83	0.83

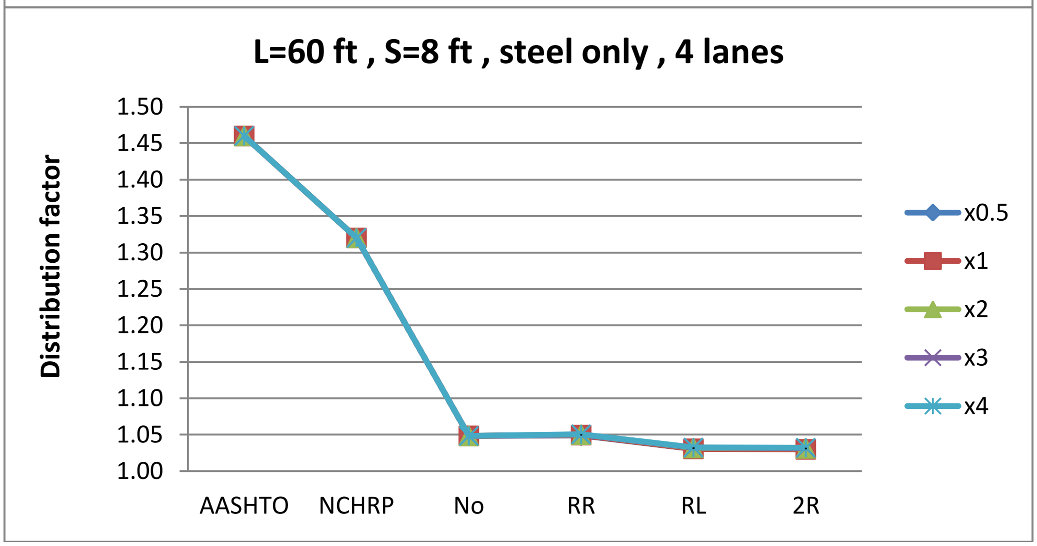




A.43. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=6ft , 4 lanes)

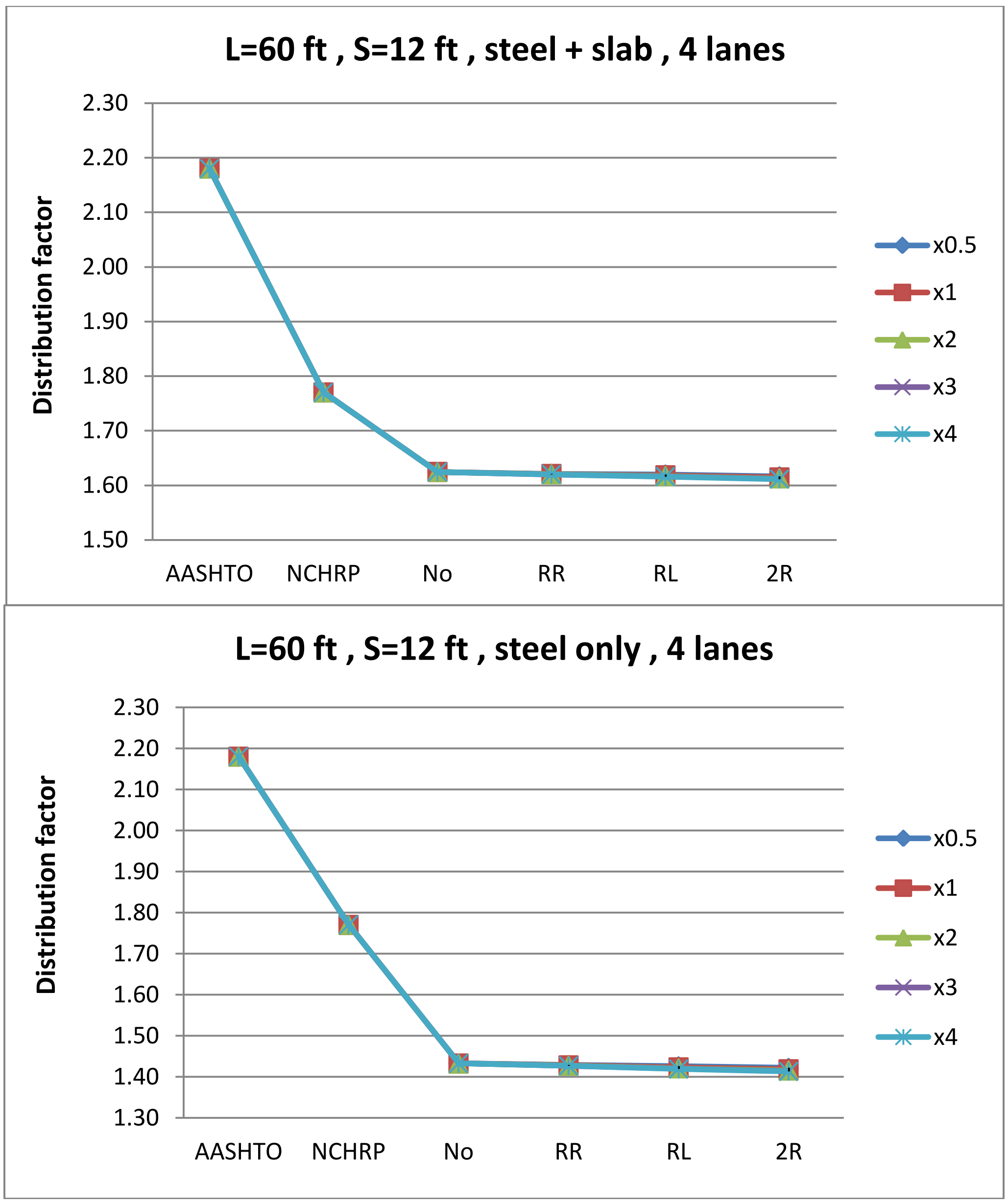
	DF=0.75*Mmax/M0									
	L=60 ft , S=8ft , steel + slab , 4 lanes					L=60 ft , S=8ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32
No	1.14	1.14	1.14	1.14	1.14	1.05	1.05	1.05	1.05	1.05
RR	1.14	1.14	1.14	1.14	1.14	1.05	1.05	1.05	1.05	1.05
RL	1.12	1.12	1.13	1.13	1.13	1.03	1.03	1.03	1.03	1.03
2R	1.12	1.12	1.13	1.13	1.13	1.03	1.03	1.03	1.03	1.03





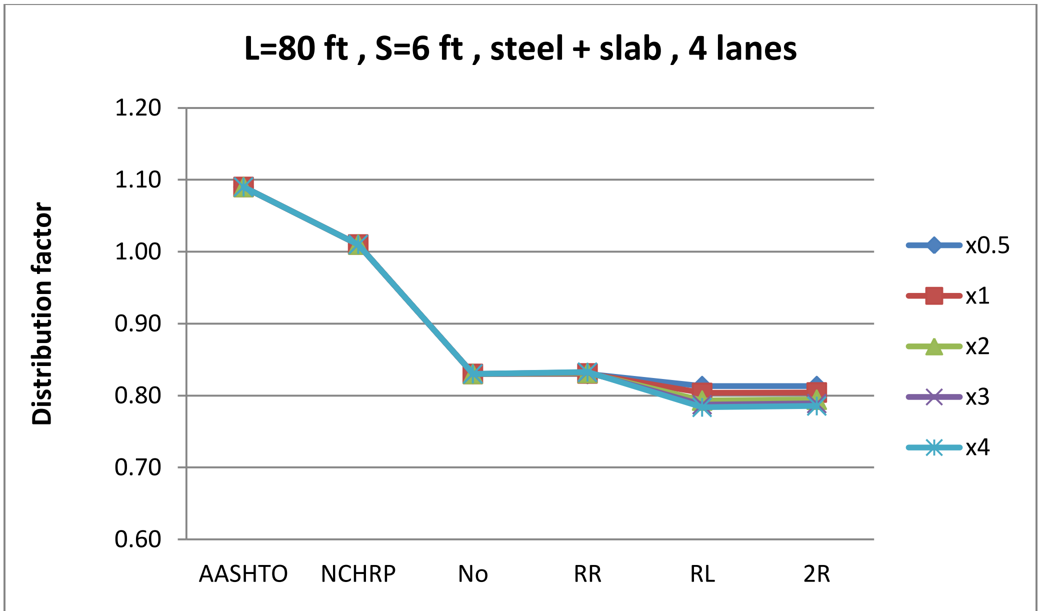
A.44. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=8ft , 4 lanes)

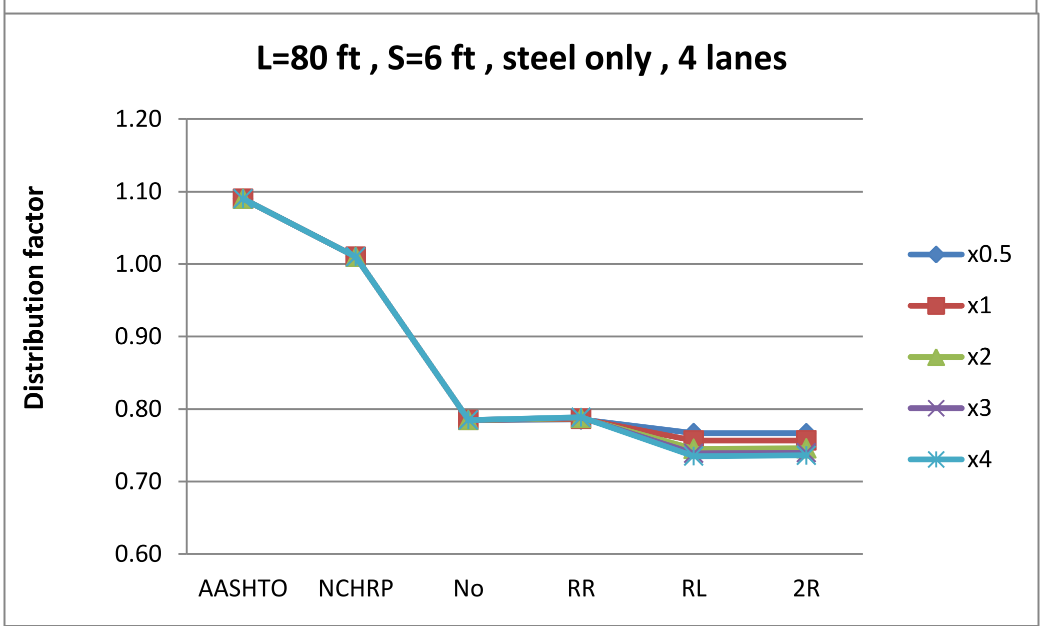
	DF=0.75*Mmax/M0									
	L=60 ft , S=12ft , steel + slab , 4 lanes					L=60 ft , S=12ft , steel only, 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77	1.77
No	1.62	1.62	1.62	1.62	1.62	1.43	1.43	1.43	1.43	1.43
RR	1.62	1.62	1.62	1.62	1.62	1.43	1.43	1.43	1.43	1.43
RL	1.62	1.62	1.62	1.62	1.62	1.43	1.42	1.42	1.42	1.42
2R	1.62	1.61	1.61	1.61	1.61	1.42	1.42	1.42	1.41	1.41



A.45. Sensitivity of Distribution Factor to railing stiffness (L=60ft , s=12ft , 4 lanes)

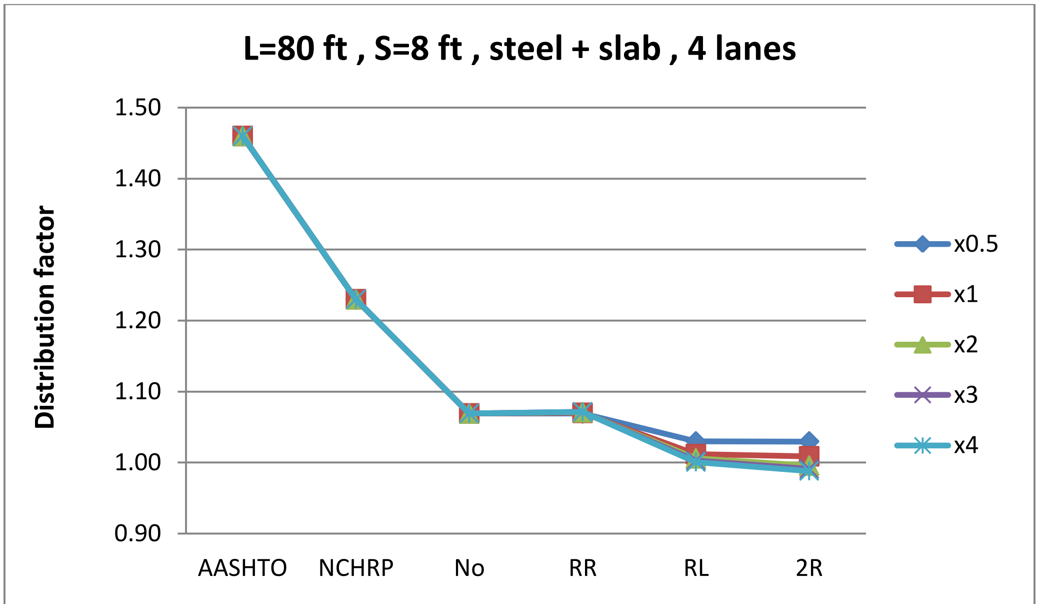
	DF=0.75*Mmax/M0									
	L=80 ft , S=6ft , steel + slab , 4 lanes					L=80 ft , S=6ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
No	0.83	0.83	0.83	0.83	0.83	0.78	0.78	0.78	0.78	0.78
RR	0.83	0.83	0.83	0.83	0.83	0.79	0.79	0.79	0.79	0.79
RL	0.81	0.80	0.79	0.79	0.78	0.77	0.76	0.75	0.74	0.73
2R	0.81	0.80	0.79	0.79	0.79	0.77	0.76	0.75	0.74	0.74

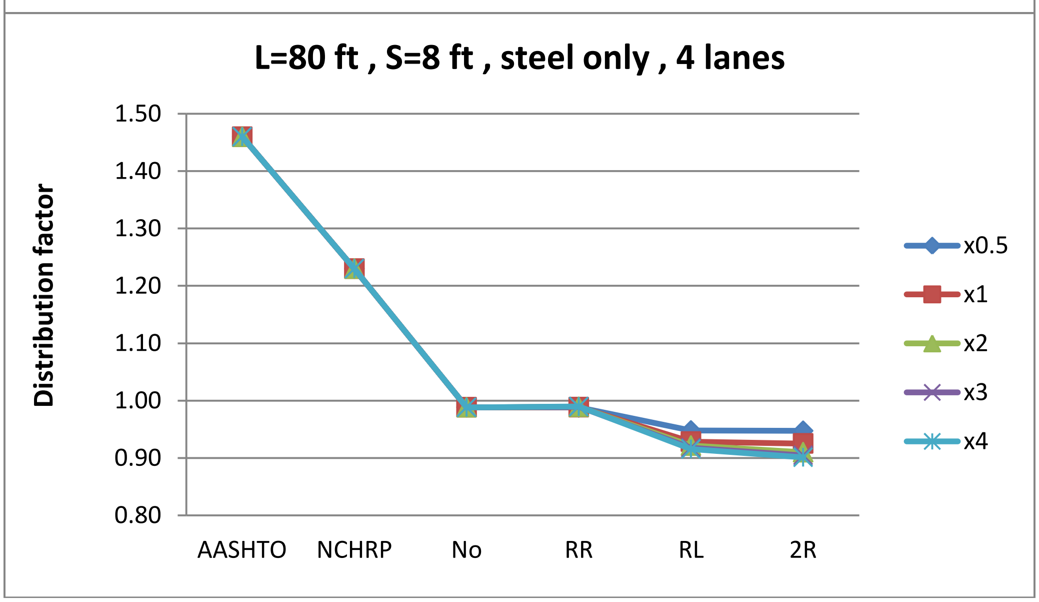




A.46. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=6ft , 4 lanes)

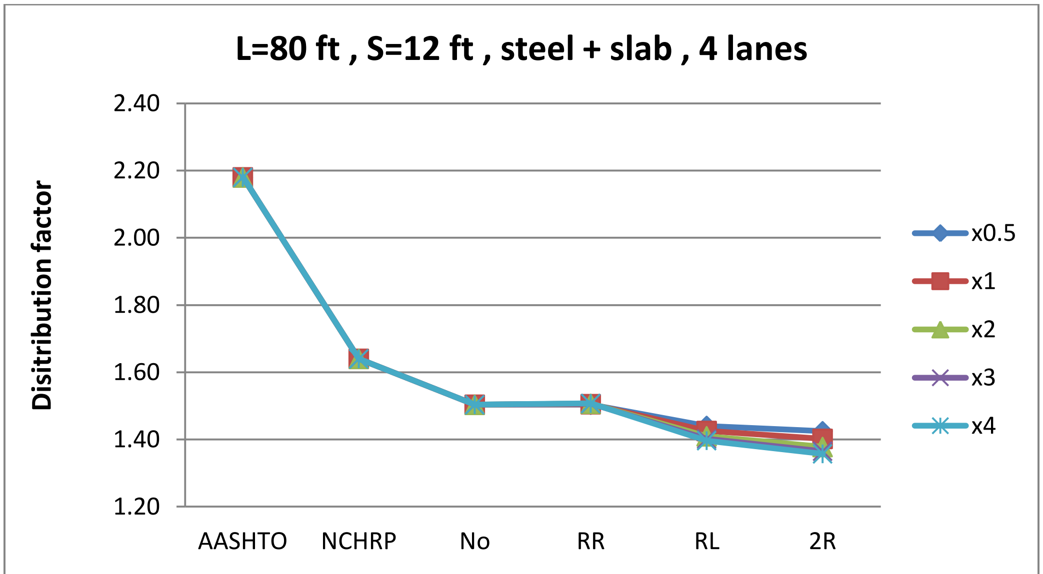
	DF=0.75*Mmax/M0									
	L=80 ft , S=8ft , steel + slab , 4 lanes					L=80 ft , S=8ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23	1.23
No	1.07	1.07	1.07	1.07	1.07	0.99	0.99	0.99	0.99	0.99
RR	1.07	1.07	1.07	1.07	1.07	0.99	0.99	0.99	0.99	0.99
RL	1.03	1.01	1.01	1.00	1.00	0.95	0.93	0.92	0.92	0.92
2R	1.03	1.01	1.00	0.99	0.99	0.95	0.93	0.91	0.90	0.90

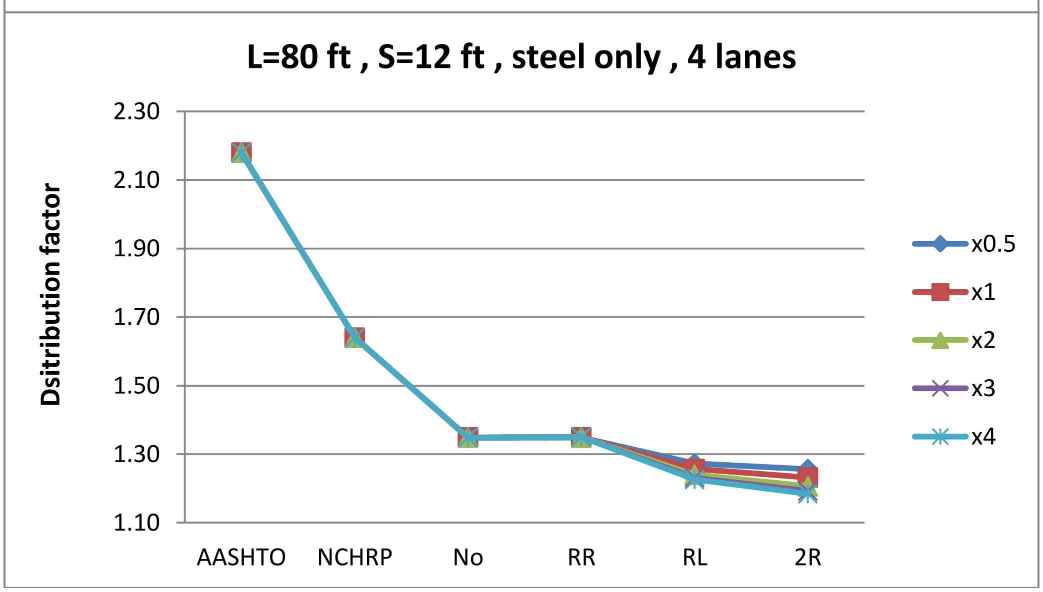




A.47. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=8ft , 4 lanes)

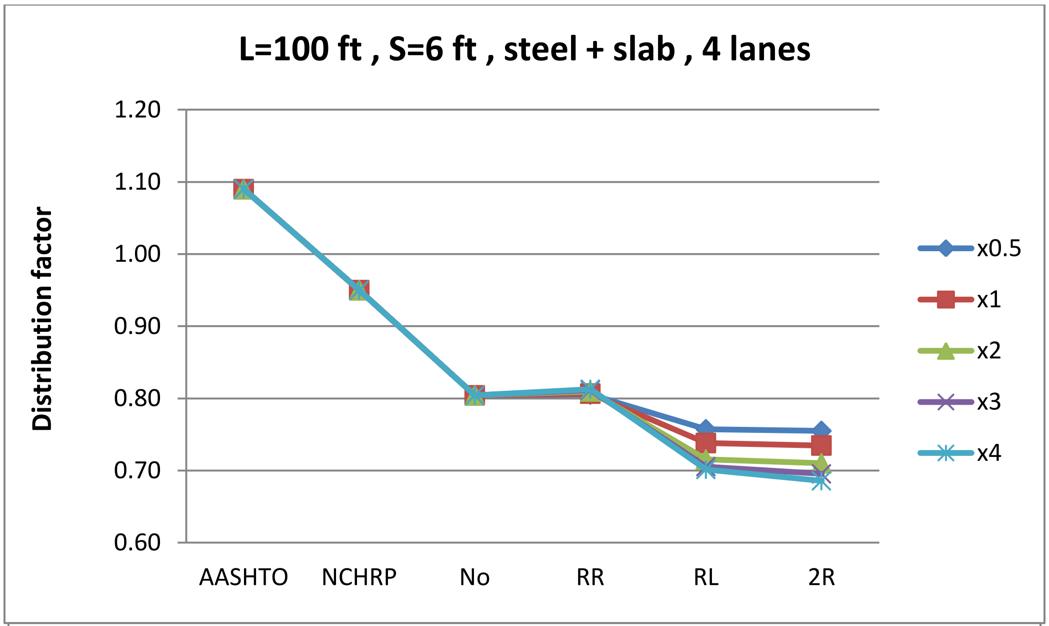
	DF=0.75*Mmax/M0									
	L=80 ft , S=12ft , steel + slab , 4 lanes					L=80 ft , S=12ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
No	1.50	1.50	1.50	1.50	1.50	1.35	1.35	1.35	1.35	1.35
RR	1.50	1.51	1.51	1.51	1.51	1.35	1.35	1.35	1.35	1.35
RL	1.44	1.43	1.41	1.40	1.40	1.27	1.26	1.24	1.23	1.23
2R	1.42	1.40	1.38	1.37	1.36	1.26	1.23	1.21	1.19	1.18

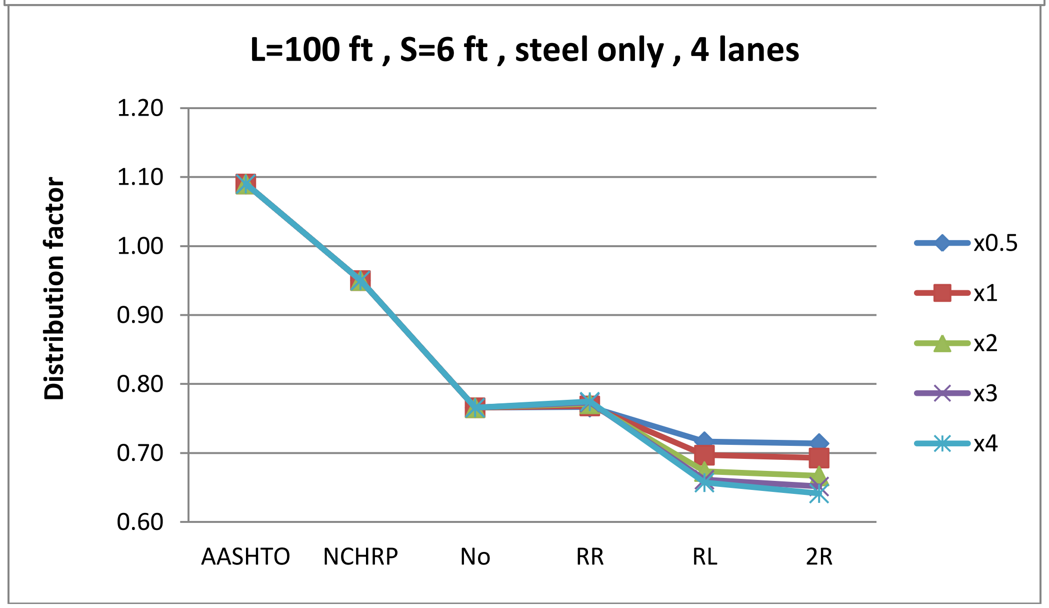




A.48. Sensitivity of Distribution Factor to railing stiffness (L=80ft , s=12ft , 4 lanes)

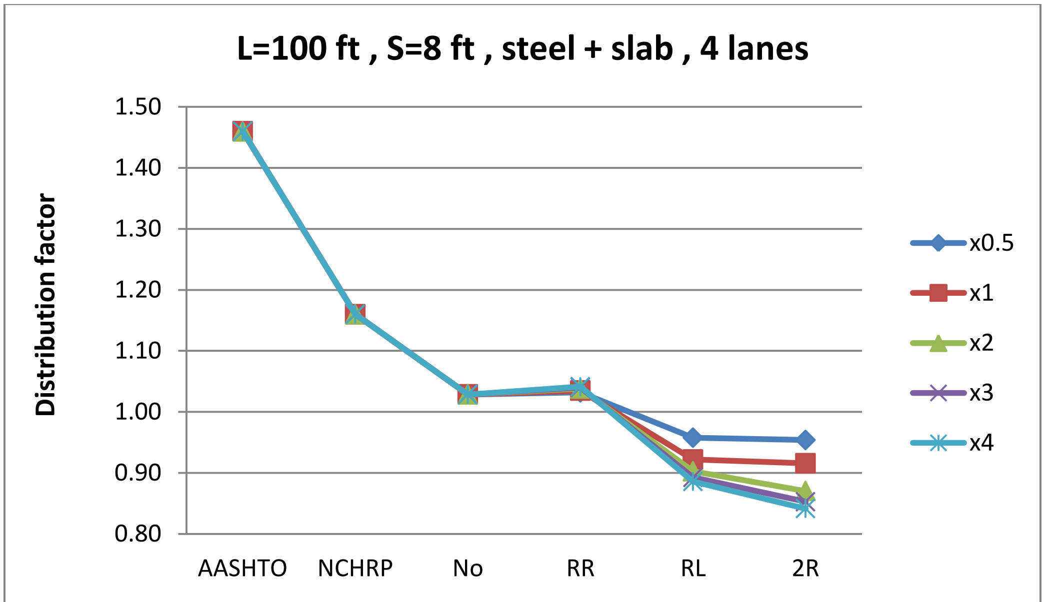
	DF=0.75*Mmax/M0									
	L=100 ft , S=6ft , steel + slab , 4 lanes					L=100 ft , S=6ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
No	0.80	0.80	0.80	0.80	0.80	0.77	0.77	0.77	0.77	0.77
RR	0.81	0.81	0.81	0.81	0.81	0.77	0.77	0.77	0.77	0.77
RL	0.76	0.74	0.72	0.71	0.70	0.72	0.70	0.67	0.66	0.66
2R	0.75	0.73	0.71	0.70	0.69	0.71	0.69	0.67	0.65	0.64

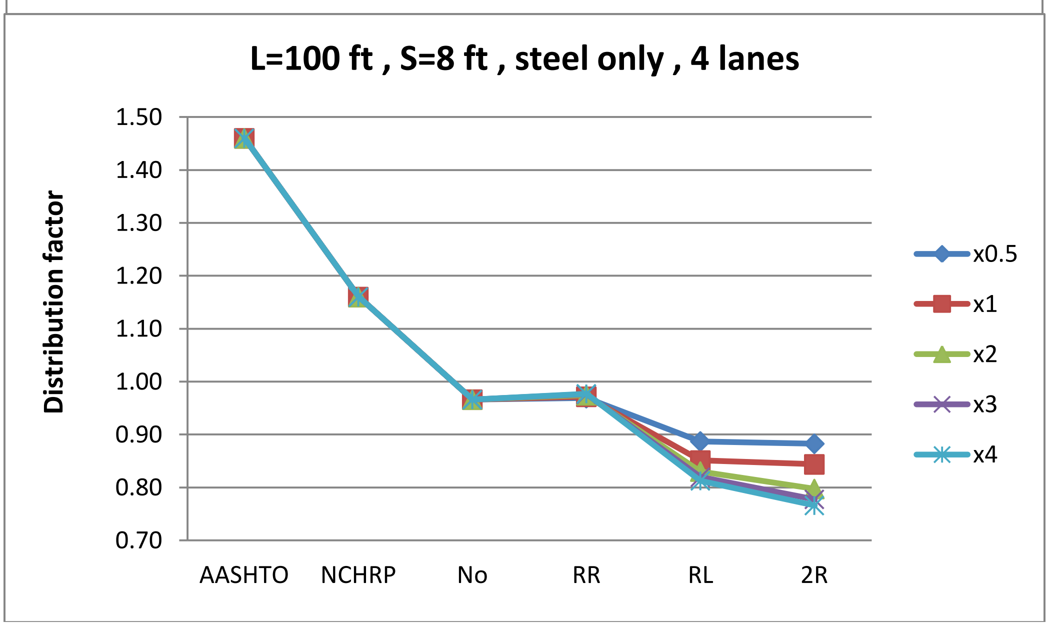




A.49. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=6ft , 4 lanes)

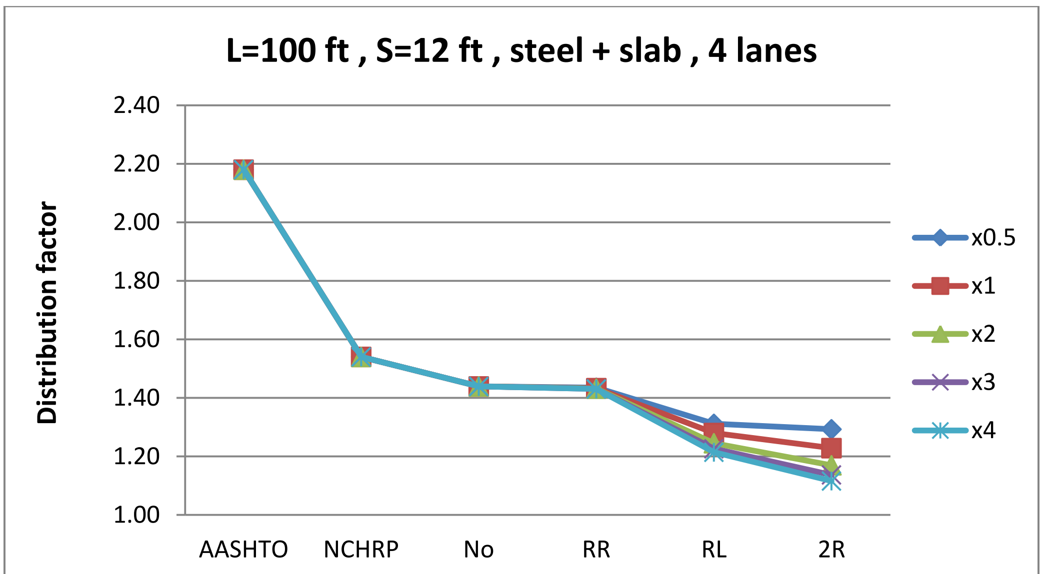
	DF=0.75*Mmax/M0									
	L=100 ft , S=8ft , steel + slab ,4 lanes					L=100 ft , S=8ft , steel only ,4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16
No	1.03	1.03	1.03	1.03	1.03	0.97	0.97	0.97	0.97	0.97
RR	1.03	1.03	1.04	1.04	1.04	0.97	0.97	0.97	0.98	0.98
RL	0.96	0.92	0.90	0.89	0.89	0.89	0.85	0.83	0.82	0.81
2R	0.95	0.92	0.87	0.85	0.84	0.88	0.84	0.80	0.78	0.77

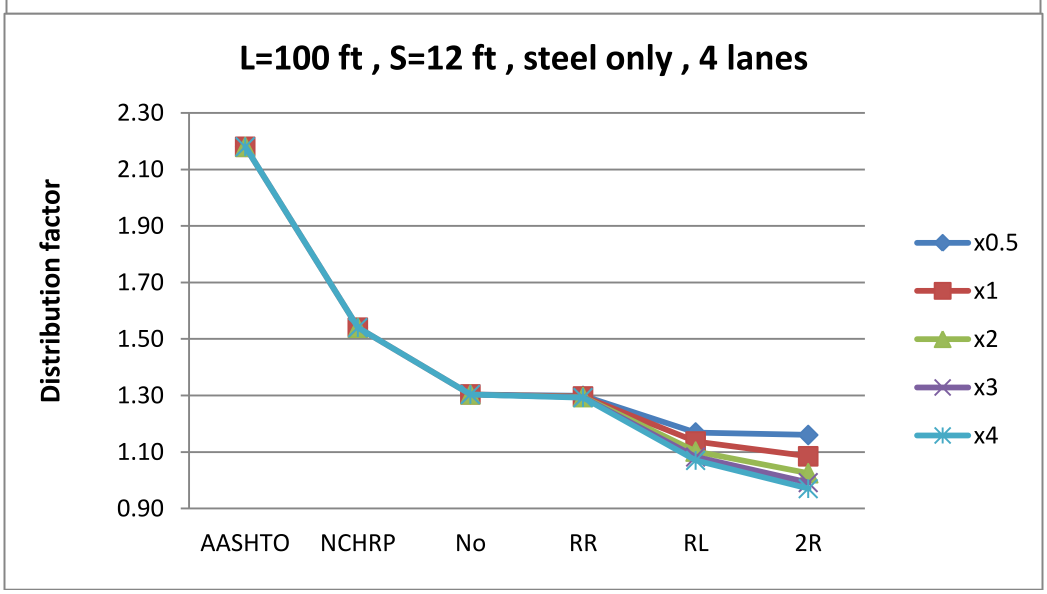




A.22. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=8ft , 4 lanes)

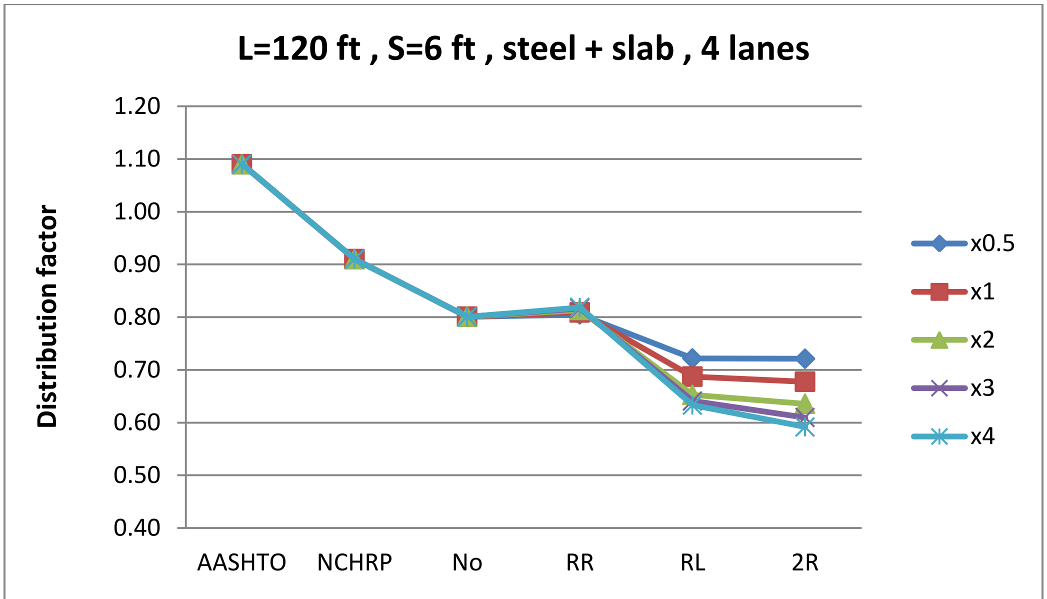
	DF=0.75*Mmax/M0									
	L=100 ft , S=12ft , steel + slab , 4 lanes					L=100 ft , S=12ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54	1.54
No	1.44	1.44	1.44	1.44	1.44	1.30	1.30	1.30	1.30	1.30
RR	1.43	1.43	1.43	1.43	1.43	1.30	1.30	1.29	1.29	1.29
RL	1.31	1.28	1.25	1.23	1.21	1.17	1.14	1.10	1.08	1.07
2R	1.29	1.23	1.17	1.14	1.12	1.16	1.08	1.03	0.99	0.97

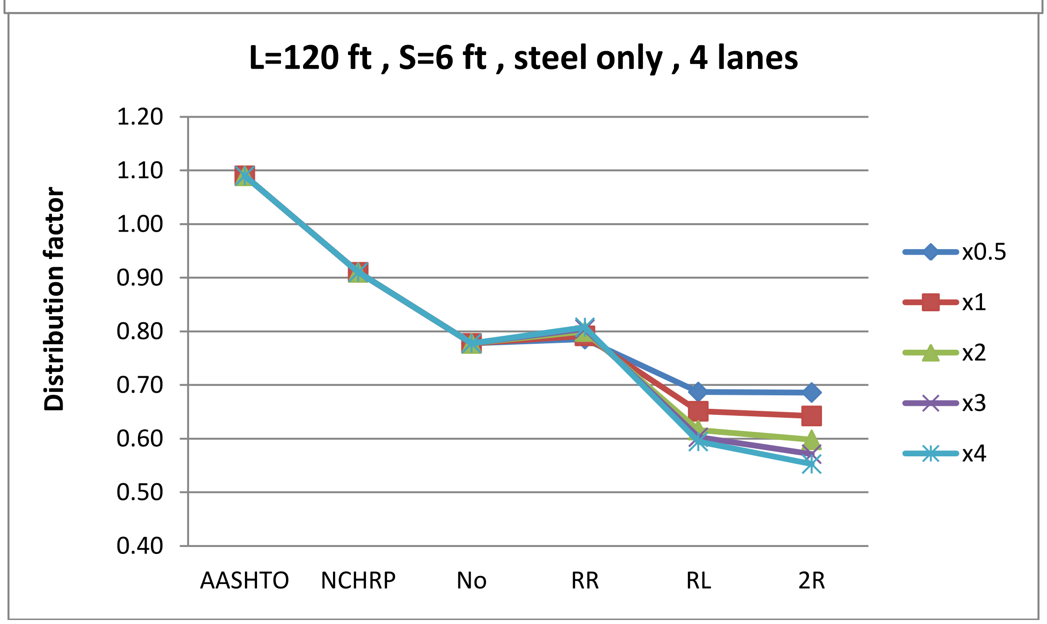




A.51. Sensitivity of Distribution Factor to railing stiffness (L=100ft , s=12ft , 4 lanes)

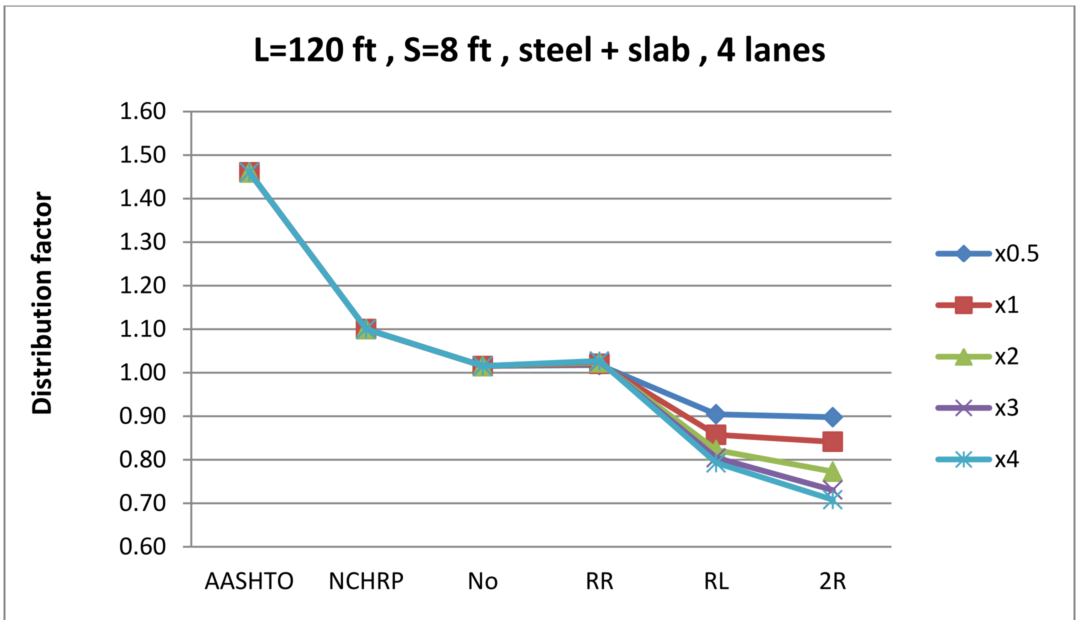
	DF=0.75*Mmax/M0									
	L=120 ft , S=6ft , steel + slab , 4 lanes					L=120 ft , S=6ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09
NCHRP	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
No	0.80	0.80	0.80	0.80	0.80	0.78	0.78	0.78	0.78	0.78
RR	0.81	0.81	0.81	0.82	0.82	0.79	0.79	0.80	0.80	0.81
RL	0.72	0.69	0.65	0.64	0.63	0.69	0.65	0.62	0.60	0.59
2R	0.72	0.68	0.64	0.61	0.59	0.69	0.64	0.60	0.57	0.55

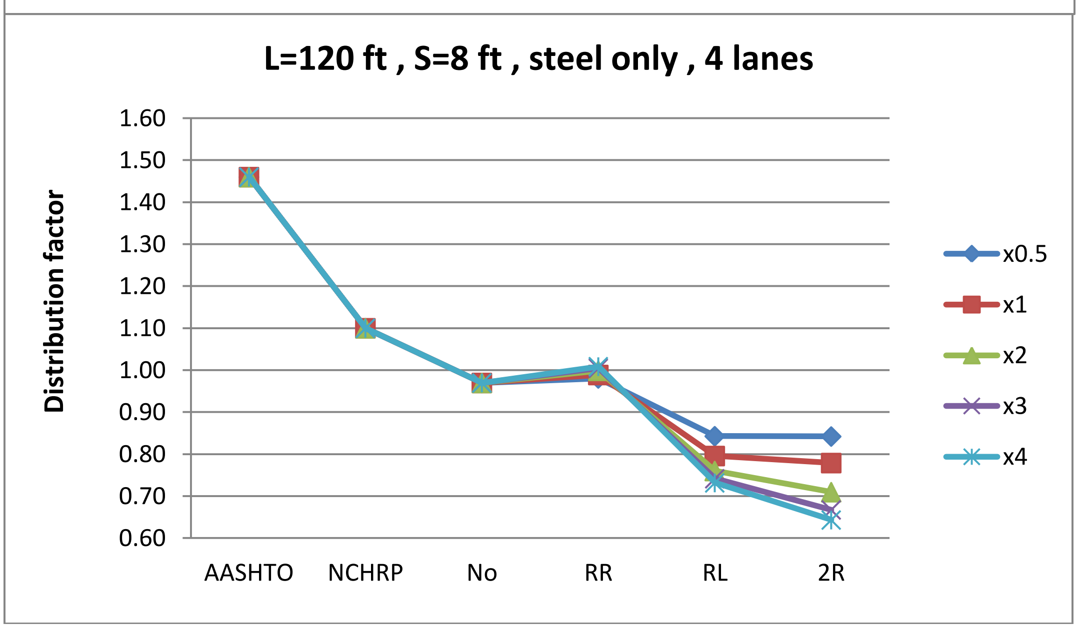




A.52. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=6ft , 4 lanes)

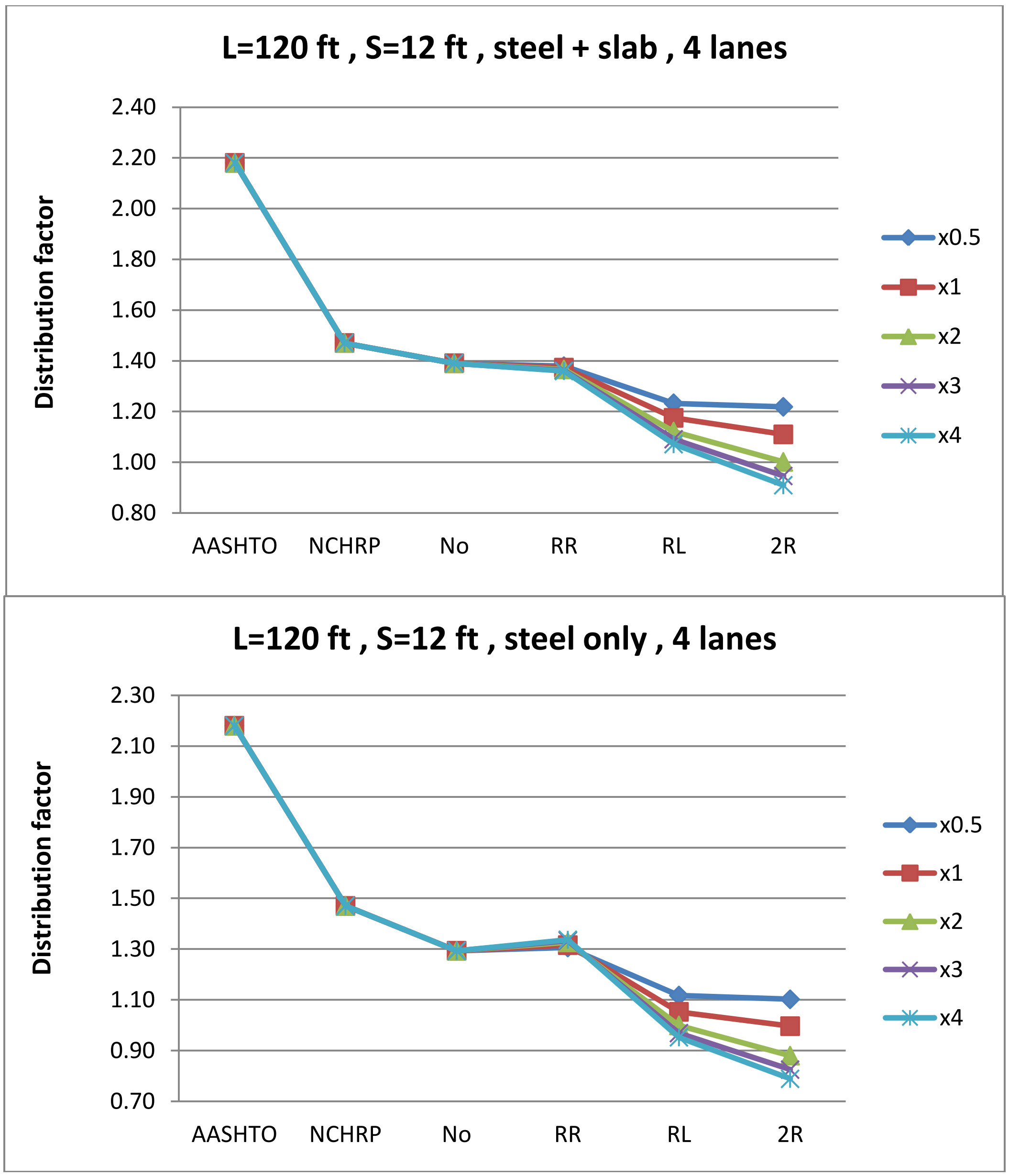
	DF=0.75*Mmax/M0									
	L=120 ft , S=8ft , steel + slab , 4 lanes					L=120 ft , S=8ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46	1.46
NCHRP	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
No	1.01	1.01	1.01	1.01	1.01	0.97	0.97	0.97	0.97	0.97
RR	1.02	1.02	1.02	1.03	1.03	0.98	0.99	1.00	1.00	1.01
RL	0.90	0.86	0.82	0.80	0.79	0.84	0.80	0.76	0.74	0.73
2R	0.90	0.84	0.77	0.73	0.71	0.84	0.78	0.71	0.67	0.64





A.53. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=8ft , 4 lanes)

	DF=0.75*Mmax/M0									
	L=120 ft , S=12ft , steel + slab , 4 lanes					L=120 ft , S=12ft , steel only , 4 lanes				
	x0.5	x1	x2	x3	x4	x0.5	x1	x2	x3	x4
AASHTO	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18	2.18
NCHRP	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47	1.47
No	1.39	1.39	1.39	1.39	1.39	1.29	1.29	1.29	1.29	1.29
RR	1.38	1.37	1.37	1.36	1.36	1.31	1.32	1.33	1.33	1.34
RL	1.23	1.17	1.12	1.09	1.07	1.12	1.05	1.00	0.97	0.95
2R	1.22	1.11	1.00	0.95	0.91	1.10	1.00	0.88	0.82	0.79



A.54. Sensitivity of Distribution Factor to railing stiffness (L=120ft , s=12ft , 4 lanes)

REFERENCES

1. Abou Nooh, M., Mabsout, M., and Tarhini, K. (2015). "Influence of Railings Stiffness on Wheel Load Distribution in Concrete Slab bridges," Journal of Bridge Structures: Assessment, Design, and Construction, IOS Press, Vol. 7(4).

2. Bou Diab, F., Mabsout, M., and Tarhini, K. (2011). "Influence of skew angles on steel girder highway bridges," Journal of Bridge Structures: Assessment, Design, and Construction, IOS Press, Vol. 7(4).

3. Burdette, E. and Goodpasture, D. – NCHRP (1988). "Correction of bridge load capacity estimates with test data," Rep. No. 306, Nat. Cooperative Hwy. Res. Program, Transportation Research Board, Washington, D.C.

4. Fawaz, G., Waked, M., Mabsout, M. and Tarhini, K. (2015) "Influence of railings on wheel load distribution in concrete slab bridges, " submitted to Journal of Bridge Engineering, ASCE (Masters theses by Fawaz, AUB, 2014, and by Waked, AUB, 2010)

5. Hajjar, M., Nuwayhid, S., Mabsout, M., and Tarhini, K. (2015). "Influence of sidewalks and railings on wheel load distribution in steel in continuous multi-lane steel girder bridges," to be submitted to Journal of Bridge Engineering, ASCE (Master"s thesis by Hajjar, AUB, 2014, and research by Nuwayhid, AUB, 2013-14).

6. LRFD Bridge Design Specifications (2010). American Association of State Highway and Transportation Officials (AASHTO), 5th Edition, Washington, D.C.

7. Mabsout, M., Tarhini, K., Frederick, G., and Kesserwan, A. (1998). "Effect of continuity on wheel load distribution in steel girder bridges," Journal of Bridge Engineering, ASCE, 3(3), 103-110.

8. Mabsout, M., Tarhini, K., Frederick, G., and Kesserwan, A. (1999). "Effect of multi-lanes on wheel load distribution in steel girder bridges," Journal of Bridge Engineering, ASCE, 4(2), 99-106.

9. Mabsout, M., Tarhini, K., Frederick, G., and Kobrosly M. (1997). "Influence of sidewalks and railings on wheel load distribution in steel girder highway bridges," Journal of Bridge Engineering, ASCE, August, Vol. 2, No. 3, pp. 88--96.

10. Mabsout, M., Tarhini, K., Frederick, G., and Tayar, C. (1997). "Finite element analysis of steel girder highway bridges," Journal of Bridge Engineering, ASCE, 2(3), 83-87.

11. Menassa, C., Mabsout, M., Tarhini, K., and Frederick, G. (2007). "Influence of skew angle on reinforced concrete slab bridges," Journal of Bridge Engineering, ASCE, 205-213.

12. Nuwayhid, S. Hajjar, M., Mabsout, M., and Tarhini, K. (2015). "Influence of sidewalks and railings on wheel load distribution in single-span multi-lane steel girder bridges," (Master"s thesis by Hajjar, AUB, 2014, and research by Nuwayhid, AUB, 2013-14).

13. SAP2000 (2013). Computers and Structures Inc., Berkeley, California.

14. Standard Specifications for Highway Bridges (2002). American Association of State Highway and Transportation Officials (AASHTO), 17th Edition, Washington, D.C.

15. Tarhini, K. and Frederick, G. (1992). "Wheel load distribution in I-girder highway bridges," Journal of Structural Engineering, ASCE, 118(5), 1285-1294.

16. Waked, M. (2010). "Influence of sidewalks and railings on wheel load distribution in concrete slab bridges," Master thesis, AUB (supervised by author of current proposal).

17. Zokaie, T., Osterkamp, T., and Imbsen, R. (1991). "Distribution of wheel loads on highway bridges," Rep. 12-26/1, Nat. Cooperative Hwy. Res. Program, Transportation Research Board, Washington, D.C.