AMERICAN UNIVERSITY OF BEIRUT

EFFECT OF DETERIORATION OF STIFFENING RAILINGS ON REINFORCED CONCRETE SLAB BRIDGES

by
FATIMA MAHMOUD DARWICH

A thesis
submitted in partial fulfillment of the requirements
for the degree of Master of Engineering
to the Department of Civil and Environmental Engineering
of the Faculty of Engineering and Architecture
at the American University of Beirut

Beirut, Lebanon
December 2016

AMERICAN UNIVERSITY OF BEIRUT

EFFECT OF DETERIORATION OF STIFFENING RAILINGS ON REINFORCED CONCRETE SLAB BRIDGES

by
FATIMA MAHMOUD DARWICH

Approved by:

Dr. Mounir Mabsout, Professor Civil and Environmental Engineering	Advisor
Dr. Salah Sadek, Professor Civil and Environmental Engineering	Member of Committee
Dr. Elie Hantouche, Assistant Professor Civil and Environmental Engineering	Member of Committee

Date of thesis defense: Dec, 20, 2016

AMERICAN UNIVERSITY OF BEIRUT

THESIS, DISSERTATION, PROJECT RELEASE FORM

Fatima Mahmoud Darwich

● Master's Thesis ○ Master's Project ○ Doctoral Dissertation

☐ I authorize the American University of Beirut to: (a) reproduce hard or electronic copies of my thesis, dissertation, or project; (b) include such copies in the archives and digital repositories of the University; and (c) make freely available such copies to third parties for research or educational purposes.

☐ I authorize the American University of Beirut, to: (a) reproduce hard or electronic copies of it; (b) include such copies in the archives and digital repositories of the University; and (c) make freely available such copies to third parties for research or educational purposes
after: **One ---- year from the date of submission of my thesis, dissertation, or project.**
Two ---- years from the date of submission of my thesis, dissertation, or project.
Three ---- years from the date of submission of my thesis, dissertation, or project.

____

Signature Date

ACKNOWLEDGEMENTS

I would like to thank my thesis advisor Prof. Mounir Mabsout for his help, guidance and support throughout my work on this research.

I would also like to thank Professor Salah Sadek and Professor Elie Hantouche for their appreciated roles as committee members.

Finally, I like to express to my gratitude to my parents for providing me with continuous encouragement throughout my years of study and through the process of researching and writing this thesis.

AN ABSTRACT OF THE THESIS

Fatima Mahmoud Darwich for Master of Engineering
Major: Civil Engineering

Title: Effect of Deterioration Of Stiffening Railings On Reinforced Concrete Slab Bridges

The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (2002) or LRFD Bridge Design Specifications (2012) empirical equations do not account for the presence of railings as integral parts bridges and the railings stiffness is neglected during the design stage. When built integrally with the bridge deck, these railings have the effect of stiffening and attracting load to the slab edge and therefore altering the lateral wheel load distribution on concrete slab highway bridges. Previous studies have shown and quantified the increase in the load-carrying capacity of bridge due to presence of railings, which tends to be significant depending on the railing size and bridge geometry. Preliminary studies have also shown that accidental or long-term railings local deterioration may cause high stress or moment concentration in the slab edges which could reach values that even exceed moments in cases when no railings were present. This study will therefore attempt to investigate and quantify the effect of railing deterioration, considering various levels of partial wearing or full breakage at different locations/extents along the span of the bridge railing. Typical one-span, simply-supported, multilane (one and two lanes) straight reinforced concrete bridges with railings on either or both edges of the slab are considered. The finite-element method is used to investigate the effect of railing deterioration occurring on one side of the slab edge. The deterioration is investigated parametrically by varying its location and width, and the extent is modeled by assuming different remaining depth or partial stiffness of the railing. The wheel load distribution and moments in the bridge at the critical sections are evaluated, namely at slab edges where deterioration occurred, with bridges with no railings and bridges with full railings serving as reference bounding cases. AASHTO design trucks loads are placed transversely and longitudinally to produce maximum moments at the critical sections of the slabs. The wheel load distribution and slab moments, and deflections in the bridge slabs at critical sections for bridge cases under study are calculated and compared with the reference cases with full or no railings, as well as with AASHTO procedures. This research will help structural engineers in understanding the effect of railing deterioration and better assess and design straight concrete slab bridges with integral railing; recommendation will also be made to contain and prevent unexpected bridge damage resulting from railing deterioration.

CONTENTS

ILLUSTRATIONS

Figure Page

TABLES

Table Page

CHAPTER 1

INTRODUCTION

1.1 Background

Since the early 1900s, 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 short-span slab bridges to suspension structures. The most common component of all bridges is the bridge superstructure or bridge deck.

Reinforced concrete slab bridges offer economic alternatives for short-span bridges. The main advantage of cast-in-place concrete slab bridges is the ability to provide a smooth finishing surface by field adjustment of the roadway profile during construction. Typically, the design of highway bridges must conform to specifications such as in the American Association of State Highway and Transportation Officials (AASHTO). These include the AASHTO Standard Specifications for Highway Bridges (AASHTO Specs 2002) and the AASHTO Load Resistance Factor Design (AASHTO LRFD 2012). These specifications are based on a thorough understanding of the lateral wheel load distribution on the bridge slab, which is required to develop a realistic design for these highway bridges.

1.2 Design Procedures

The AASHTO design procedures were originally developed in the 1940s, based on the research work of Westergaard (1926, 1930), Newmark (1938), and Jensen (1938, 1939) on moments and stress distribution in reinforced concrete slabs. The analysis, which was based on the classic plate theory, assumed the slabs to be homogeneous and perfectly elastic material. Results for various loading and edge conditions were summarized in tables and charts, developing calculations of various coefficients.

1.3 Research Objectives

The presence of railings built integrally with the bridge deck has the effect of attracting the load to the slab edges and thus altering the lateral wheel load distribution on concrete slab highway bridges. These railings when exposed to long-term or accidental local deterioration lose some of their stiffening effect and subject the bridge slab edges to high moment concentration.

In this research, the influence of integral railings deterioration on wheel load distribution and load-carrying capacity of straight reinforced concrete slab bridges will be investigated and quantified using the finite-element method. AASHTO design trucks (HS20) are positioned, longitudinally and transversally, in order to produce maximum bending moments. The cases of straight bridges with no railings and straight bridges with full railings will serve as reference cases

The wheel load distribution, slab moments and deflections in the bridge slabs at critical sections for the bridge cases under study are calculated and compared with the reference bridge cases and with AASHTO procedures. Recommendations related to the interpretation of the effect of railings' deterioration on straight reinforced concrete bridges will be proposed to bridge engineers.

1.4 Scope and Methodology of Proposed Research

The current research will present the finite element results of a parametric study to evaluate the effect of deterioration of railings on wheel load distribution in straight reinforced concrete slab highway bridges.

Typical one-span, simply supported, one and two lanes, straight reinforced concrete slab bridges with railings on either or both edges of the slab are considered. A parametric study will be conducted with a variable deterioration widths and depths.

In the finite element method, the bridge slab is discretized into a convenient number of elements, which are assumed to be interconnected at nodal points; each element has the properties corresponding to the original structure. In this research, the finite element model of the bridge consists of square shell elements of size 1ft x 1ft (0.3m x 0.3m) for the slab, rectangular shell elements of size 1ft x 1.25ft (0.3m x 0.38m) for the railing, and hinged/roller supports for the piers. The finite element program SAP2000 (2012) is used for the analysis.

The finite element method is used to investigate the effect of railing deterioration width and depth on simply supported, one-span, one-lane and two-lane straight reinforced concrete slab bridges with railings on either or both edges of the slab. Two typical span lengths are considered: 36 ft (10.8 m) and 54 ft (16.2 m). The slab widths are assumed to be: 14 ft (4.2 m) for one-lane and 24 ft (7.2 m) for two-lanes. One typical rectangular cross-section of 8 in x 30 in (20 cm x 76 cm) is assigned to railings. Five railing deterioration widths will be considered: 0, 1, 2, 4, and 8 ft (0, 0.3, 0.6, 1.2, and 2.4 m), where the 0 ft correspond to the full railing case without deterioration. Two deterioration depths will be considered: 30in (76cm) and 15in (38cm) assumed to represent full and half breakage of the railing, respectively. The critical location of deterioration will be near the critical sections of the slab (i.e. near the centerline of the span).

Design trucks are assumed to be traveling in the same direction. Transversally, edge loading condition is considered. In the edge loading, the HS20 design trucks are placed side-by-side close to one edge of the slab, such that the center of the left wheel of the leftmost truck is positioned at 1 ft (0.3 m) from the left edge of the slab. The distance between the adjacent trucks is selected to be 4 ft (1.2 m) to produce the worst loading condition on the bridge. Various positions of the design trucks are assumed, longitudinally and transversally, in order to produce maximum bending moments. The cases of straight bridges without railings and straight bridges with full railings placed at either or both edges of the slab are considered as the reference cases. Railings are then modified for various deterioration cases considered. The wheel load distribution, moments and deflections in the bridge slabs at the critical section for bridge cases under study are calculated and compared to that of the reference cases and AASHTO procedure.

1.5 Thesis Organization

The thesis is divided into five chapters including this introduction Chapter 1. Chapter 2 is a general description of the research work including a description of reinforced concrete slab bridges, AASHTO Standard Specifications and LRFD design procedures. Chapter 3 includes a description of the bridge cases considered and the finite element models used in the analyses. Chapter 4 discusses the effect of railings deterioration on the different bridge models considered with tables showing the different results and assessment with AASHTO design standards. Conclusions and recommendations are presented in Chapter 5.

CHAPTER 2

BACKGROUND AND AASHTO DESIGN PROCEDURES

2.1 Introduction

In this chapter, a background section is presented, and, for later comparison between FEA results and conventional methods of bridge design, a summary of AASHTO Standard Specifications (2002) and AASHTO LRFD (2012) design procedures are provided.

2.2 Background Studies

A concrete slab bridge is designed according to the provisions for main reinforcement parallel to traffic. The AASHTO design procedures were originally developed in the 1940s, based on the research work of Westergaard (1926, 1930) and Jensen (1938, 1939) and is presented in AASHTO Standard Specifications for Highway Bridges (2002) (section 3.24 " Distribution of Loads and Design of Concrete Slabs").

Mabsout et al. (2004) reported the results of a parametric investigation, using finite-element analysis (FEA), of straight, single-span, simply-supported reinforced concrete slab bridges. The study considered various span lengths, slab widths with varied number of lanes, and live loading conditions for bridges, with and/or without shoulders. Longitudinal bending moments and deflections in the concrete slab were evaluated and compared with procedures specified by AASHTO Standard Specifications and LRFD. However, this published research did not consider the effect of railings on the load carrying capacity of concrete slab bridges. The research results indicated that AASHTO Standard Specifications slab moments overestimated the FEA moments by 30% for one lane with span length up to 7.5 m, and the AASHTO slab moments agreed with the FEA moments for spans longer than 8 m. When considering two or more lanes with spans up to 10.5 m, AASHTO slab moments were similar to FEA moments. However, as the span lengths increases, AASHTO slab moments were less than the FEA bending moment by 15 to 30%. The AASHTO LRFD procedure gives higher bending moments than AASHTO Standard Specifications as well as the FEA results.

Mabsout et al. (2004) Davids et al. (2013) reported the development of finite element analysis software designed specifically for the load rating of flat slab bridges. The FEA software formulation and convergence were verified with commercial FEA software. Results of live load tests of an instrumented, in-service flat slab bridge were also reported. The FEA model predicted slab moments that were shown to be conservative relative to the moments inferred from the load test data for a range of truck positions. Fourteen in-service flat slab bridges were load rated with both FEA analysis and the AASHTO equivalent strip method to assess the degree of conservatism inherent in the AASHTO approximate analysis. The FEA results showed an average increase in rating factor of 26% for short-span, two lane flat-slab bridges, when compared with the AASHTO strip width method.

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 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%.

Eamon and Nowak (2002) reported the contribution of bridge stiffening elements such as barriers (railings), sidewalks, and diaphragms on the ultimate capacity and wheel load distribution in composite steel and prestressed concrete girder bridges. Typical secondary elements were shown to reduce girder distribution factors by 10 to 40% depending on the stiffness and bridge geometry. It was also shown that the bridge system ultimate capacity was increased from 1.1 to 2.2 times that of the reference bridge without secondary elements.

Chung et al. (2006) conducted a study investigating 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 railings produces load distribution factors up to 40% lower than the AASHTO LRFD values.

Conner and Huo (2006) investigated the effect of railings and bridge aspect ratio on live-load moment distribution in 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 railings 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 railing 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 or railings were included in the analysis.

Fawaz et al. (2016) studied the influence of railings on load carrying capacity of simply-supported, one-span, multi-lane reinforced concrete slab bridges using the finite element method. A total of 112 bridge cases were modeled using finite element analysis (FEA) subject to AASHTO HS20 truck loadings positioned transversally and longitudinally to produce the maximum bending moments. Typical railings were placed on either edge or both edges of concrete slab bridges. The maximum bending moments and deflections were calculated using the FEA results for cases with and without railings which were compared with AASHTO procedures. For bridges without railings, AASHTO Standard Specifications overestimated the FEA bending moment by 20% for one-lane bridges, and it underestimated the FEA bending moments by 20% for bridges with two or more lanes. Placing two railings on the bridge, AASHTO Standard Specifications overestimated the FEA moments by 100% for one-lane bridges, and by 20% for bridges with two or more lanes. For bridges without railings, the AASHTO LRFD design procedure overestimated the FEA longitudinal bending moment by an average of 50% for one-lane, 25% for two-lanes, and gave similar results to FEA for three- and four-lane concrete slab bridges. With the presence of two railings, the AASHTO LRFD significantly overestimated the FEA moments in all bridge cases by 150% for one-lane, 70% for two-lanes, and a 30% for three- and four-lanes. Similar patterns were also observed for the FEA edge beam moments which were also compared to AASHTO procedures. The maximum live load deflection decreased due to the addition of railings and was most significant in bridges with one- and two-lane slab bridges.

The study done by Fawaz et al. (2016) will form the basis for the current research which will address the influence of deterioration of integral railings on straight reinforced concrete slab bridges.

2.3 AASHTO Standard Specifications for Highway Bridges

2.3.1 Slab Design

A concrete slab bridge is designed with the provisions for main reinforcement parallel to traffic. AASHTO specifies a distribution width for highway loading or an empirical formula to reduce the two-way bending problem into a beam (one-way) bending problem. Therefore, reinforced concrete slab bridges are typically designed as a series of beam strips. AASHTO Standard Specifications (2002) suggest three approaches to determine the live load bending moment for HS20 loading. One approach, which will be adopted for the assessment in this study, is described below.

Section 3.24.3.2 of AASHTO (2002) provides empirical equations for the longitudinal bending moment M per foot width, for the case of main reinforcement parallel to traffic and is applicable only to simple spans.

- M_{AASHTO} **(Kip-ft/ft) =0.9S** **for S <50 ft** **(1a)**

or

- M_{AASHTO} **(Kip-ft/ft) =(1.30S-20)** **for 50 ft < S <100 ft** **(1b)**

where S=span length in feet.

The analysis of bridges according to the AASHTO must consider both truck and lane loading, with the design being based on the governing of the two load cases. However for short-span structures, the truck loading governs the design.

Also, AASHTO gives special provisions for transverse reinforcement placed perpendicular to the main steel reinforcement in bridge slabs. The amount of distribution reinforcement is given as a percentage of the main reinforcement equal to $100/(S)^{1/2}$, where S is in feet, and shall not exceed 50%.

2.3.2 Edge Beam

According to section 3.24.8, a longitudinal AASHTO edge beam moment of a simple span is provided for slabs having main reinforcement parallel to traffic as:

- **M_{edge_AASHTO} (Kip-ft) =0.1xPxS** **(2)**

where:

P=16 Kips for the AASHTO HS20 design truck;

S = the span length in feet.

AASHTO does not specify a width for the edge beam. However, some departments of transportation use an edge beam width of 1.5 ft, which leads to:

- **M_{edge_AASHTO} (kip-ft/ft) = 0.1xPxS/1.5** **(3)**

2.3.3 Live Load Deflection

AASHTO maximum live load deflection D for simple or continuous spans (section 8.9.3.1) shall not exceed:

- **D (in)** $= \dfrac{S}{800}$ where S is the span length of the bridge in inches **(4)**

2.4 AASHTO Load Resistance Factor Design (LRFD)

2.4.1 Slab Design

According to AASHTO LRFD (2012) section 3.6.1.2.1, the vehicular live loading on the roadways of bridges shall consist of a combination of design truck HS20 (section 3.6.1.2.2) or tandem (section 3.6.1.2.3) with design lane load (section 3.6.1.2.4) similar to the AASHTO Standard Specifications lane load (AASHTO Specs fig 3.7.6B) and consists of a uniformly distributed load in the longitudinal direction of 0.64 Kip/ft and occupying 10 ft transversally.

AASHTO LRFD section 4.6.2.3 provides an equivalent strip width to design slab bridges similar to the previous bridge specifications. This simplistic approach is to divide the total statical moment M_0 by the bridge equivalent width E to achieve a moment per unit width for design. The equivalent width E of longitudinal strips per lane for both shear and moment is determined using the following formulas:

The width for one lane (two lines of wheels) loaded is:

- **$E=10+5(L_1 x W_1)^{1/2}$ in inches (LRFD Equation 4.6.2.3-1) (5a)**

The width for multilane loaded is:

- **$E=84+1.44(L_1 x W_1)^{1/2} \le \frac{W}{N_L}$ in inches (LRFD Equation 4.6.2.3-2) (5b)**

where:

E=equivalent width in inches;

L_1=span length in feet taken equal to the lesser of the actual span or (60 ft);

W_1=modified edge-to-edge width of bridge taken to be equal to the lesser of the actual width or (60 ft) for multi-lane loading, or (30 ft) for single-lane loading;

W=physical edge-to-edge width of bridge;

N_L= number of design lanes.

The live load longitudinal bending moment M is therefore obtained as:

- **M_{LRFD} (Kip-ft/ft) =** $\frac{M_0}{E}$

2.4.2 Edge Beam

AASHTO LRFD edge beam moment (article 4.6.2.1.4b) shall be assumed to support one line of wheel load and a tributary portion of the design lane load. Where the effective width is the sum of the distance between the edge of the deck and the inside face of the barrier (assumed equal to 1 ft), plus 1 ft, plus one quarter of the strip width specified above, but shall not exceed either one-half the full strip width or 6 ft.

2.4.3 Live Load Deflection

AASHTO LRFD maximum deflection D for simple or continuous spans (article 2.5.2.6.2) shall not exceed:

- **D (in)** $= \frac{S}{800}$ where S is the span length of the bridge in inches **(7)**

CHAPTER 3

BRIDGE CASES DESCRIPTION, MODELING AND ANALYSIS

3.1 Introduction

This chapter presents the parametric study carried out on the analysis of reinforced concrete slab bridges. The various geometric and physical characteristics of the bridges as well as the different railings configurations and loading patterns are presented. The chapter also outlines the three-dimensional (3D) finite element modeling technique adopted and summarizes all the bridge cases considered.

3.2 Bridge Cases Description

3.2.1 Geometry and Dimensions

A total number of one hundred twelve geometrically distinct simply supported one-span reinforced concrete slab bridge cases are considered in the study, whereby the following geometrical properties are varied:

- Span length
- Number of lanes
- Presence of railings
- Transverse loading position
- Deterioration of railings (width and depth)

The two span lengths considered, with the corresponding slab thicknesses chosen to control deflection, are as follows:

- Span length of 36 ft with slab thickness of 21 inches

- Span length of 54 ft with slab thickness of 27 inches

A typical lane is considered to have a fixed width of 12 ft. Cases of one-lane bridges have an additional 1 ft width of slab on each side. For the number of lanes considered, one and two, the corresponding slab widths are as follows:

- 14 ft for one-lane bridges (1+1x12+1= 14 ft)
- 24 ft for two-lane bridges (2x12 = 24 ft)

Other parameters of this study is the width and depth of deterioration. Five widths of deterioration are considered including 0, 1, 2, 4, and 8 ft with the 0 ft corresponding to the full railing case, or railing without deterioration. Two deterioration depths are considered: 30 in and 15 in assumed to represent full and half breakage of the railing, respectively. The location of deterioration is considered to be near the centerline of the bridge span.

When present, railings are 8 in wide and 30 in deep above slab. These railings may be on either or both edges of the bridge. Figure 3.1 illustrates typical cross-sections for one-lane and two-lane bridge cases with/without railings. Figure 3.2 illustrates typical cross-sections for one-lane and two-lane bridge cases with half/fully deteriorated railings. Table 3.1 summarizes the geometrical characteristics and dimensions of all the bridge cases analyzed.

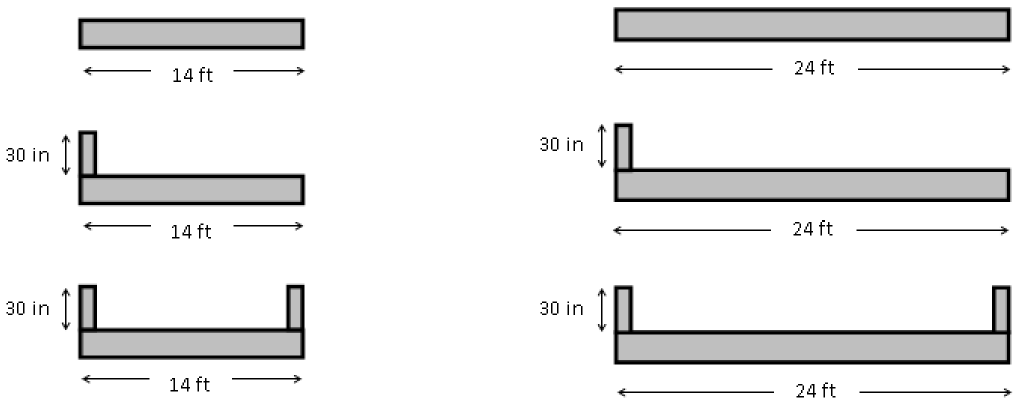


Figure 3.1: Typical Cross Sections for One-lane and Two-Lane Bridge Cases with/without Railings

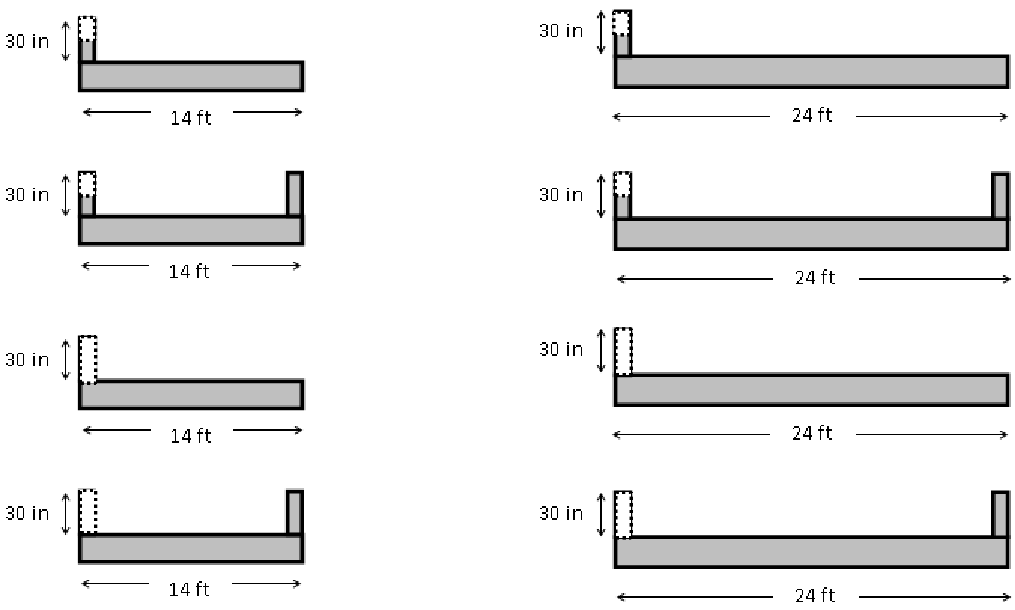


Figure 3.2: Typical Cross Sections for One-lane and Two-Lane Bridge Cases with Half/Fully Deteriorated Railings

Table 3.1: Geometrical Characteristics and Dimensions of Modeled Bridges

No. of Lanes	Span Length (ft)	Slab Thickness (in)	Slab Width (ft)	Depth of Deterioration (in)	Width of Deterioration (ft)
1	36	21	14	15 and 30	0, 1, 2, 4 and 8
	54	27			
2	36	21	24	15 and 30	0, 1, 2, 4 and 8
	54	27			

3.2.2 Physical Properties

The material properties of the normal-strength concrete adopted in the study are as follows:

- Compressive Strength: f'_c (28 days) = 4,000 psi
- Modulus of Elasticity: $E_c = 3.60 \times 10^6$ psi
- Poisson's Ratio: $v = 0.2$

3.2.3 AASHTO Design Truck

The analysis and design of any highway bridge must consider truck and lane loading. However, truck-loading provisions govern for short-span structures when considering AASHTO Standard Specifications (2002). Therefore, the bridges in this study are analyzed for HS20-44 Truck load as given in AASHTO (Figure 3.3). The maximum weight of this truck is 72 Kips distributed over two rear axles and one front axle as follows:

- 32 Kips for each of the rear axles
- 8 Kips for the front axle

The three axles are equally spaced at 14 ft.

3.2.4 Longitudinal Loading Position of Design Trucks

The maximum moment is determined when the midpoint between the center of gravity of the HS-20 truck and the center load (at the middle axle) coincides with the mid-span of the bridge (Refer to Figure 3.4.a). In the study by Mabsout et al. (2004) on straight concrete bridges, it is shown that minor deviations in maximum positive moment occur if the truck is positioned with its center load coinciding with the mid span of the bridge (Refer to Figure 3.4.b). The same simplification mentioned above is adopted in our study on straight concrete bridges. Table 3.2 shows the longitudinal truck position for the various span lengths considered.

3.2.5 Transverse Loading Position of Design Trucks

AASHTO HS20 design trucks are assumed to be traveling in the same direction on the bridge. Transversally, Edge loading condition is considered since it is always governing according to the studies done for bridges without railings (Mabsout et al., 2004) and bridges with railings (Fawaz et al., 2016) In the Edge loading condition the design trucks are placed side-by-side close to one edge (left) of the slab, such that the center of the left wheel of the leftmost truck is positioned at one foot from the left edge of the slab; the distance between the adjacent trucks is selected to be 4 ft and produce worst loading condition on the bridge (Refer to Figure 3.5).

Because of the One Railing Case, there are two edge loading conditions E1 and E2. E1 where the design truck is placed to the side of the railing in order to get maximum moment in the railing and E2 where the design truck is placed on the opposite side of the railing to get maximum moment in the slab.

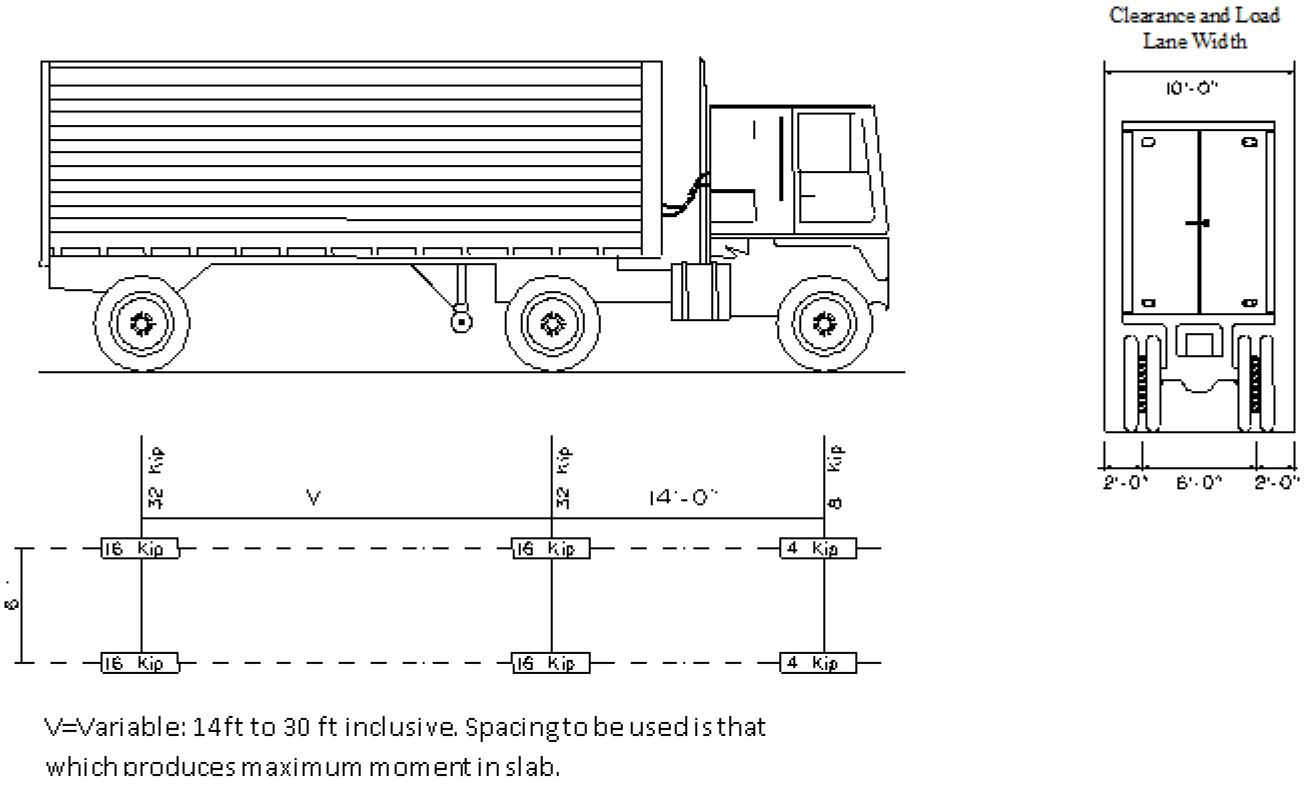


Figure 3.3: AASHTO HS-20 Design Truck (Source: AASHTO Standard Specifications for Highway Bridges, 2002**).**

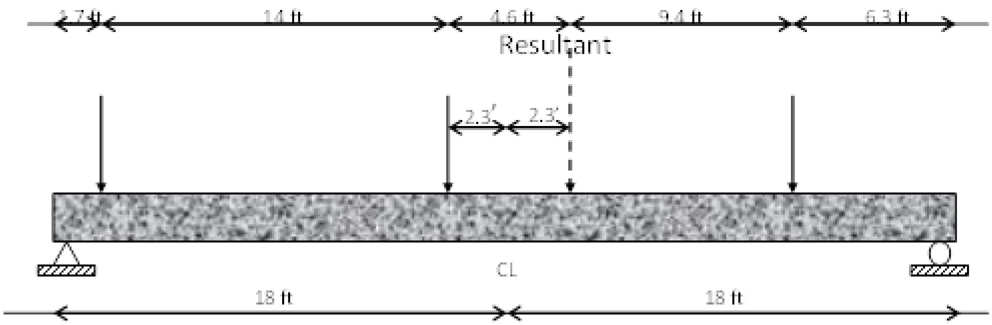


Figure 3.4 (a): Longitudinal Truck Position in a Typical 36 ft One-Span Bridge for Maximum Positive Bending Moment.

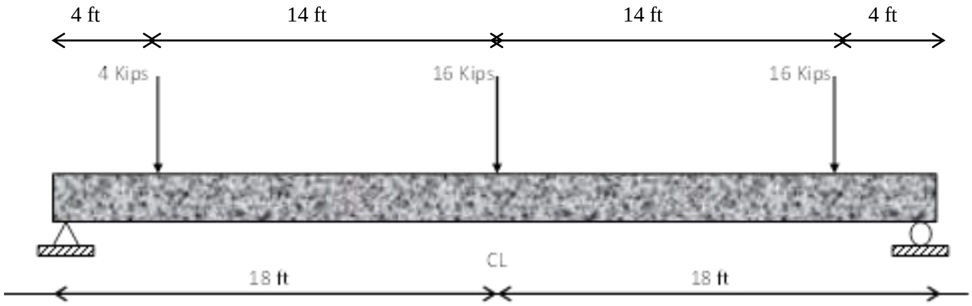
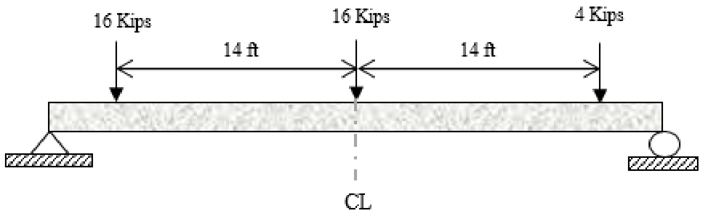
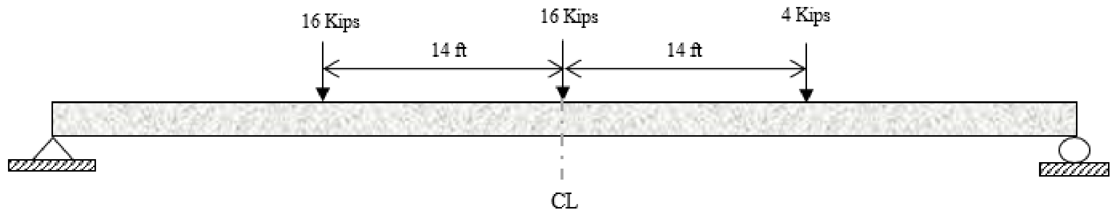


Figure 3.4 (b): Assumed Longitudinal Truck Position in a Typical 36 ft One-Span Bridge for Maximum Positive Bending Moment in the Current Study.

Table 3.2: Longitudinal Truck Position in One-Span Bridges for Maximum Positive Moment at Centerline

Span S (ft)	Thick t (in)	Truck Loading Position
36	21	
54	27	

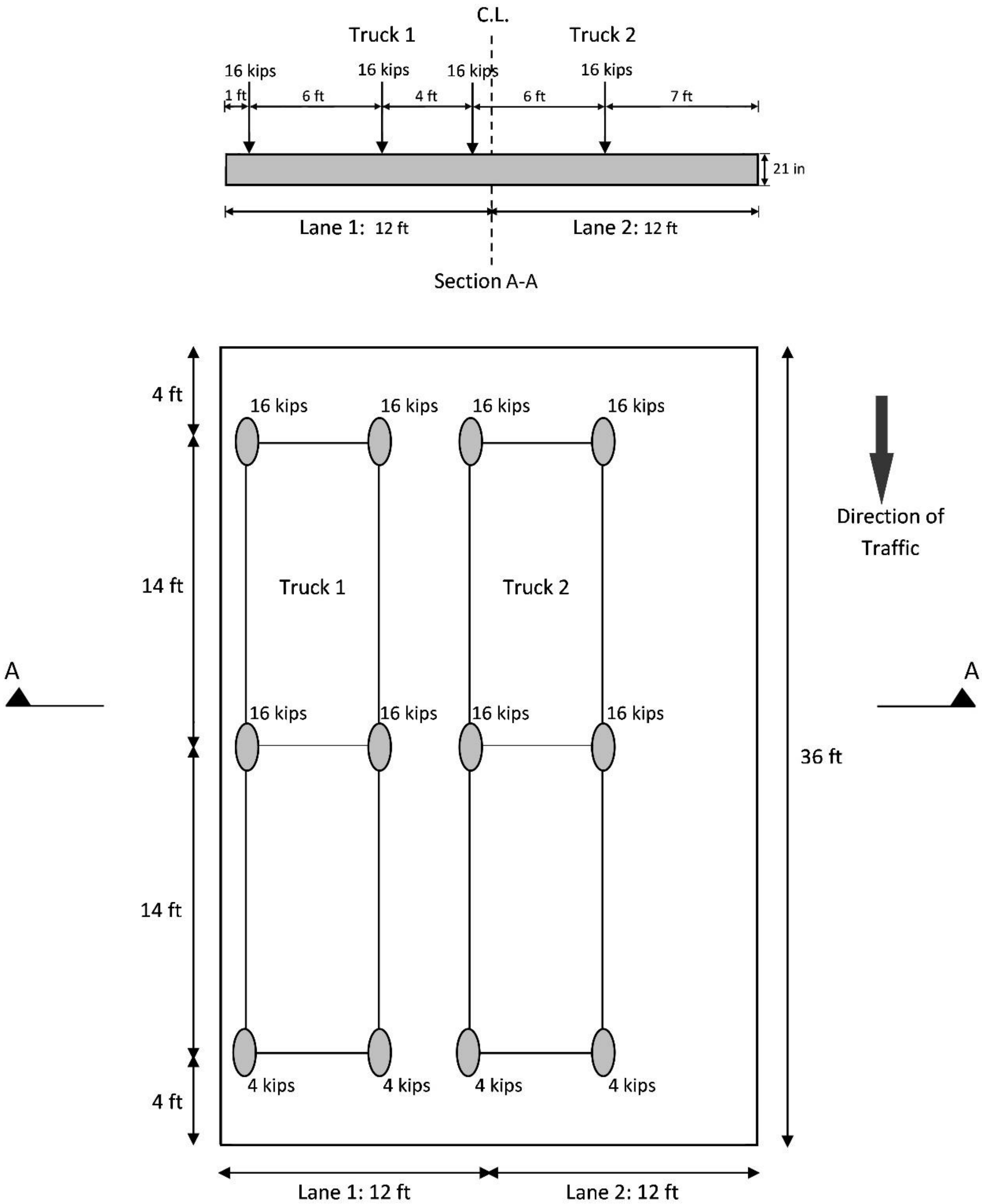
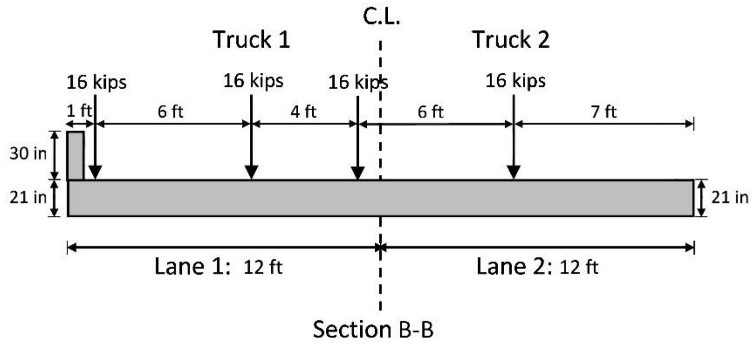


Figure 3.5: Typical Cross-Section and Plan of a Two-Lane 36 ft Span Straight Bridge with No Railings under Edge Loading Condition**.**

3.2.6 Deteriorated Railings Implementation Methodology

The railings are assumed to be constructed integrally with the bridge slab providing stiffness to the slab edges. However the railings may be subjected to accidental local deterioration, such as full or half breakage of portion of the railing that affects their stiffness and alters the wheel load distribution on the slab. The bridge cases without railings and those with non-deteriorated railings, placed integrally at either or both edges of the slab, are considered as the reference cases. The railing is then deteriorated near the centerline of the bridge span by the assumed width and depth corresponding to each deterioration case. The design trucks are positioned, longitudinally and transversally, in order to produce maximum bending moments. Figures 3.6 to 3.11 show typical cross sections and plans of straight bridges with different combinations of transverse loading conditions and railings with and without deterioration.

The wheel load distribution on the bridge slab at the critical section for the reference and deteriorated railings bridge cases are calculated and compared. The results are also assessed with the AASHTO Standard Specifications (2002) and AASHTO LRFD (2012) procedures.



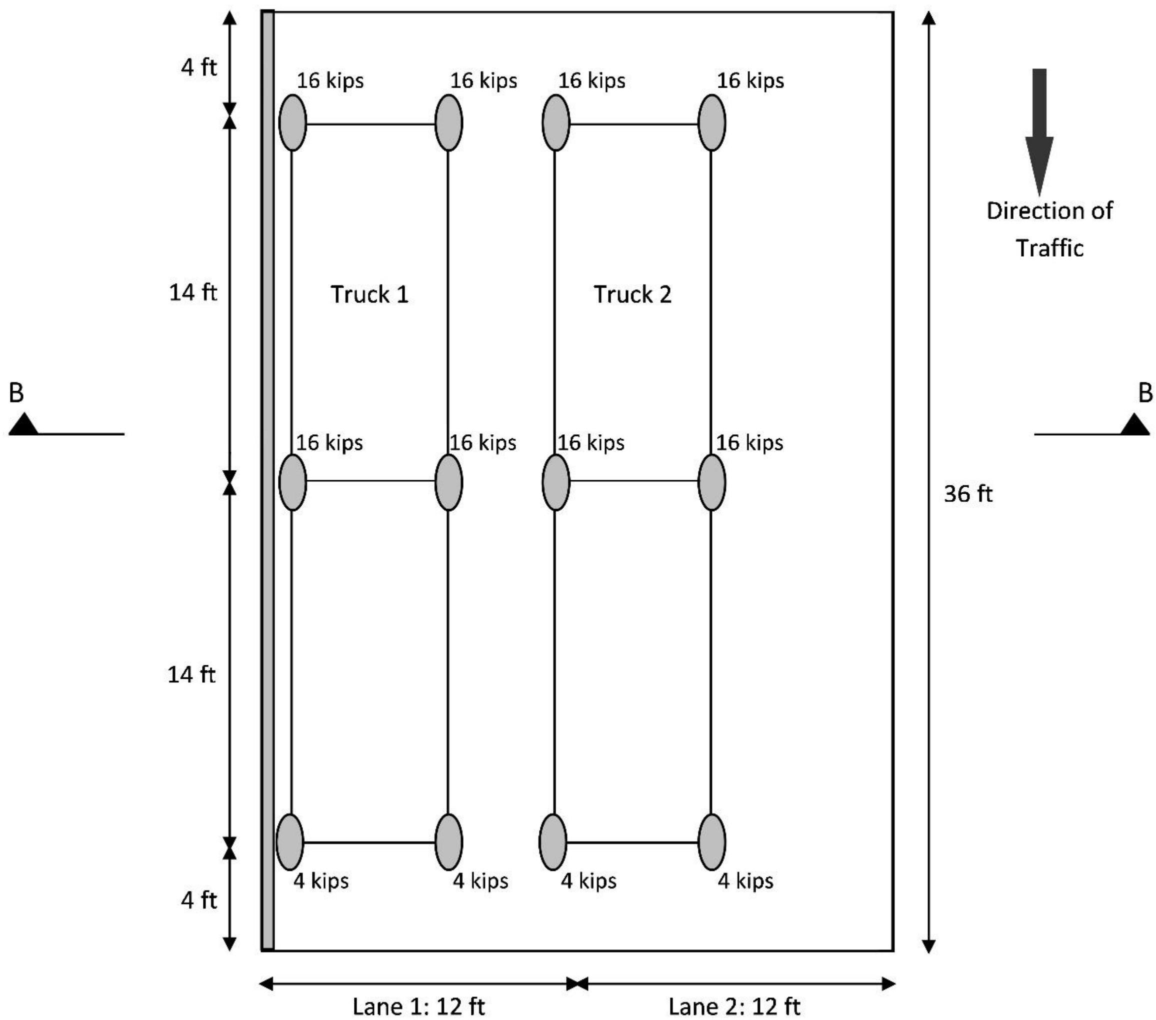


Figure 3.6: Typical Cross-Section and Plan of a Two-Lane 36 ft Straight Bridge with One Railing without deterioration under Edge Loading Condition E1.

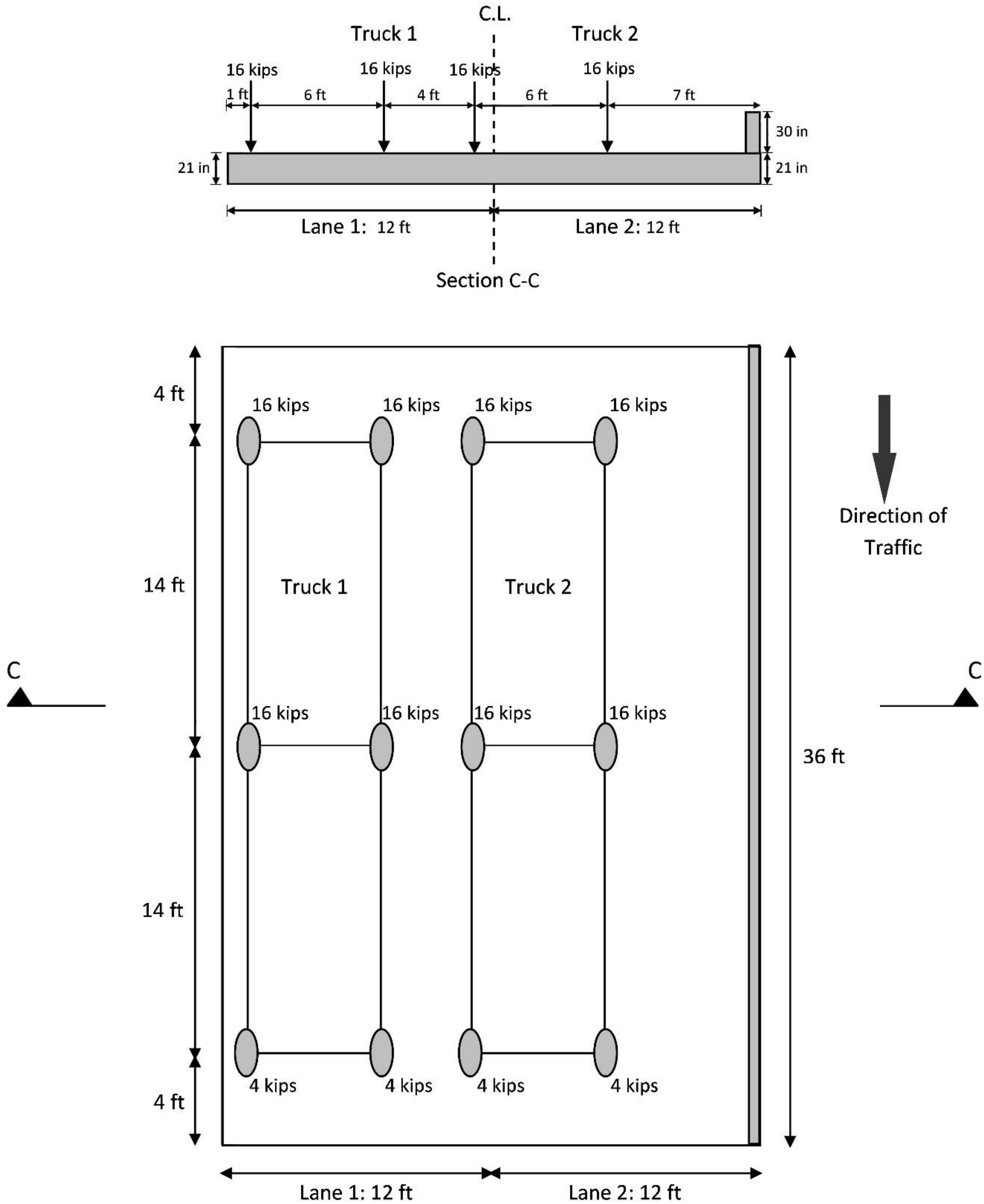


Figure 3.7: Typical Cross-Section and Plan of a Two-Lane 36 ft Span Straight Bridge with One Railing without deterioration under Edge Loading Condition E2.

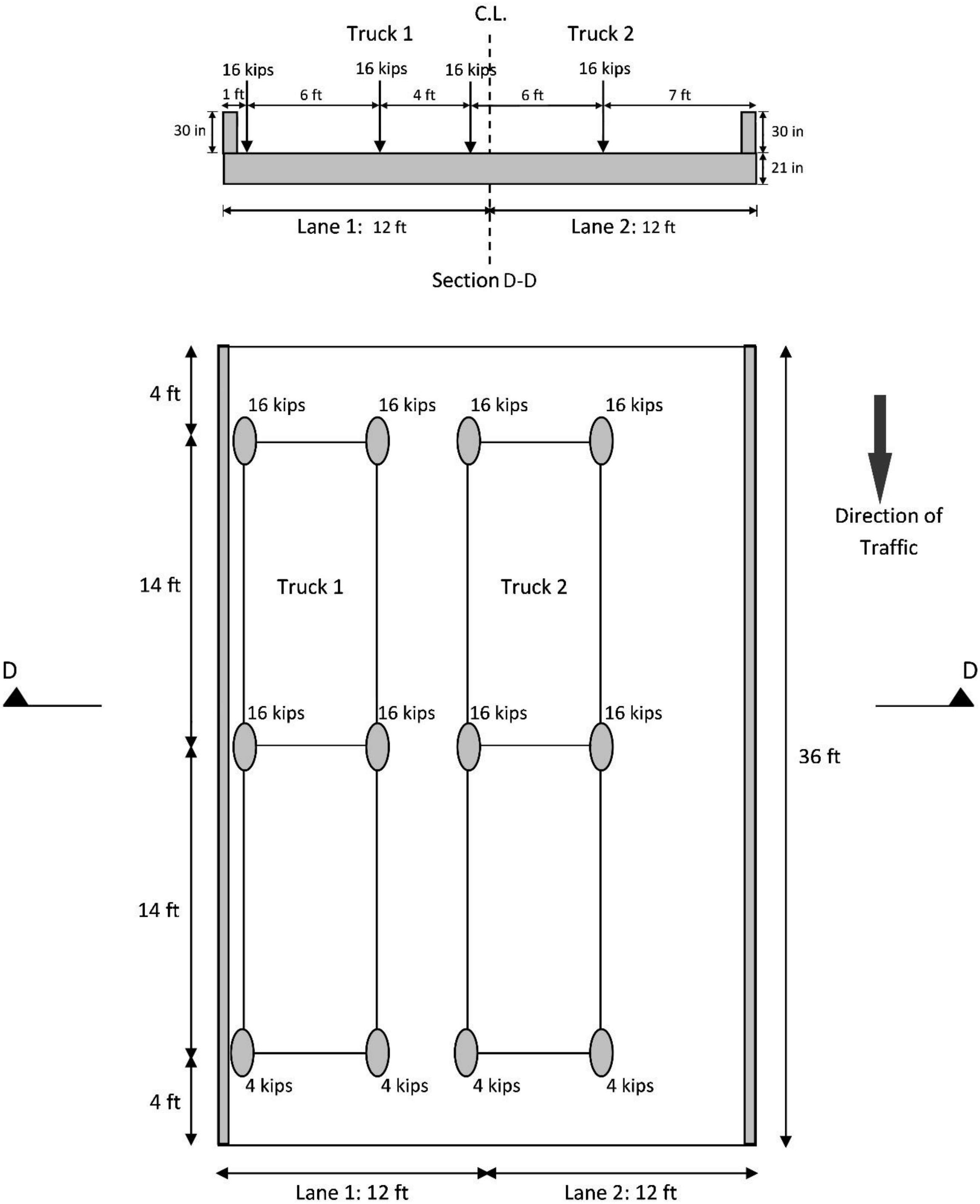


Figure 3.8: Typical Cross-Section and Plan of a Two-Lane 36 ft Span Straight Bridge with Two Railings without deterioration under Edge Loading Condition E1.

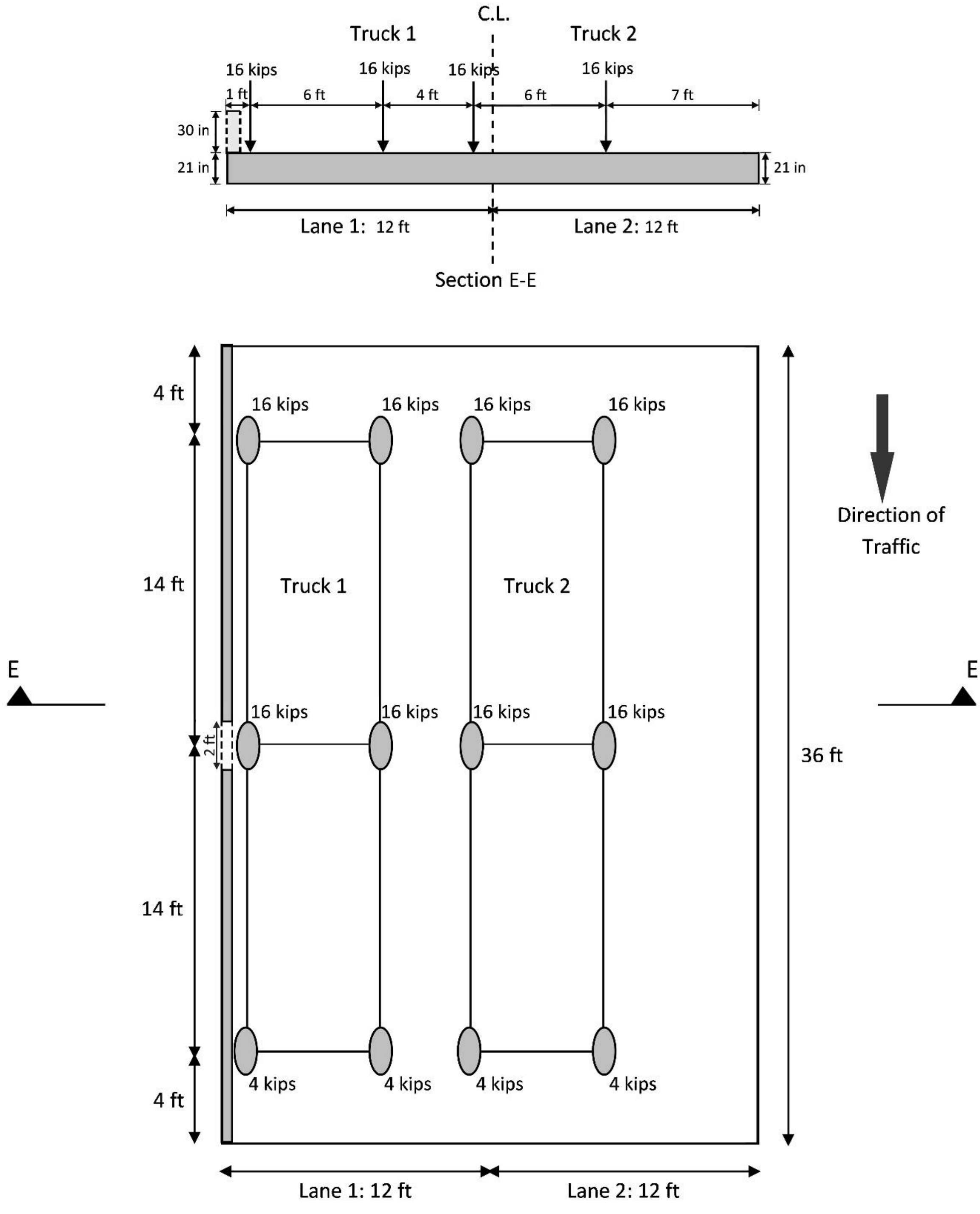


Figure 3.9: Typical Cross-Section and Plan of a Two-Lane 36 ft Span Straight Bridge with One Railing with full-deterioration width of 2 ft under Edge Loading Condition E1

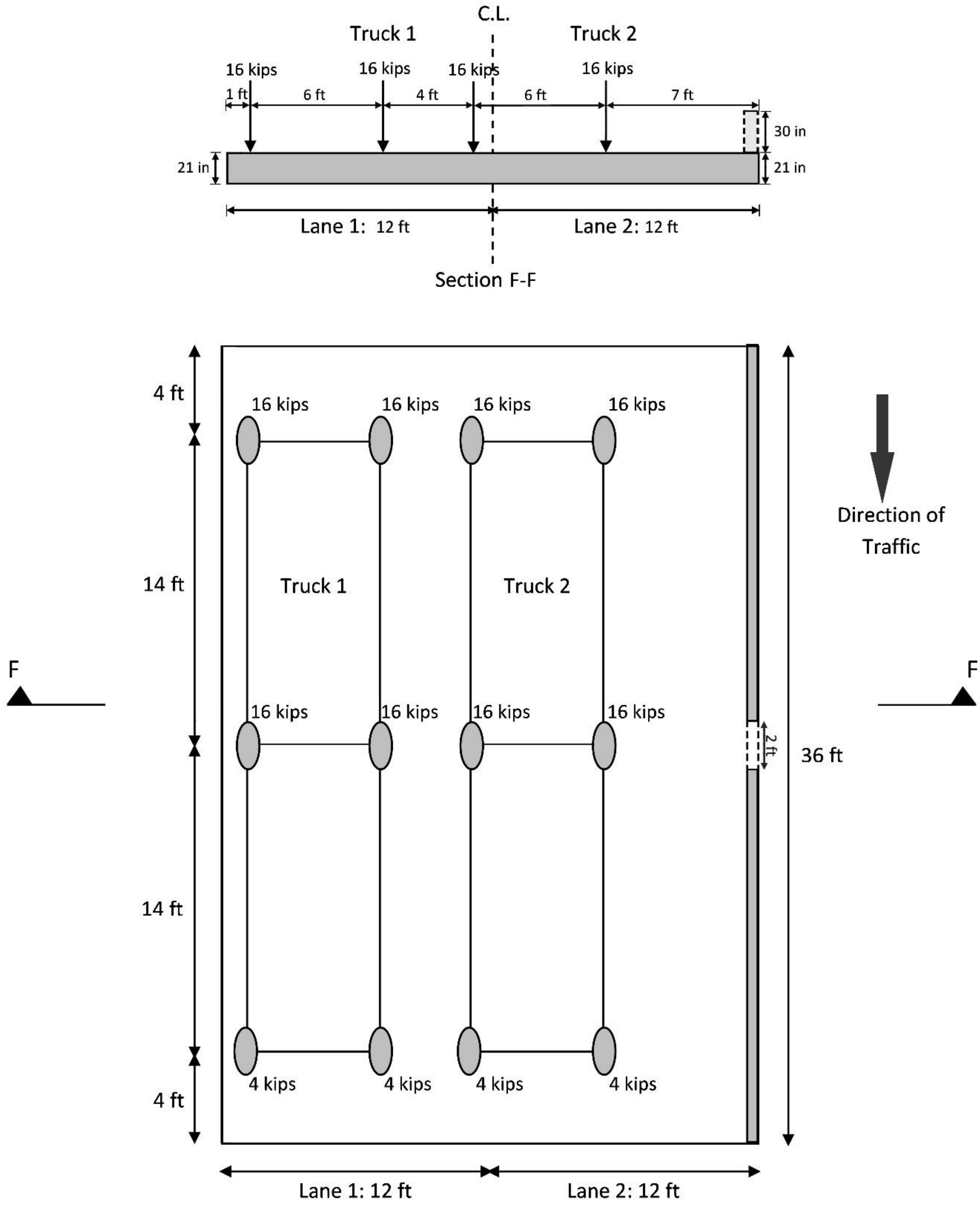


Figure 3.10: Typical Cross-Section and Plan of a Two-Lane 36 ft Span Straight Bridge with One Railing with full-deterioration width of 2 ft under Edge Loading Condition E2.

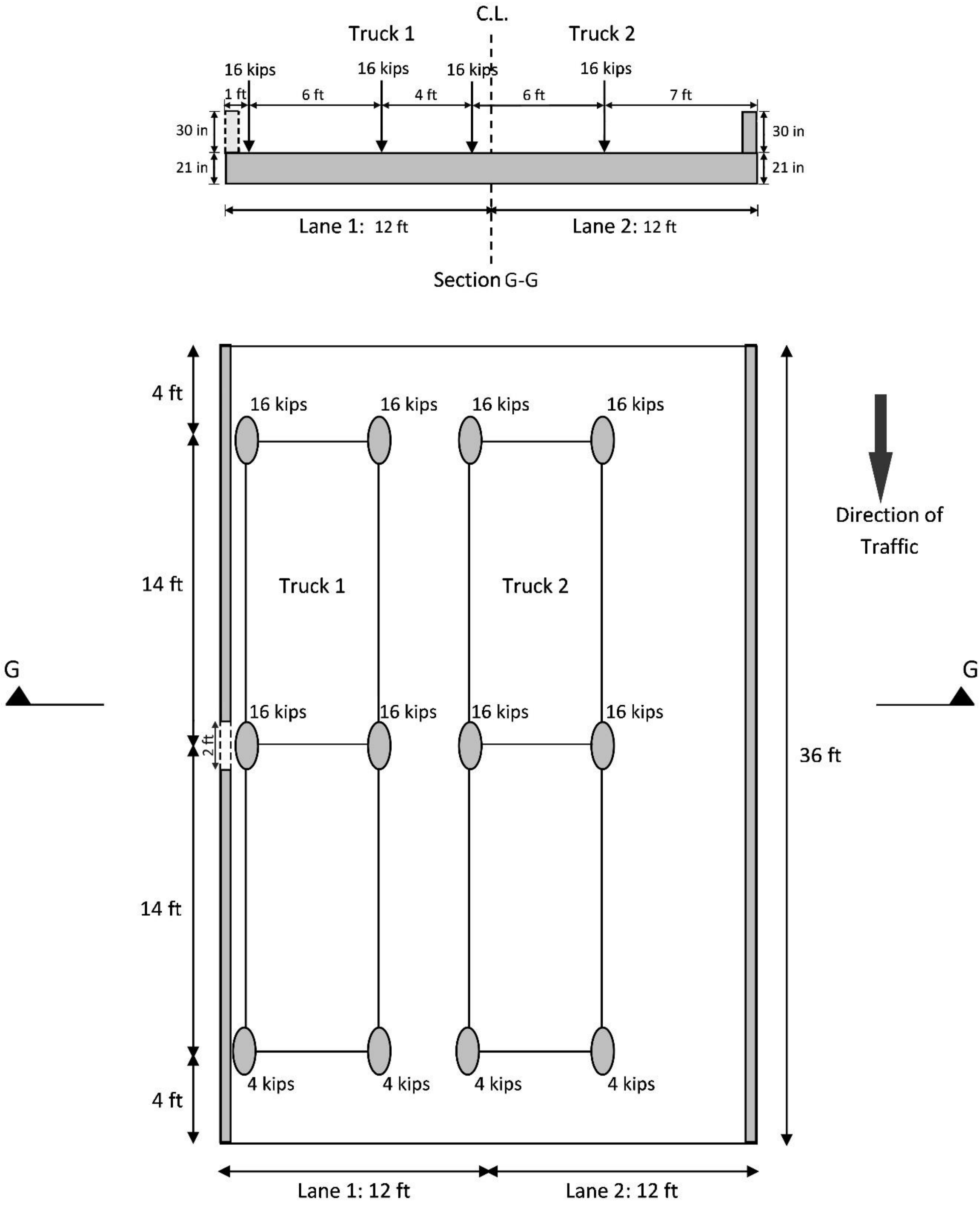


Figure 3.11: Typical Cross-Section and Plan of a Two-Lane 36 ft Span Straight Bridge with Two Railings with full-deterioration width of 2 ft under Edge Loading Condition E1.

3.3 Finite Element Modeling and Analysis

The finite element method is used to investigate the effect of railing deterioration width and depth on a one-span simply supported one and two lanes concrete slab bridges. Using SAP2000 (2012), the bridge slab is discretized into a convenient number of square four-node shell elements with six degrees of freedom per node, capable of simulating the membrane and plate-bending behavior. All elements are assumed to be linear elastic and the analysis assumed small deformations and deflections, and shear deformation was neglected. The selection of shell elements dimensions was based on the previous study by Mabsout et al. (2004) on simply supported concrete slab bridges which investigated the appropriate mesh discretization. A comparison was made on 0.5ft x 0.5ft, 1ft x 1ft and 2ft x 2ft elements, and the results obtained were nearly identical for the three cases. Thus, the 1ft x 1ft element size was adopted as sufficient for the bridge cases modeling. This mesh is also convenient for placing truck loads at 1 ft intervals to investigate maximum moments.

Railings were modeled as shell elements since the frame elements modeling used by Fawaz et al. (2016) is no more appropriate for modeling deteriorated railings.

The deteriorated portion of railing is modeled by assigning the corresponding shell element a zero stiffness in the case of broken railing.

The supports of the one-span simply supported bridges, were modeled by assigning the left pier a hinge support and the right one a roller. The concentrated wheel loads of the HS-20 truck are applied to nodes to produce the maximum bending moment.

Longitudinal bending moments and deflections are reported and investigated in this study. SAP2000 generates the finite element models and contour plots of bending moments and deflections.

The geometry, loading, deflection diagram and longitudinal moment contours for a typical 36 ft length, two-lane Bridge having two railings with one railing fully deteriorated 2 ft at center are presented in Figures 3.12(a), 3.12(b) and 3.12(c).

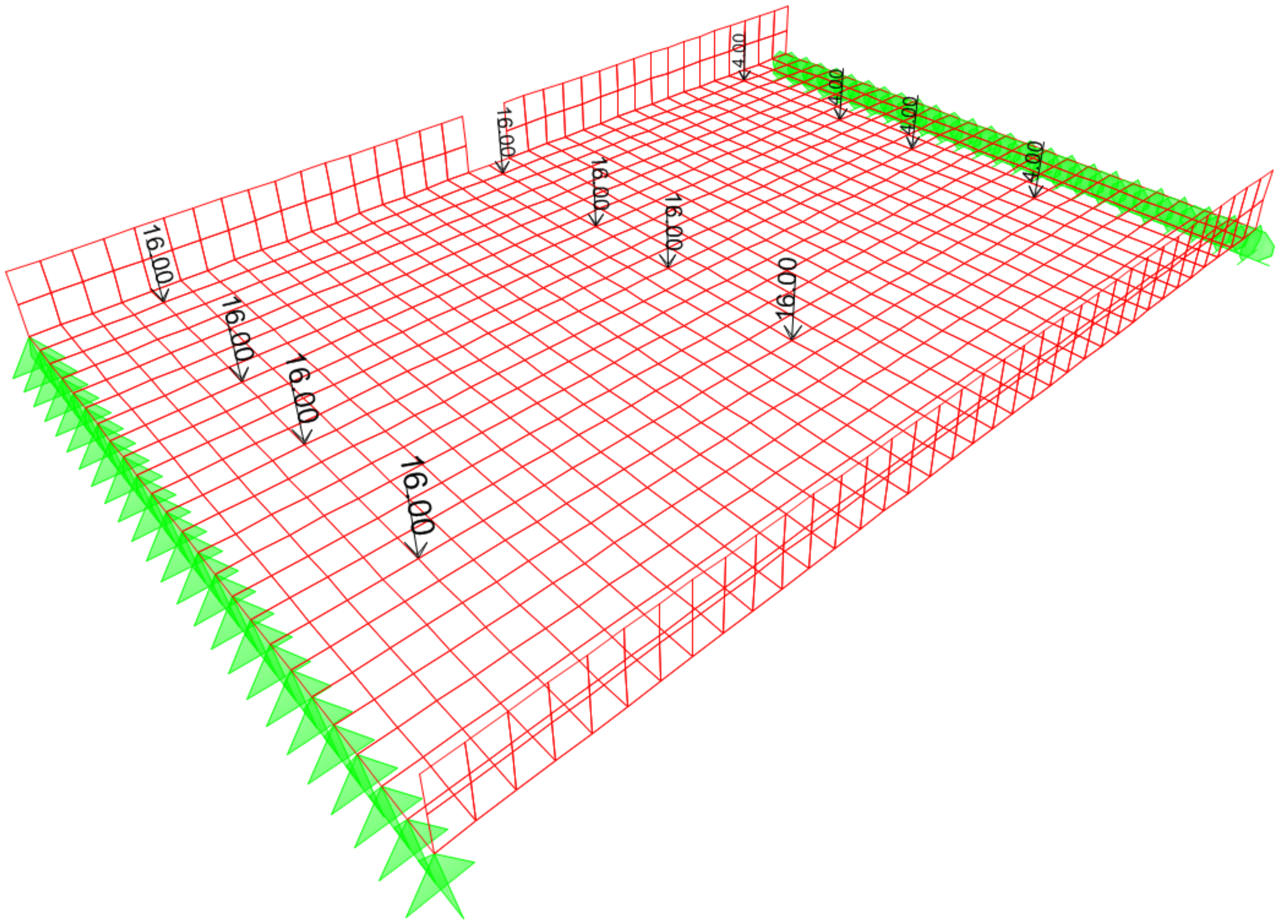


Figure 3.12(a): Finite Element Model for a Two-Lane, 36 ft Span Bridge, Two Railings - Geometry and Loading**.**

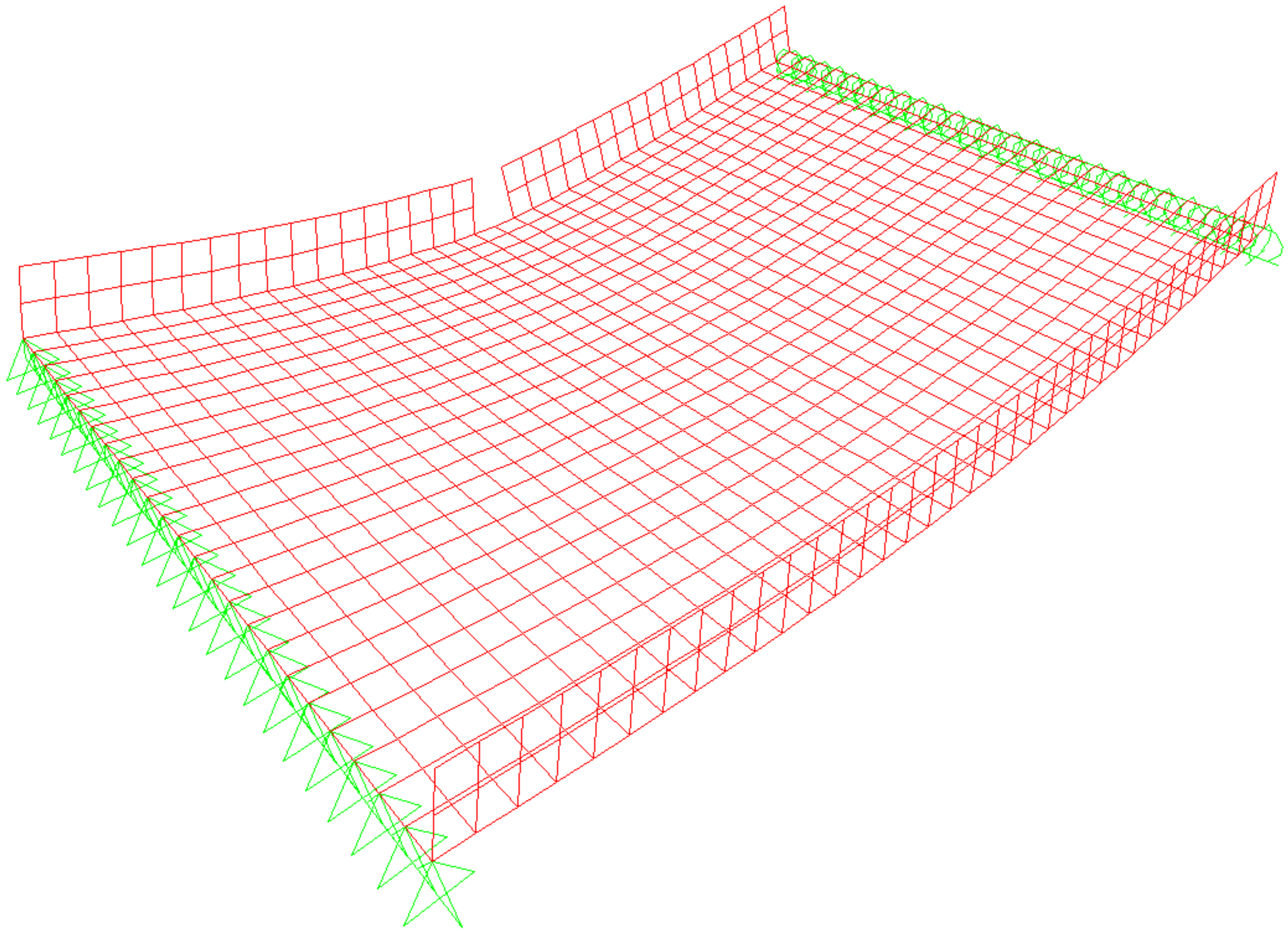


Figure 3.12(b): Finite Element Model for a Two-Lane, 36 ft Span Bridge, Two Railings, 2ft Fully Deteriorated Railing - Deformed Shape.

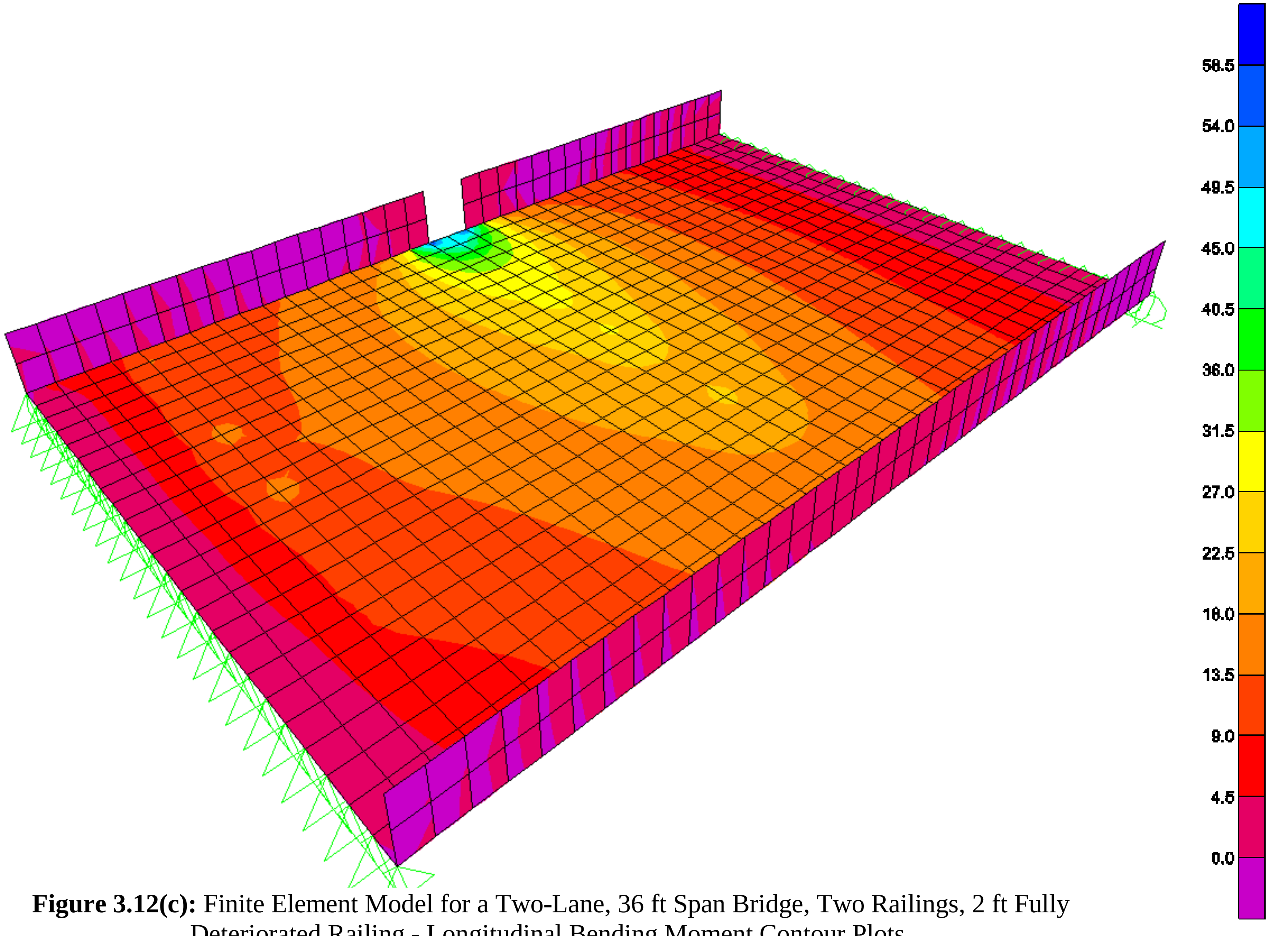


Figure 3.12(c): Finite Element Model for a Two-Lane, 36 ft Span Bridge, Two Railings, 2 ft Fully Deteriorated Railing - Longitudinal Bending Moment Contour Plots.

3.4 Summary

A total number of 112 bridge cases are analyzed based on the variation of the geometric parameters, loading distribution and railings presence and deterioration.

Two different span sizes were adopted with a total number of two span widths. Five widths and two depths of railing deterioration are considered.

The cases of bridges with no railings and bridges with non-deteriorated railing will serve as reference bridges in order to investigate the influence of deterioration of railings on straight concrete slab bridges.

CHAPTER 4

ANALYSIS RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the finite element results of the parametric study of the bridge cases described in Chapter 3. SAP2000 software is used to analyze the various bridge cases and load configurations. The results of the analyzed bridge cases are presented in tables and graphs and then summarized and compared with the reference bridge cases and with the AASHTO design procedure.

The analyzed bridge cases are divided into 7 categories as follows:

➢ **Category R0-Ref: Reference Bridge Cases with No Railing**

- Span= 36 ft - 1 Lane - No Railing - E1
- Span= 36 ft - 2 Lanes - No Railing - E1
- Span= 54 ft - 1 Lane - No Railing - E1
- Span= 54 ft - 2 Lanes - No Railing - E1

➢ **Category R1-Ref: Reference Bridge Cases with 1 Railing - Without Deterioration**

- Span= 36 ft - 1 Lane - One Railing - E2 - Without Deterioration
- Span= 36 ft - 2 Lanes - One Railing - E2 - Without Deterioration
- Span= 54 ft - 1 Lane - One Railing - E2 - Without Deterioration
- Span= 54 ft - 2 Lanes - One Railing - E2 - Without Deterioration

- **Category R2-Ref: Reference Bridge Cases with 2 Railings - Without Deterioration**
  - Span= 36 ft - 1 Lane - Two Railings - E1 - Without Deterioration
  - Span= 36 ft - 2 Lanes - Two Railings - E1 - Without Deterioration
  - Span= 54 ft - 1 Lane - Two Railings - E1 - Without Deterioration
  - Span= 54 ft - 2 Lanes - Two Railings - E1 - Without Deterioration
- **Category R1-Full: Bridges with 1 Railing & Full Depth Deterioration**
  - Span= 36 ft - 1 Lane - One Railing - E1 - Full Depth Deterioration
  - Span= 36 ft - 1 Lane - One Railing - E2 - Full Depth Deterioration
  - Span= 36 ft - 2 Lanes - One Railing - E1 - Full Depth Deterioration
  - Span= 36 ft - 2 Lanes - One Railing - E2 - Full Depth Deterioration
  - Span= 54 ft - 1 Lane - One Railing - E1 - Full Depth Deterioration
  - Span= 54 ft - 1 Lane - One Railing - E2 - Full Depth Deterioration
  - Span= 54 ft - 2 Lanes - One Railing - E1 - Full Depth Deterioration
  - Span= 54 ft - 2 Lanes - One Railing - E2 - Full Depth Deterioration
- **Category R1-Half: Bridges with 1 Railing & Half Depth Deterioration**
  - Span= 36 ft - 1 Lane - One Railing - E1 - Half Depth Deterioration
  - Span= 36 ft - 1 Lane - One Railing - E2 - Half Depth Deterioration
  - Span= 36 ft - 2 Lanes - One Railing - E1 - Half Depth Deterioration
  - Span= 36 ft - 2 Lanes - One Railing - E2 - Half Depth Deterioration
  - Span= 54 ft - 1 Lane - One Railing - E1 - Half Depth Deterioration
  - Span= 54 ft - 1 Lane - One Railing - E2 - Half Depth Deterioration
  - Span= 54 ft - 2 Lanes - One Railing - E1 - Half Depth Deterioration

- Span= 54 ft - 2 Lanes - One Railing - E2 - Half Depth Deterioration

➢ **Category R2-Full: Bridges with 2 Railings & Full Depth Deterioration**

- Span= 36 ft - 1 Lane - Two Railings - E1 - Full Depth Deterioration
- Span= 36 ft - 2 Lanes - Two Railings - E1 - Full Depth Deterioration
- Span= 54 ft - 1 Lane - Two Railings - E1 - Full Depth Deterioration
- Span= 54 ft - 2 Lanes - Two Railings - E1 - Full Depth Deterioration

➢ **Category R2-Half: Bridges with 2 Railings & Half Depth Deterioration**

- Span= 36 ft - 1 Lane - Two Railings - E1 - Half Depth Deterioration
- Span= 36 ft - 2 Lanes - Two Railings - E1 - Half Depth Deterioration
- Span= 54 ft - 1 Lane - Two Railings - E1 - Half Depth Deterioration
- Span= 54 ft - 2 Lanes - Two Railings - E1 - Half Depth Deterioration

4.2 Presentation of Results

The FEA results evaluated consist of the maximum longitudinal bending moments, edge beam moments, and maximum live load deflections at critical locations of the bridge slabs.

The maximum longitudinal bending moment in the slab is defined as the first peak value after the left edge peak moment. The maximum peak moment at the edge is resisted by an edge beam.

The edge beam moment is defined as the maximum moment at the slab edge along the critical cross-section. For the edge beam moment, the critical cross-section may be along the centerline of the span or along the extents of deterioration.

It should also be noted that the edge beam moment is taken to be the larger at the two edges in the absence of railings. In the cases of bridges with one railing the edge beam moment is taken to be at the edge without railing (before deterioration) and to the railing edge (after deterioration). In the cases of bridges with two railings, no edge beam is provided and the edge moment is mostly carried by the railings (before deterioration). The edge beam moments presented in the tables are the moments carried by the slab edges regardless of the existence of a beam at the edge.

The maximum live load deflections from FEA for all the cases are obtained and compared to the AASHTO criterion of S/800. It is worth noting that the FEA is an elastic analysis, and not the actual cracked section analysis, which would yield higher deflection values.

The FEA longitudinal bending moments and deflections are extracted from SAP2000 output files for each of the bridge cases, with different railing deterioration widths, and their corresponding bridge cases with no railings.

The AASHTO moments are computed using Eqs. (1) and (2) for the Standard Specifications, and Eq. (5) for LRFD.

The FEA longitudinal bending moments per unit foot along the critical cross-section for all bridge cases and their reference bridge cases are tabulated and plotted along with AASHTO specs and LRFD moments in Tables 4.1a to 4.24a and Figures 4.1 to 4.24, respectively. The FEA results; maximum longitudinal bending moments, edge beam moments, and maximum live load deflections for all bridge cases are tabulated as shown in Tables 4.1b to 4.24b.

The FEA results for each bridge category are summarized in tables along with the corresponding AASHTO results and reference cases results for comparison purposes. The FEA results are first compared to AASHTO Specs and AASHTO LRFD results in terms of percentage difference and tabulated in Tables 4.25 to 4.44 (Refer to Section 4.3.1). The FEA results are then compared to the reference cases of bridges with no railing, and bridges with non-deteriorated railing in terms of ratios and tabulated in Tables 4.45 to 4.68 (Refer to Sections 4.3.2 and 4.3.3).

4.2.1 FEA Longitudinal Moments and Deflections for Categories "R1-Full", "R1-Ref", and "R0-Ref" with AASHTO Moments

- **Span= 36 ft; One Lane; One Railing; E1; Full Deterioration**
- **Span= 36 ft; One Lane; One Railing; E2; Full Deterioration**
- **Span= 36 ft; Two Lane; One Railing; E1; Full Deterioration**
- **Span= 36 ft; Two Lane; One Railing; E2; Full Deterioration**
- **Span= 54 ft; One Lane; One Railing; E1; Full Deterioration**
- **Span= 54 ft; One Lane; One Railing; E2; Full Deterioration**
- **Span= 54 ft; Two Lane; One Railing; E1; Full Deterioration**
- **Span= 54 ft; Two Lane; One Railing; E2; Full Deterioration**

- **Span= 36 ft; One Lane; One Railing; E1; Full Deterioration**

Table 4.1a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	31.4	20.0	50.7	39.4	36.1	33.0	32.4	47.2
1	32.7	22.3	42.7	41.7	37.0	34.3	32.4	47.2
2	29.1	19.5	33.4	34.0	32.6	30.5	32.4	47.2
3	27.5	18.5	29.0	29.8	29.7	28.6	32.4	47.2
4	26.7	18.3	26.5	27.2	27.8	27.5	32.4	47.2
5	26.4	18.4	25.1	25.8	26.6	26.8	32.4	47.2
6	26.9	19.2	24.8	25.5	26.3	26.9	32.4	47.2
7	29.1	21.7	26.6	27.2	28.0	28.8	32.4	47.2
8	25.9	18.8	23.2	23.7	24.5	25.4	32.4	47.2
9	24.5	17.6	21.6	22.1	22.9	23.8	32.4	47.2
10	23.7	17.0	20.7	21.1	21.9	22.9	32.4	47.2
11	23.2	16.6	20.0	20.5	21.2	22.2	32.4	47.2
12	22.8	16.3	19.6	20.0	20.7	21.7	32.4	47.2
13	22.5	16.2	19.3	19.8	20.4	21.4	32.4	47.2
14	22.4	16.1	19.2	19.6	20.3	21.2	32.4	47.2

Table 4.1b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	32.7	22.3	50.7	41.7	37.0	34.3
	At Deterioration	32.7	22.3	50.7	47.0	41.6	36.2
Maximum Longitudinal Moment (Kip-ft/ft)		29.1	21.7	26.6	27.2	28.0	28.8
Maximum Live Load Deflection (in)		0.180	0.115	0.147	0.151	0.157	0.165

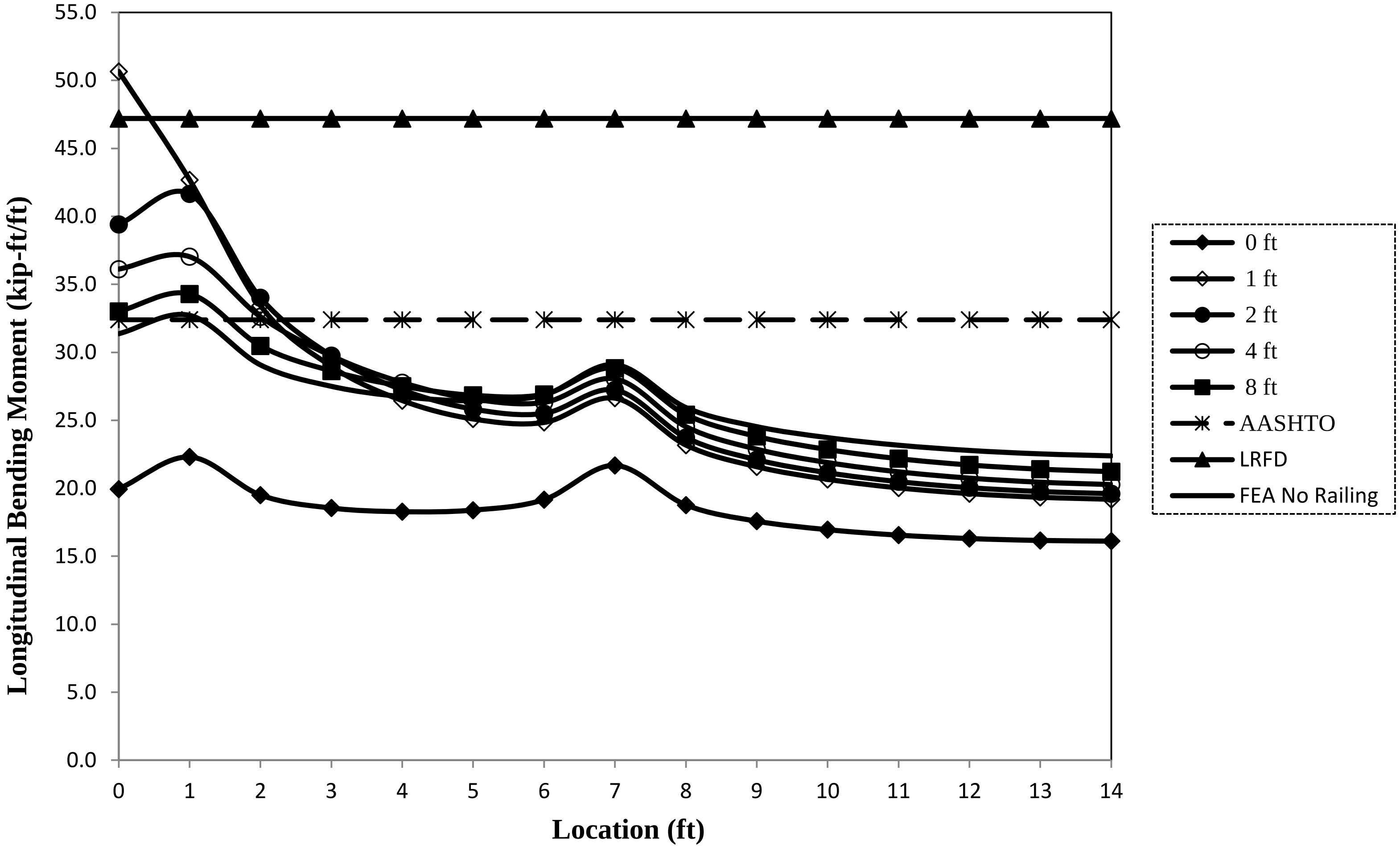


Figure 4.1: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

- **Span= 36 ft; One Lane; One Railing; E2; Full Deterioration**

Table 4.2a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	31.4	26.1	28.6	28.9	29.5	30.3	32.4	47.2
1	32.7	27.4	29.9	30.3	30.9	31.7	32.4	47.2
2	29.1	23.6	26.3	26.7	27.3	28.1	32.4	47.2
3	27.5	21.9	24.8	25.1	25.8	26.6	32.4	47.2
4	26.7	21.1	24.1	24.5	25.1	26.0	32.4	47.2
5	26.4	20.6	23.9	24.3	25.0	25.8	32.4	47.2
6	26.9	20.9	24.5	24.9	25.6	26.4	32.4	47.2
7	29.1	23.0	27.0	27.5	28.2	28.9	32.4	47.2
8	25.9	19.6	24.2	24.7	25.4	26.0	32.4	47.2
9	24.5	18.0	23.4	24.0	24.6	24.9	32.4	47.2
10	23.7	16.9	23.5	24.2	24.6	24.4	32.4	47.2
11	23.2	16.1	24.5	25.2	25.2	24.2	32.4	47.2
12	22.8	15.4	26.6	27.1	25.9	24.0	32.4	47.2
13	22.5	14.8	31.2	30.3	26.4	23.9	32.4	47.2
14	22.4	14.3	38.8	29.4	26.6	23.8	32.4	47.2

Table 4.2b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	32.7	27.4	38.8	30.3	30.9	31.7
	At Deterioration	32.7	27.4	39.1	37.4	35.0	31.9
Maximum Longitudinal Moment (Kip-ft/ft)		29.1	23.0	27.0	27.5	28.2	28.9
Maximum Live Load Deflection (in)		0.180	0.143	0.158	0.160	0.163	0.169

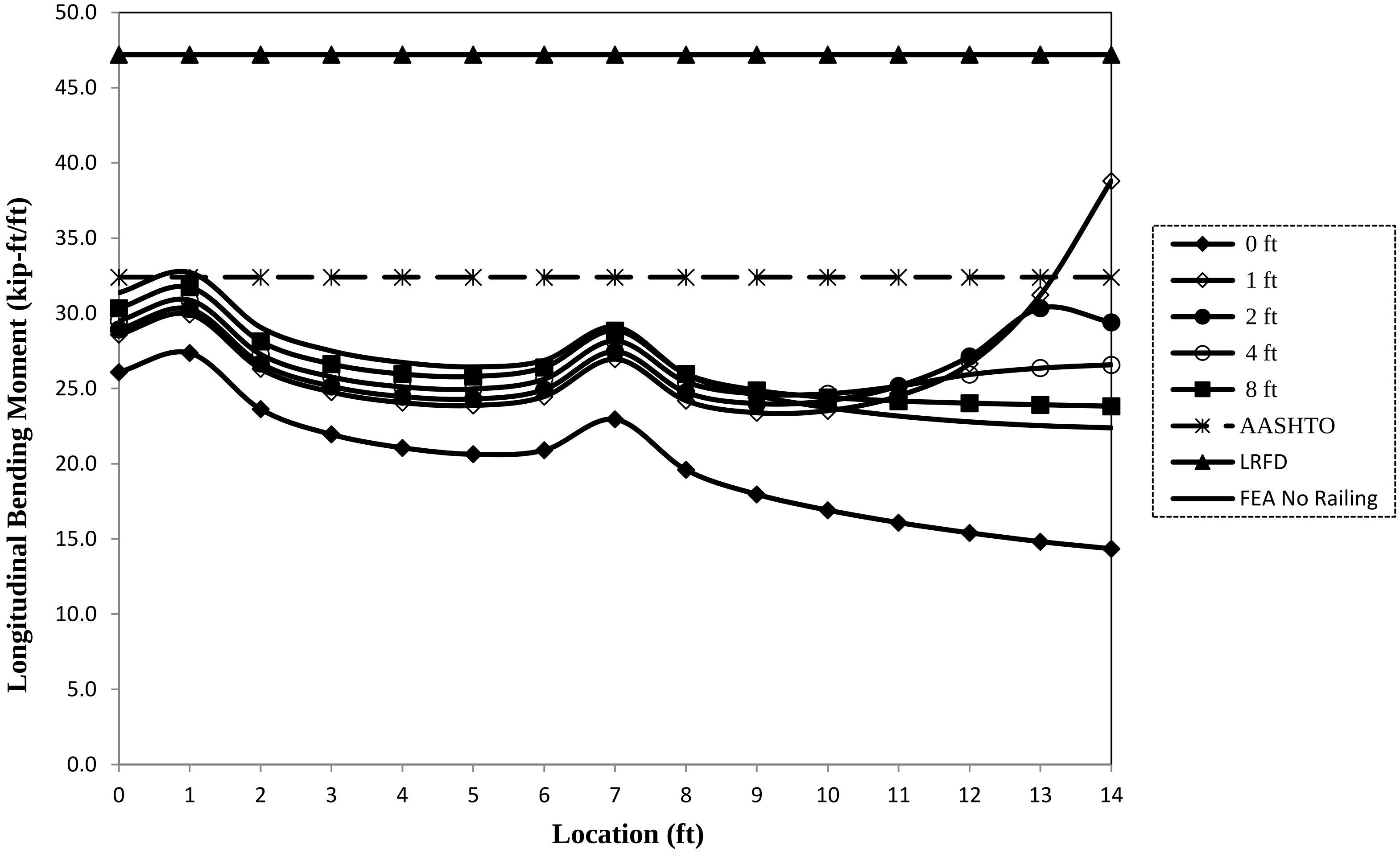


Figure 4.2: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration.

- **Span= 36 ft; Two Lane; One Railing; E1; Full Deterioration**

Table 4.3a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	37.2	25.0	63.9	49.1	44.5	40.0	32.4	45.6
1	38.5	27.5	53.1	51.5	45.2	41.2	32.4	45.6
2	34.8	24.8	42.1	42.7	40.5	37.3	32.4	45.6
3	33.3	24.0	36.8	37.6	37.3	35.4	32.4	45.6
4	32.6	24.0	33.8	34.6	35.0	34.2	32.4	45.6
5	32.3	24.3	32.1	32.9	33.6	33.5	32.4	45.6
6	32.9	25.3	31.8	32.5	33.3	33.6	32.4	45.6
7	35.3	28.2	33.7	34.3	35.1	35.7	32.4	45.6
8	32.5	25.8	30.5	31.0	31.8	32.5	32.4	45.6
9	31.6	25.2	29.3	29.8	30.6	31.3	32.4	45.6
10	31.8	25.7	29.4	29.9	30.5	31.3	32.4	45.6
11	33.9	28.2	31.5	31.9	32.5	33.3	32.4	45.6
12	30.8	25.3	28.3	28.7	29.2	30.0	32.4	45.6
13	29.4	24.2	26.9	27.3	27.9	28.6	32.4	45.6
14	28.8	23.8	26.4	26.7	27.2	27.9	32.4	45.6
15	28.6	23.8	26.2	26.5	27.0	27.7	32.4	45.6
16	29.1	24.5	26.7	27.0	27.5	28.2	32.4	45.6
17	31.4	26.9	29.1	29.3	29.8	30.4	32.4	45.6
18	28.3	24.0	26.0	26.2	26.7	27.3	32.4	45.6
19	26.9	22.8	24.7	24.9	25.3	26.0	32.4	45.6
20	26.1	22.1	23.9	24.2	24.6	25.2	32.4	45.6
21	25.6	21.7	23.5	23.7	24.1	24.7	32.4	45.6
22	25.3	21.4	23.2	23.4	23.8	24.3	32.4	45.6
23	25.1	21.3	23.0	23.2	23.6	24.1	32.4	45.6
24	25.0	21.3	22.9	23.1	23.5	24.0	32.4	45.6

Table 4.3b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	38.5	27.5	63.9	51.5	45.2	41.2
	At Deterioration	38.5	27.5	63.9	59.5	52.9	45.9
Maximum Longitudinal Moment (Kip-ft/ft)		35.3	28.2	33.7	34.3	35.1	35.7
Maximum Live Load Deflection (in)		0.219	0.149	0.187	0.191	0.197	0.206

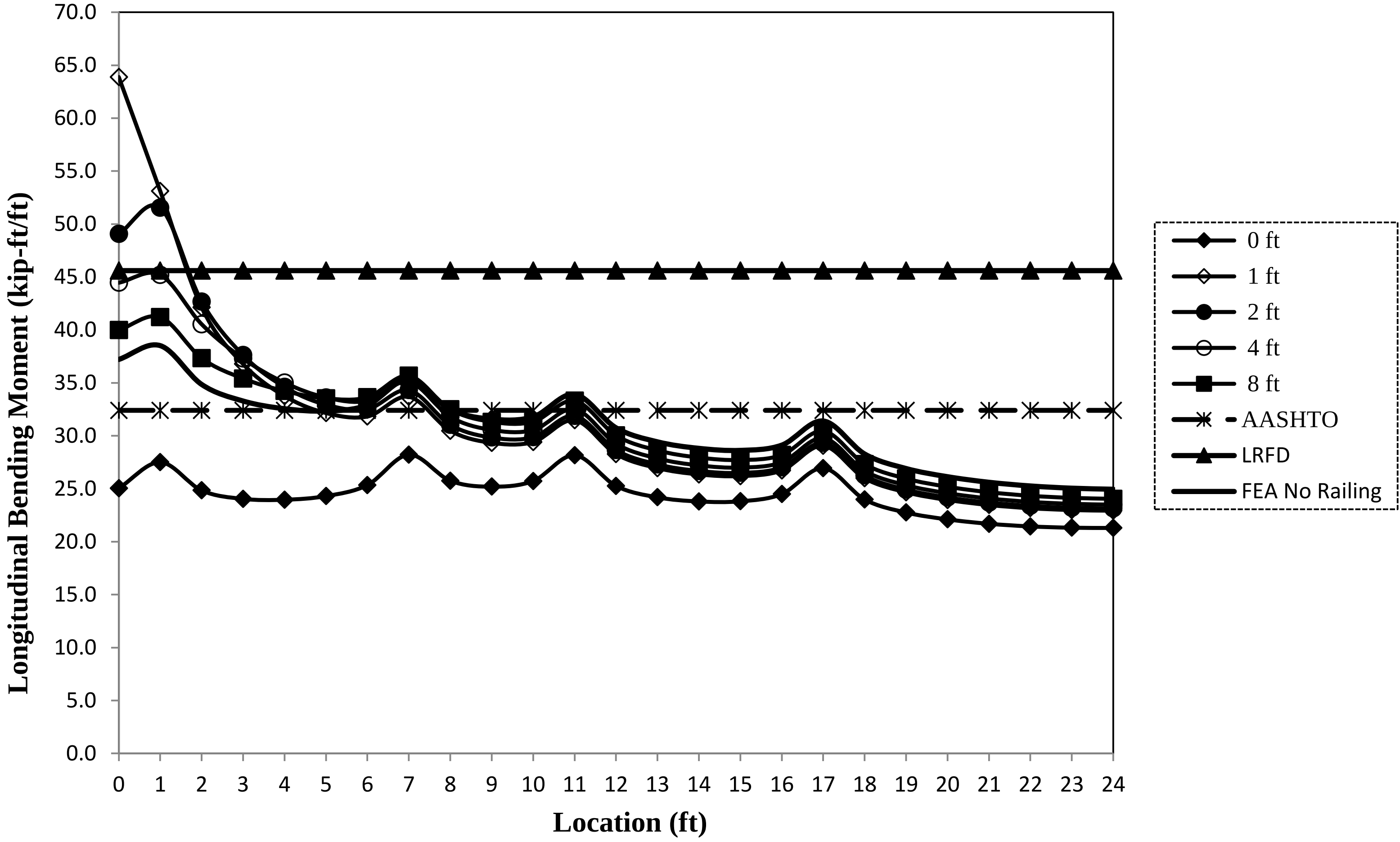


Figure 4.3: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration.

- **Span= 36 ft; Two Lane; One Railing; E2; Full Deterioration**

Table 4.4a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	37.2	34.4	35.6	35.8	36.1	36.5	32.4	45.6
1	38.5	35.6	36.9	37.1	37.3	37.8	32.4	45.6
2	34.8	31.9	33.2	33.4	33.7	34.1	32.4	45.6
3	33.3	30.3	31.6	31.8	32.1	32.5	32.4	45.6
4	32.6	29.4	30.8	31.0	31.3	31.8	32.4	45.6
5	32.3	29.1	30.6	30.8	31.1	31.6	32.4	45.6
6	32.9	29.6	31.1	31.3	31.6	32.1	32.4	45.6
7	35.3	31.9	33.5	33.7	34.0	34.6	32.4	45.6
8	32.5	28.9	30.6	30.8	31.2	31.7	32.4	45.6
9	31.6	27.9	29.7	29.9	30.3	30.9	32.4	45.6
10	31.8	27.9	29.8	30.1	30.5	31.1	32.4	45.6
11	33.9	29.9	32.0	32.2	32.7	33.3	32.4	45.6
12	30.8	26.5	28.8	29.1	29.5	30.2	32.4	45.6
13	29.4	25.0	27.5	27.8	28.3	28.9	32.4	45.6
14	28.8	24.2	26.9	27.3	27.8	28.4	32.4	45.6
15	28.6	23.8	26.8	27.2	27.8	28.4	32.4	45.6
16	29.1	24.0	27.5	28.0	28.5	29.1	32.4	45.6
17	31.4	26.0	30.1	30.6	31.2	31.7	32.4	45.6
18	28.3	22.6	27.4	28.0	28.6	28.9	32.4	45.6
19	26.9	20.9	26.8	27.4	27.9	27.8	32.4	45.6
20	26.1	19.8	27.1	27.8	28.1	27.5	32.4	45.6
21	25.6	18.9	28.4	29.1	28.8	27.3	32.4	45.6
22	25.3	18.2	31.1	31.5	29.8	27.2	32.4	45.6
23	25.1	17.5	36.6	35.4	30.4	27.2	32.4	45.6
24	25.0	17.0	45.7	34.3	30.7	27.1	32.4	45.6

Table 4.4b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	38.5	35.6	45.7	37.1	37.3	37.8
	At Deterioration	38.5	35.6	46.1	43.7	40.6	36.4
Maximum Longitudinal Moment (Kip-ft/ft)		35.3	31.9	33.5	33.7	34.0	34.6
Maximum Live Load Deflection (in)		0.219	0.199	0.207	0.208	0.210	0.213

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	38.5	35.6	45.7	37.1	37.3	37.8
	At Deterioration	38.5	35.6	46.1	43.7	40.6	36.4
Maximum Longitudinal Moment (Kip-ft/ft)		35.3	31.9	33.5	33.7	34.0	34.6
Maximum Live Load Deflection (in)		0.219	0.199	0.207	0.208	0.210	0.213

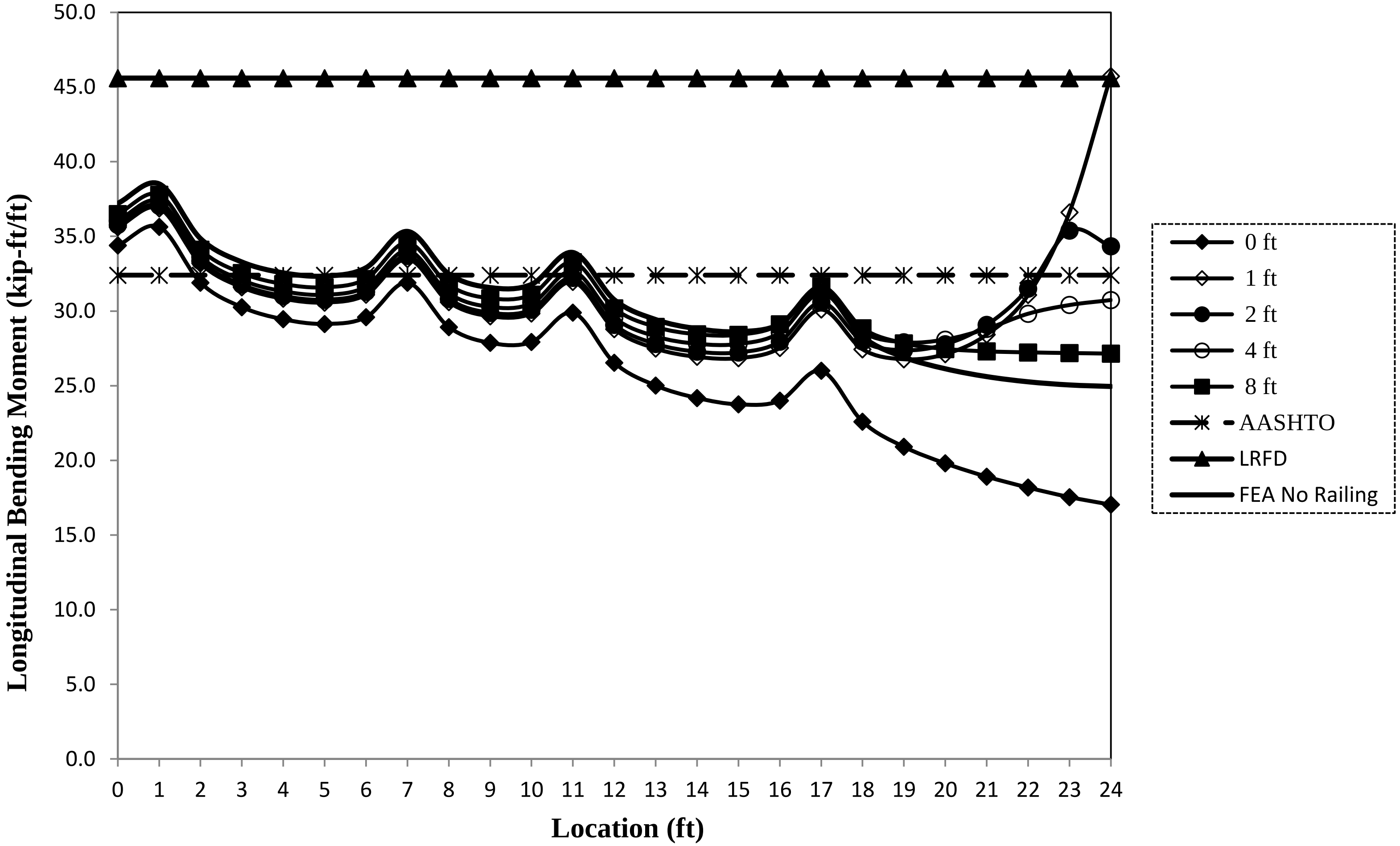


Figure 4.4: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

- **Span= 54 ft; One Lane; One Railing; E1; Full Deterioration**

Table 4.5a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	54.5	43.4	77.4	64.9	61.0	57.1	50.2	75.3
1	55.9	45.3	68.6	67.3	61.8	58.3	50.2	75.3
2	52.2	42.2	58.3	58.9	57.2	54.4	50.2	75.3
3	50.6	41.0	53.1	54.0	53.9	52.5	50.2	75.3
4	49.9	40.5	50.0	50.9	51.5	51.2	50.2	75.3
5	49.6	40.5	48.3	49.1	50.0	50.3	50.2	75.3
6	50.0	41.1	47.7	48.4	49.4	50.2	50.2	75.3
7	52.2	43.5	49.2	49.9	50.9	51.9	50.2	75.3
8	49.1	40.5	45.6	46.2	47.2	48.3	50.2	75.3
9	47.7	39.2	43.8	44.4	45.3	46.6	50.2	75.3
10	46.9	38.5	42.8	43.3	44.2	45.5	50.2	75.3
11	46.3	38.1	42.0	42.6	43.4	44.7	50.2	75.3
12	45.9	37.8	41.5	42.0	42.9	44.2	50.2	75.3
13	45.7	37.6	41.2	41.7	42.5	43.8	50.2	75.3
14	45.5	37.5	41.0	41.5	42.3	43.5	50.2	75.3

Table 4.5b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	55.9	45.3	77.4	67.3	61.8	58.3
	At Deterioration	55.9	45.3	77.4	74.3	69.3	64.9
Maximum Longitudinal Moment (Kip-ft/ft)		52.2	43.5	49.2	49.9	50.9	51.9
Maximum Live Load Deflection (in)		0.352	0.282	0.306	0.309	0.314	0.323

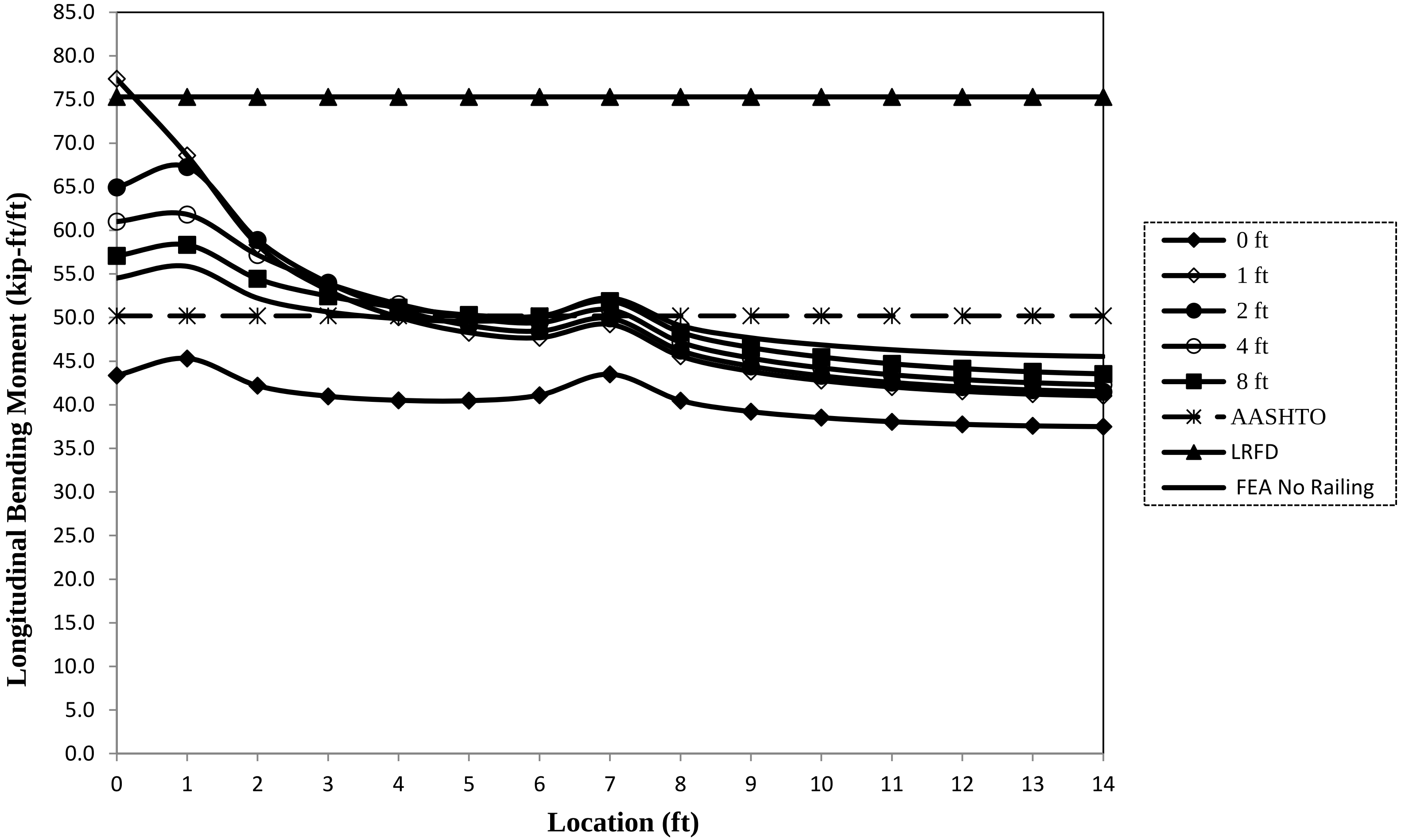


Figure 4.5: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

- **Span= 54 ft; One Lane; One Railing; E2; Full Deterioration**

Table 4.6a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	54.5	47.1	50.3	50.7	51.5	52.7	50.2	75.3
1	55.9	48.4	51.7	52.1	52.9	54.1	50.2	75.3
2	52.2	44.7	48.1	48.6	49.4	50.6	50.2	75.3
3	50.6	43.1	46.6	47.1	47.9	49.1	50.2	75.3
4	49.9	42.2	46.0	46.5	47.4	48.6	50.2	75.3
5	49.6	41.8	46.0	46.5	47.4	48.5	50.2	75.3
6	50.0	42.2	46.7	47.3	48.2	49.3	50.2	75.3
7	52.2	44.3	49.4	50.1	51.0	51.9	50.2	75.3
8	49.1	41.0	46.9	47.6	48.5	49.2	50.2	75.3
9	47.7	39.5	46.5	47.2	48.0	48.3	50.2	75.3
10	46.9	38.5	47.1	47.9	48.4	48.1	50.2	75.3
11	46.3	37.8	48.7	49.5	49.4	48.0	50.2	75.3
12	45.9	37.2	51.7	52.2	50.6	48.0	50.2	75.3
13	45.7	36.8	57.6	56.4	51.3	48.0	50.2	75.3
14	45.5	36.4	66.7	55.3	51.6	47.9	50.2	75.3

Table 4.6b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	55.9	48.4	66.7	55.3	52.9	54.1
	At Deterioration	55.9	48.4	67.6	65.5	63.3	61.1
Maximum Longitudinal Moment (Kip-ft/ft)		52.2	44.3	49.4	50.1	51.0	51.9
Maximum Live Load Deflection (in)		0.352	0.298	0.313	0.316	0.319	0.327

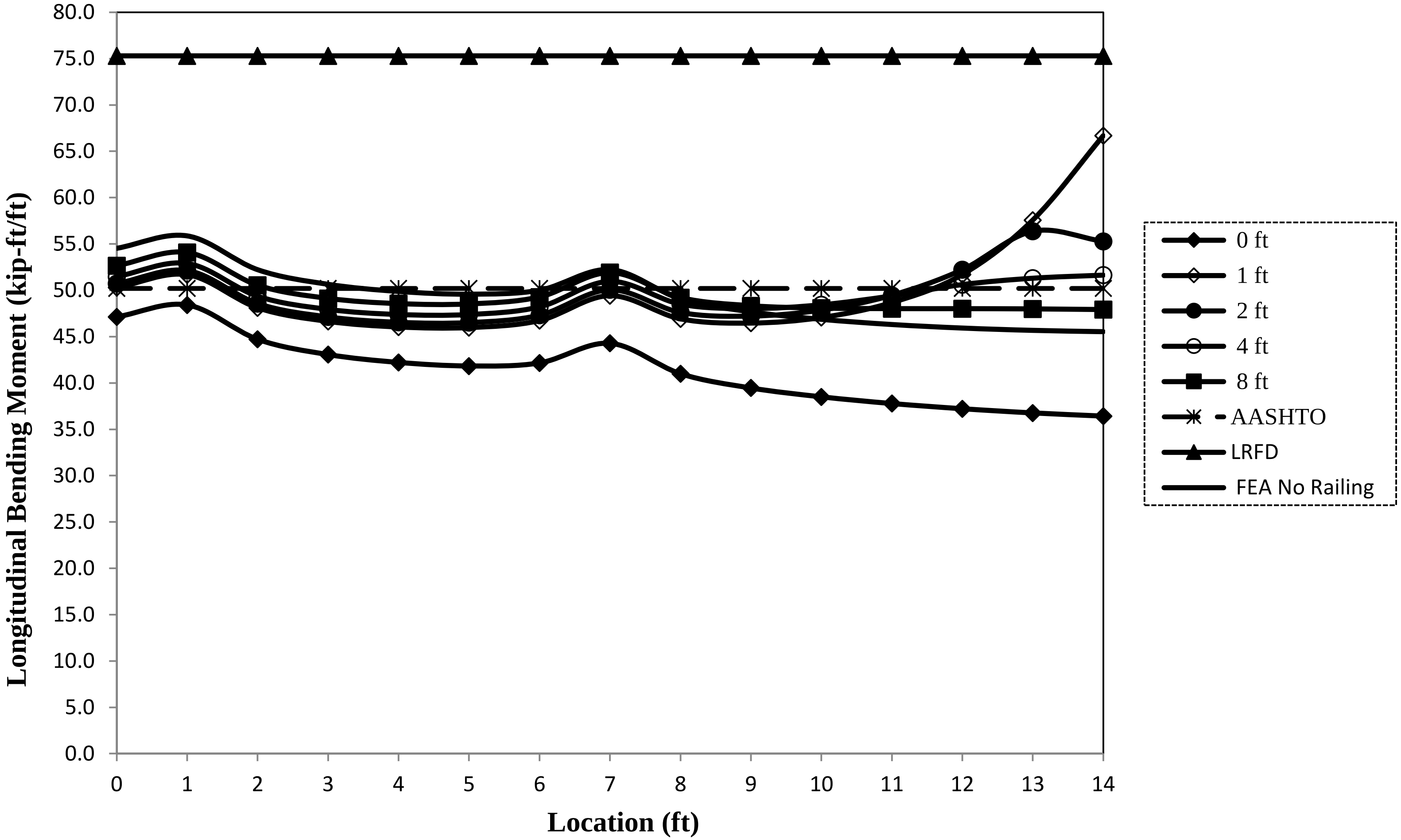


Figure 4.6: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

- **Span= 54 ft; Two Lane; One Railing; E1; Full Deterioration**

Table 4.7a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	64.5	54.0	96.5	80.3	74.9	69.3	50.2	81.7
1	65.8	56.0	84.8	82.9	75.5	70.4	50.2	81.7
2	62.1	52.8	72.6	73.1	70.5	66.4	50.2	81.7
3	60.5	51.7	66.3	67.2	66.7	64.2	50.2	81.7
4	59.7	51.3	62.5	63.4	63.9	62.8	50.2	81.7
5	59.4	51.4	60.3	61.2	61.9	61.8	50.2	81.7
6	60.0	52.2	59.5	60.3	61.2	61.6	50.2	81.7
7	62.3	54.8	61.0	61.7	62.6	63.3	50.2	81.7
8	59.5	52.2	57.5	58.1	59.0	59.8	50.2	81.7
9	58.5	51.4	56.1	56.6	57.5	58.4	50.2	81.7
10	58.7	51.8	55.9	56.4	57.2	58.2	50.2	81.7
11	60.8	54.1	57.8	58.3	59.0	60.0	50.2	81.7
12	57.7	51.1	54.4	54.9	55.6	56.6	50.2	81.7
13	56.3	49.9	53.0	53.4	54.0	55.0	50.2	81.7
14	55.7	49.4	52.3	52.6	53.3	54.2	50.2	81.7
15	55.5	49.4	52.0	52.4	53.0	53.9	50.2	81.7
16	56.0	50.0	52.5	52.8	53.4	54.3	50.2	81.7
17	58.3	52.4	54.7	55.0	55.6	56.5	50.2	81.7
18	55.2	49.4	51.6	51.9	52.4	53.3	50.2	81.7
19	53.8	48.1	50.3	50.6	51.1	51.9	50.2	81.7
20	53.1	47.4	49.5	49.8	50.3	51.1	50.2	81.7
21	52.6	47.0	49.0	49.3	49.8	50.6	50.2	81.7
22	52.3	46.8	48.7	49.0	49.4	50.2	50.2	81.7
23	52.1	46.6	48.5	48.8	49.3	50.0	50.2	81.7
24	52.0	46.6	48.5	48.7	49.2	49.9	50.2	81.7

Table 4.7b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	65.8	56.0	96.5	82.9	75.5	70.4
	At Deterioration	65.8	56.0	96.5	92.9	86.9	81.2
Maximum Longitudinal Moment (Kip-ft/ft)		62.3	54.8	61.0	61.7	62.6	63.3
Maximum Live Load Deflection (in)		0.423	0.357	0.383	0.386	0.391	0.399

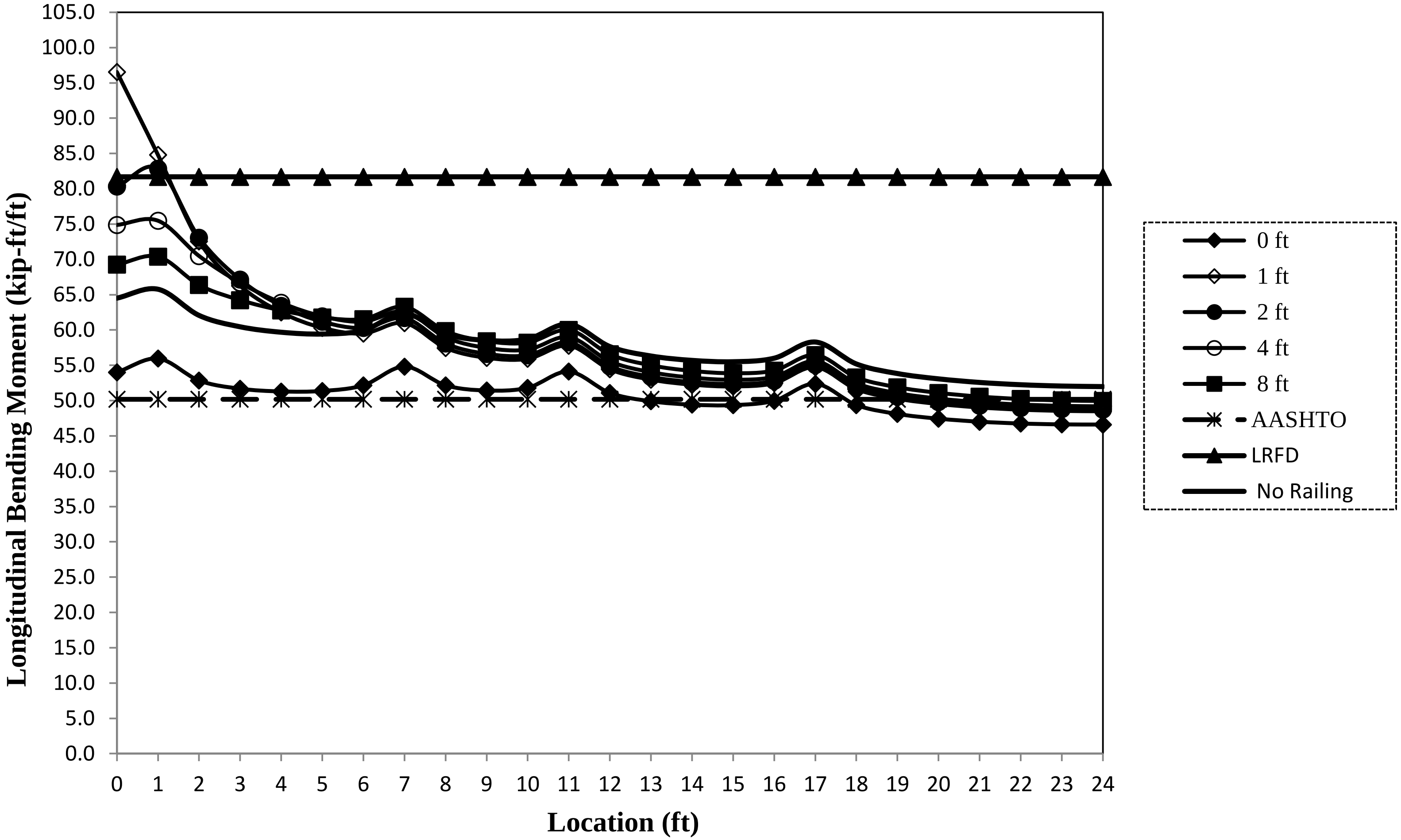


Figure 4.7: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Full Depth Railing Deterioration

- **Span= 54 ft; Two Lane; One Railing; E2; Full Deterioration**

Table 4.8a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	64.5	59.8	61.4	61.6	62.0	62.7	50.2	81.7
1	65.8	61.0	62.7	62.9	63.3	64.0	50.2	81.7
2	62.1	57.2	58.9	59.2	59.6	60.3	50.2	81.7
3	60.5	55.6	57.3	57.5	58.0	58.7	50.2	81.7
4	59.7	54.7	56.5	56.8	57.2	57.9	50.2	81.7
5	59.4	54.4	56.3	56.5	57.0	57.7	50.2	81.7
6	60.0	54.8	56.8	57.0	57.5	58.3	50.2	81.7
7	62.3	57.2	59.2	59.5	59.9	60.7	50.2	81.7
8	59.5	54.2	56.3	56.6	57.1	57.9	50.2	81.7
9	58.5	53.1	55.4	55.7	56.2	57.1	50.2	81.7
10	58.7	53.2	55.7	56.0	56.5	57.4	50.2	81.7
11	60.8	55.2	57.9	58.2	58.8	59.7	50.2	81.7
12	57.7	51.9	54.8	55.2	55.8	56.7	50.2	81.7
13	56.3	50.4	53.6	54.0	54.7	55.6	50.2	81.7
14	55.7	49.7	53.2	53.7	54.4	55.2	50.2	81.7
15	55.5	49.3	53.3	53.8	54.6	55.4	50.2	81.7
16	56.0	49.7	54.2	54.8	55.6	56.3	50.2	81.7
17	58.3	51.8	57.1	57.7	58.5	59.1	50.2	81.7
18	55.2	48.5	54.8	55.5	56.3	56.6	50.2	81.7
19	53.8	47.0	54.6	55.4	56.1	55.9	50.2	81.7
20	53.1	46.0	55.6	56.4	56.8	55.8	50.2	81.7
21	52.6	45.3	57.8	58.6	58.2	56.0	50.2	81.7
22	52.3	44.7	61.6	62.1	59.8	56.1	50.2	81.7
23	52.1	44.3	69.0	67.3	60.7	56.2	50.2	81.7
24	52.0	43.9	80.3	66.0	61.2	56.2	50.2	81.7

Table 4.8b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	65.8	61.0	80.3	66.0	63.3	64.0
	At Deterioration	65.8	61.0	81.3	78.5	75.4	72.1
Maximum Longitudinal Moment (Kip-ft/ft)		62.3	57.2	59.2	59.5	59.9	60.7
Maximum Live Load Deflection (in)		0.423	0.389	0.398	0.400	0.402	0.407

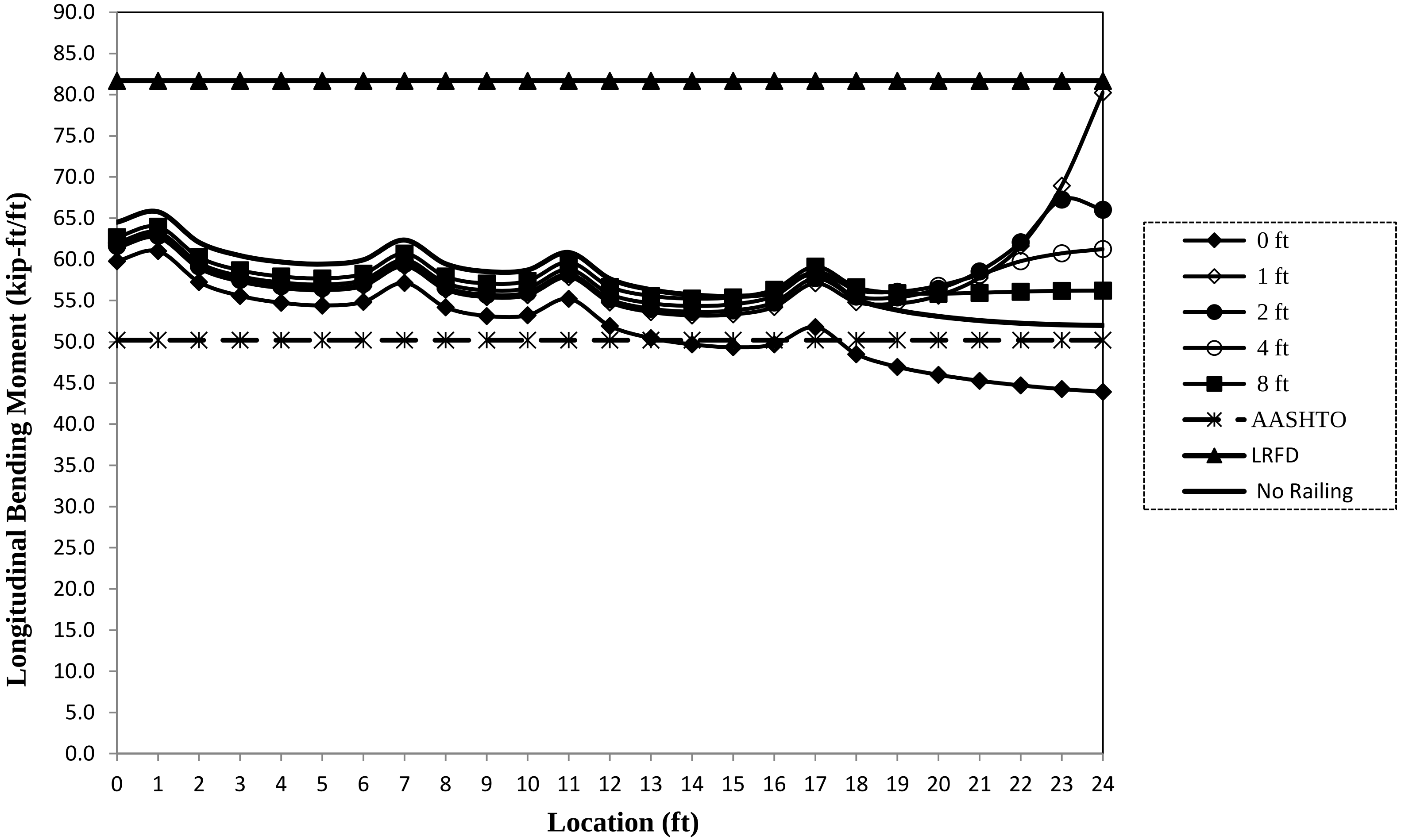


Figure 4.8: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Full Depth Railing Deterioration

4.2.2 *FEA Longitudinal Moments and Deflections for Categories "R1-Half", "R1-Ref", and "R0-Ref" with AASHTO Moments*

- **Span= 36 ft; One Lane; One Railing; E1; Half Deterioration**
- **Span= 36 ft; One Lane; One Railing; E2; Half Deterioration**
- **Span= 36 ft; Two Lane; One Railing; E1; Half Deterioration**
- **Span= 36 ft; Two Lane; One Railing; E2; Half Deterioration**
- **Span= 54 ft; One Lane; One Railing; E1; Half Deterioration**
- **Span= 54 ft; One Lane; One Railing; E2; Half Deterioration**
- **Span= 54 ft; Two Lane; One Railing; E1; Half Deterioration**
- **Span= 54 ft; Two Lane; One Railing; E2; Half Deterioration**

- **Span= 36 ft; One Lane; One Railing; E1; Half Deterioration**

Table 4.9a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	31.4	20.0	38.10	40.8	33.9	30.7	32.4	47.2
1	32.7	22.3	33.59	35.7	34.8	32.1	32.4	47.2
2	29.1	19.5	27.16	28.7	29.4	28.3	32.4	47.2
3	27.5	18.5	24.12	25.4	26.5	26.5	32.4	47.2
4	26.7	18.3	22.59	23.6	24.8	25.4	32.4	47.2
5	26.4	18.4	21.88	22.8	23.9	24.8	32.4	47.2
6	26.9	19.2	22.09	22.8	23.9	24.9	32.4	47.2
7	29.1	21.7	24.22	24.9	25.8	26.9	32.4	47.2
8	25.9	18.8	21.02	21.6	22.5	23.6	32.4	47.2
9	24.5	17.6	19.61	20.1	21.0	22.1	32.4	47.2
10	23.7	17.0	18.83	19.3	20.1	21.2	32.4	47.2
11	23.2	16.6	18.31	18.8	19.5	20.6	32.4	47.2
12	22.8	16.3	17.97	18.4	19.1	20.2	32.4	47.2
13	22.5	16.2	17.76	18.2	18.9	19.9	32.4	47.2
14	22.4	16.1	17.66	18.1	18.8	19.7	32.4	47.2

Table 4.9b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	32.7	22.3	38.1	40.8	34.8	32.1
	At Deterioration	32.7	22.3	38.1	33.2	29.2	25.4
Maximum Longitudinal Moment (Kip-ft/ft)		29.1	21.7	24.2	24.9	25.8	26.9
Maximum Live Load Deflection (in)		0.180	0.115	0.131	0.135	0.142	0.151

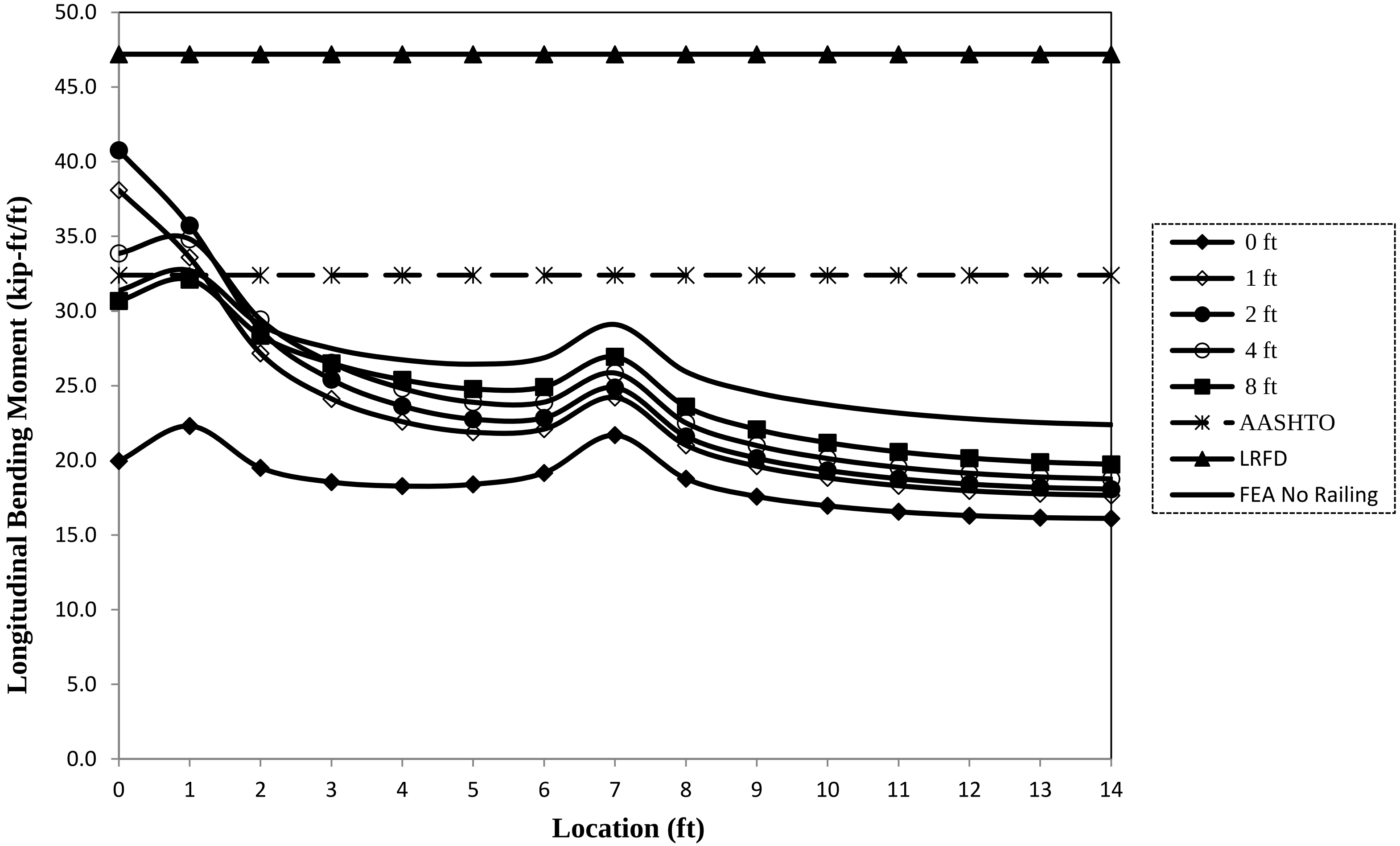


Figure 4.9: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

- **Span= 36 ft; One Lane; One Railing; E2; Half Deterioration**

Table 4.10a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	31.4	26.1	27.3	27.7	28.2	29.1	32.4	47.2
1	32.7	27.4	28.6	29.0	29.6	30.4	32.4	47.2
2	29.1	23.6	25.0	25.3	25.9	26.8	32.4	47.2
3	27.5	21.9	23.4	23.7	24.4	25.3	32.4	47.2
4	26.7	21.1	22.6	23.0	23.6	24.6	32.4	47.2
5	26.4	20.6	22.3	22.7	23.4	24.3	32.4	47.2
6	26.9	20.9	22.7	23.2	23.9	24.9	32.4	47.2
7	29.1	23.0	25.0	25.5	26.3	27.3	32.4	47.2
8	25.9	19.6	22.0	22.6	23.4	24.3	32.4	47.2
9	24.5	18.0	20.8	21.5	22.4	23.2	32.4	47.2
10	23.7	16.9	20.4	21.2	22.2	22.7	32.4	47.2
11	23.2	16.1	20.6	21.6	22.6	22.5	32.4	47.2
12	22.8	15.4	21.6	22.9	23.4	22.5	32.4	47.2
13	22.5	14.8	23.9	25.6	24.9	22.5	32.4	47.2
14	22.4	14.3	28.9	31.1	25.3	22.6	32.4	47.2

Table 4.10b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	32.7	27.4	28.9	31.1	29.6	30.4
	At Deterioration	32.7	27.4	29.0	26.0	24.3	22.2
Maximum Longitudinal Moment (Kip-ft/ft)		29.1	23.0	25.0	25.5	26.3	27.3
Maximum Live Load Deflection (in)		0.180	0.143	0.151	0.152	0.156	0.161

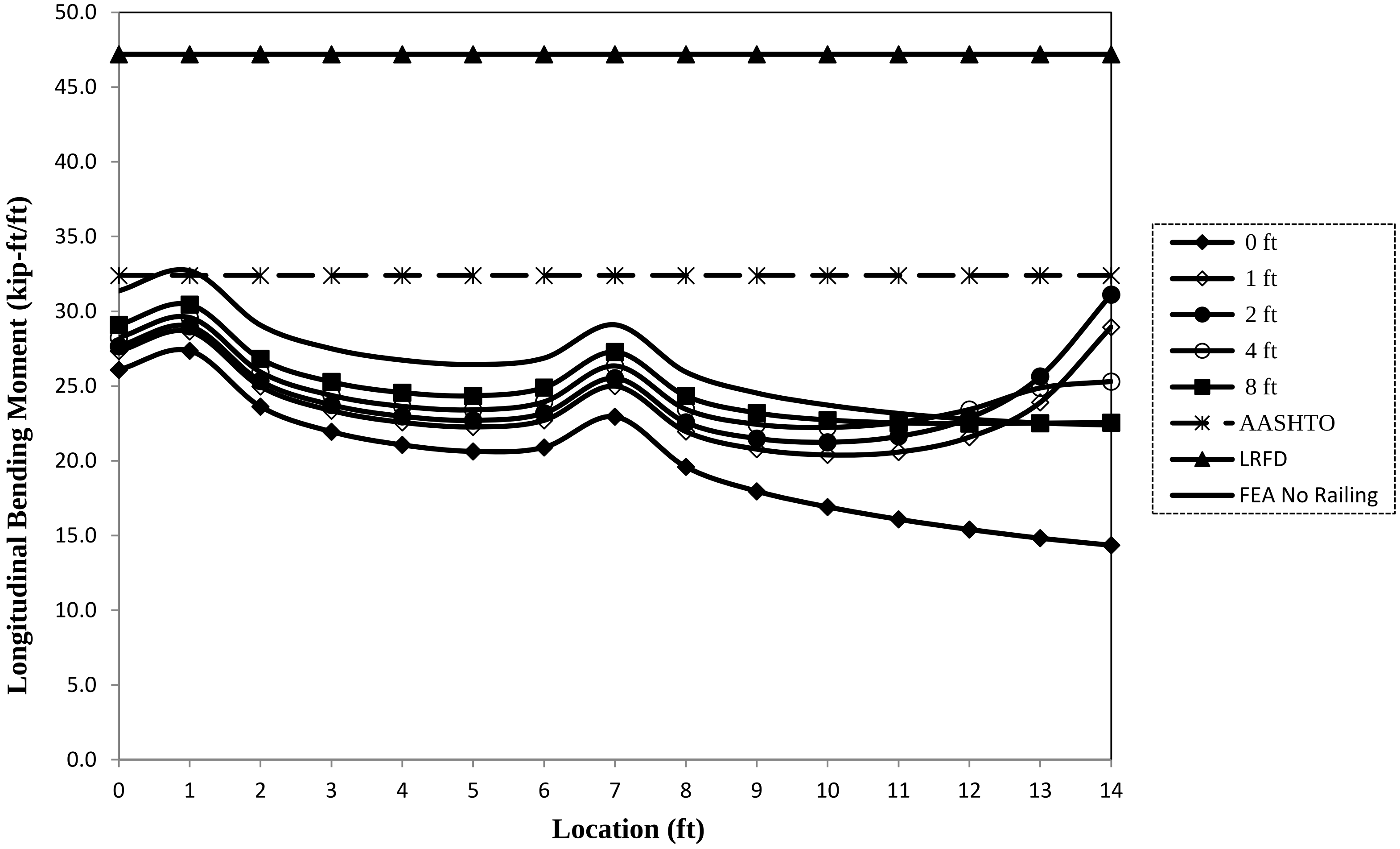


Figure 4.10: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration.

- **Span= 36 ft; Two Lane; One Railing; E1; Half Deterioration**

Table 4.11a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	37.2	25.0	48.5	51.8	42.4	37.8	32.4	45.6
1	38.5	27.5	41.9	44.6	43.1	39.1	32.4	45.6
2	34.8	24.8	34.5	36.4	37.1	35.3	32.4	45.6
3	33.3	24.0	31.0	32.5	33.7	33.3	32.4	45.6
4	32.6	24.0	29.2	30.4	31.8	32.1	32.4	45.6
5	32.3	24.3	28.5	29.5	30.7	31.5	32.4	45.6
6	32.9	25.3	28.7	29.6	30.7	31.7	32.4	45.6
7	35.3	28.2	31.1	31.8	32.8	33.8	32.4	45.6
8	32.5	25.8	28.2	28.8	29.7	30.8	32.4	45.6
9	31.6	25.2	27.3	27.9	28.7	29.7	32.4	45.6
10	31.8	25.7	27.6	28.1	28.9	29.8	32.4	45.6
11	33.9	28.2	29.8	30.3	31.0	31.9	32.4	45.6
12	30.8	25.3	26.8	27.2	27.8	28.7	32.4	45.6
13	29.4	24.2	25.6	26.0	26.5	27.4	32.4	45.6
14	28.8	23.8	25.1	25.4	26.0	26.8	32.4	45.6
15	28.6	23.8	25.0	25.3	25.9	26.6	32.4	45.6
16	29.1	24.5	25.6	25.9	26.4	27.1	32.4	45.6
17	31.4	26.9	28.0	28.3	28.8	29.4	32.4	45.6
18	28.3	24.0	25.0	25.3	25.7	26.4	32.4	45.6
19	26.9	22.8	23.7	24.0	24.4	25.0	32.4	45.6
20	26.1	22.1	23.0	23.3	23.7	24.3	32.4	45.6
21	25.6	21.7	22.6	22.8	23.2	23.8	32.4	45.6
22	25.3	21.4	22.3	22.5	22.9	23.5	32.4	45.6
23	25.1	21.3	22.2	22.4	22.8	23.3	32.4	45.6
24	25.0	21.3	22.1	22.3	22.7	23.2	32.4	45.6

Table 4.11b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	38.5	27.5	48.5	51.8	43.1	39.1
	At Deterioration	38.5	27.5	48.5	42.3	37.4	32.5
Maximum Longitudinal Moment (Kip-ft/ft)		35.3	28.2	31.1	31.8	32.8	33.8
Maximum Live Load Deflection (in)		0.219	0.149	0.168	0.173	0.181	0.190

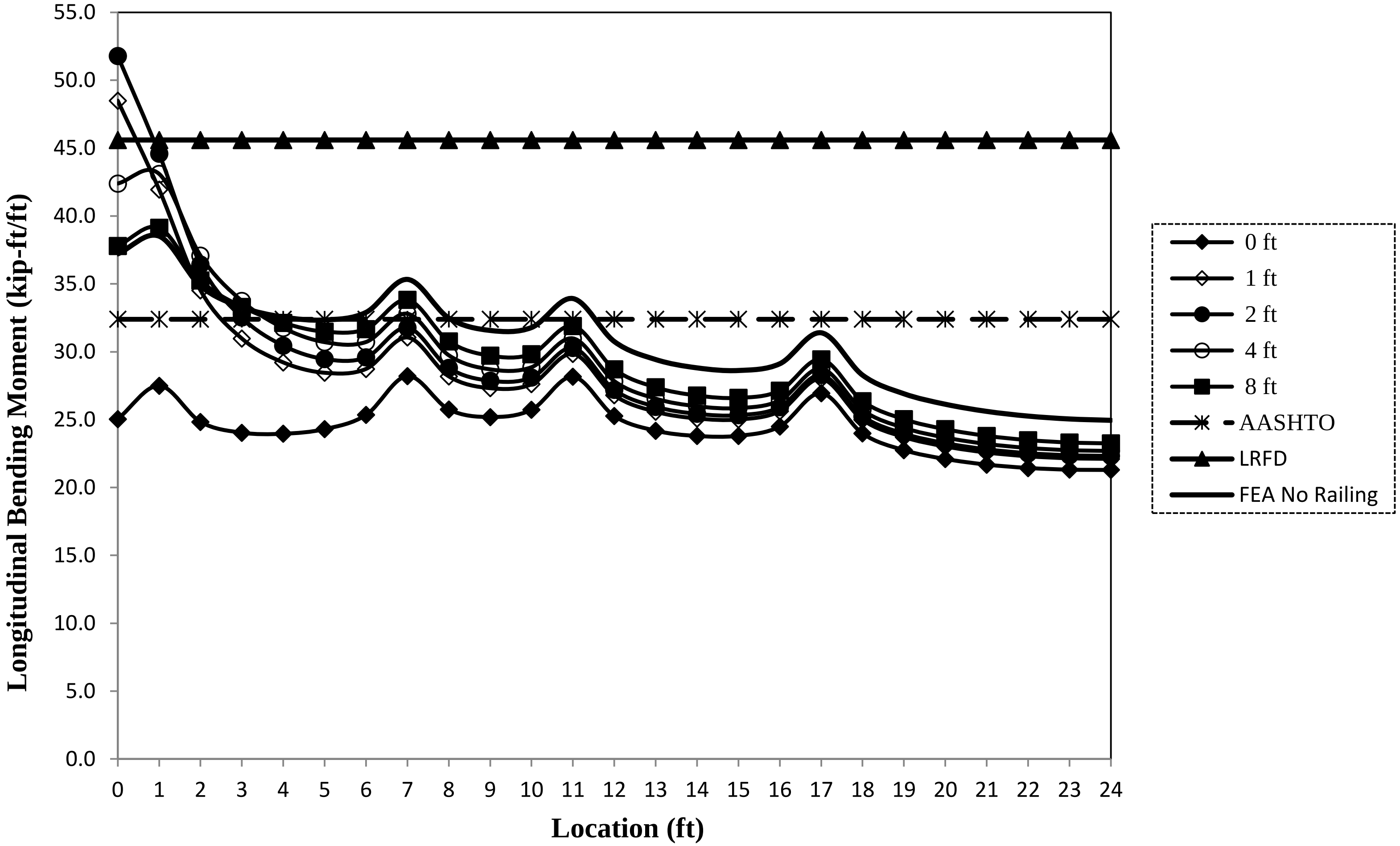


Figure 4.11: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration.

- **Span= 36 ft; Two Lane; One Railing; E2; Half Deterioration**

Table 4.12a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	37.2	34.4	35.0	35.2	35.4	35.9	32.4	45.6
1	38.5	35.6	36.3	36.4	36.7	37.2	32.4	45.6
2	34.8	31.9	32.5	32.7	33.0	33.5	32.4	45.6
3	33.3	30.3	30.9	31.1	31.4	31.9	32.4	45.6
4	32.6	29.4	30.1	30.3	30.6	31.1	32.4	45.6
5	32.3	29.1	29.9	30.0	30.4	30.9	32.4	45.6
6	32.9	29.6	30.3	30.5	30.9	31.4	32.4	45.6
7	35.3	31.9	32.7	32.9	33.3	33.8	32.4	45.6
8	32.5	28.9	29.8	30.0	30.4	30.9	32.4	45.6
9	31.6	27.9	28.8	29.0	29.4	30.0	32.4	45.6
10	31.8	27.9	28.9	29.1	29.6	30.2	32.4	45.6
11	33.9	29.9	30.9	31.2	31.7	32.3	32.4	45.6
12	30.8	26.5	27.7	28.0	28.5	29.2	32.4	45.6
13	29.4	25.0	26.3	26.6	27.1	27.9	32.4	45.6
14	28.8	24.2	25.6	25.9	26.5	27.3	32.4	45.6
15	28.6	23.8	25.3	25.7	26.4	27.2	32.4	45.6
16	29.1	24.0	25.8	26.3	27.0	27.8	32.4	45.6
17	31.4	26.0	28.1	28.7	29.5	30.3	32.4	45.6
18	28.3	22.6	25.1	25.8	26.6	27.4	32.4	45.6
19	26.9	20.9	24.0	24.8	25.7	26.3	32.4	45.6
20	26.1	19.8	23.7	24.6	25.6	25.9	32.4	45.6
21	25.6	18.9	24.1	25.2	26.2	25.8	32.4	45.6
22	25.3	18.2	25.4	26.8	27.3	25.9	32.4	45.6
23	25.1	17.5	28.3	30.3	29.1	26.0	32.4	45.6
24	25.0	17.0	34.5	36.9	29.7	26.1	32.4	45.6

Table 4.12b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	38.5	35.6	36.3	36.9	36.7	37.2
	At Deterioration	38.5	35.6	34.5	30.8	28.5	25.7
Maximum Longitudinal Moment (Kip-ft/ft)		35.3	31.9	32.7	32.9	33.3	33.8
Maximum Live Load Deflection (in)		0.219	0.199	0.203	0.204	0.206	0.209

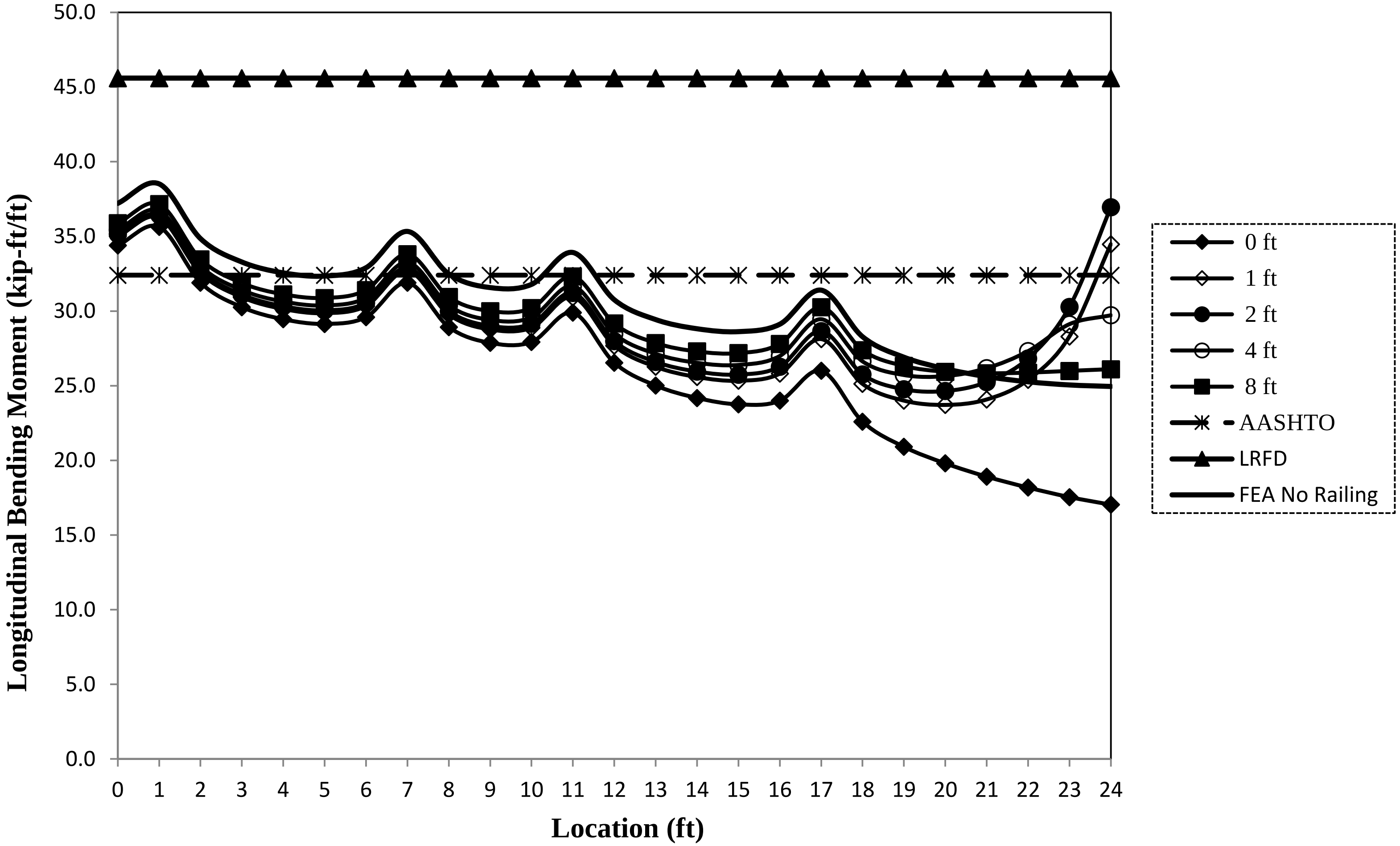


Figure 4.12: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

- **Span= 54 ft; One Lane; One Railing; E1; Half Deterioration**

Table 4.13a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	54.5	43.4	65.8	68.6	60.3	55.7	50.2	75.3
1	55.9	45.3	59.6	61.9	60.7	56.9	50.2	75.3
2	52.2	42.2	51.9	53.7	54.4	52.8	50.2	75.3
3	50.6	41.0	48.1	49.6	50.9	50.7	50.2	75.3
4	49.9	40.5	46.0	47.2	48.6	49.2	50.2	75.3
5	49.6	40.5	44.9	45.9	47.3	48.3	50.2	75.3
6	50.0	41.1	44.8	45.7	47.0	48.2	50.2	75.3
7	52.2	43.5	46.7	47.5	48.6	50.0	50.2	75.3
8	49.1	40.5	43.3	44.0	45.1	46.5	50.2	75.3
9	47.7	39.2	41.8	42.4	43.4	44.8	50.2	75.3
10	46.9	38.5	40.9	41.5	42.4	43.7	50.2	75.3
11	46.3	38.1	40.3	40.8	41.7	43.0	50.2	75.3
12	45.9	37.8	39.8	40.4	41.2	42.5	50.2	75.3
13	45.7	37.6	39.6	40.1	40.9	42.2	50.2	75.3
14	45.5	37.5	39.4	39.9	40.7	42.0	50.2	75.3

Table 4.13b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	55.9	45.3	65.8	68.6	60.7	56.9
	At Deterioration	55.9	45.3	65.8	60.4	55.8	52.0
Maximum Longitudinal Moment (Kip-ft/ft)		52.2	43.5	46.7	47.5	48.6	50.0
Maximum Live Load Deflection (in)		0.352	0.282	0.295	0.298	0.304	0.312

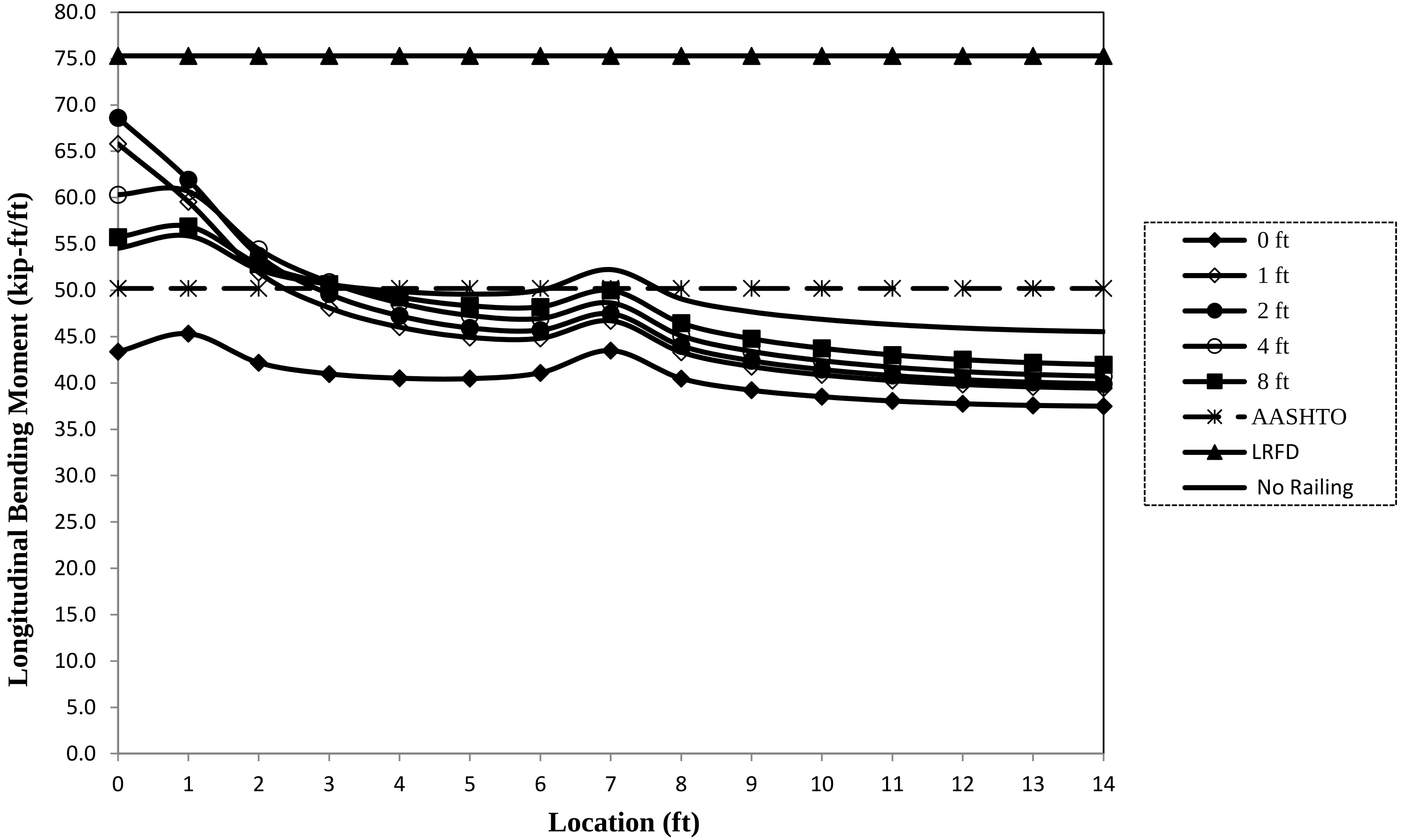


Figure 4.13: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

- **Span= 54 ft; One Lane; One Railing; E2; Half Deterioration**

Table 4.14a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	54.5	47.1	48.9	49.3	50.0	51.2	50.2	75.3
1	55.9	48.4	50.2	50.7	51.4	52.6	50.2	75.3
2	52.2	44.7	46.6	47.0	47.8	49.1	50.2	75.3
3	50.6	43.1	45.0	45.5	46.4	47.6	50.2	75.3
4	49.9	42.2	44.3	44.8	45.7	47.0	50.2	75.3
5	49.6	41.8	44.1	44.7	45.6	46.9	50.2	75.3
6	50.0	42.2	44.7	45.3	46.3	47.6	50.2	75.3
7	52.2	44.3	47.2	47.9	48.9	50.2	50.2	75.3
8	49.1	41.0	44.3	45.1	46.3	47.4	50.2	75.3
9	47.7	39.5	43.4	44.4	45.6	46.6	50.2	75.3
10	46.9	38.5	43.4	44.5	45.8	46.4	50.2	75.3
11	46.3	37.8	44.2	45.5	46.7	46.5	50.2	75.3
12	45.9	37.2	45.9	47.6	48.2	46.7	50.2	75.3
13	45.7	36.8	49.5	51.6	50.5	47.0	50.2	75.3
14	45.5	36.4	56.5	59.0	51.4	47.1	50.2	75.3

Table 4.14b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	55.9	48.4	56.5	59.0	51.4	52.6
	At Deterioration	55.9	48.4	56.8	53.0	50.8	48.8
Maximum Longitudinal Moment (Kip-ft/ft)		52.2	44.3	47.2	47.9	48.9	50.2
Maximum Live Load Deflection (in)		0.352	0.298	0.307	0.309	0.312	0.319

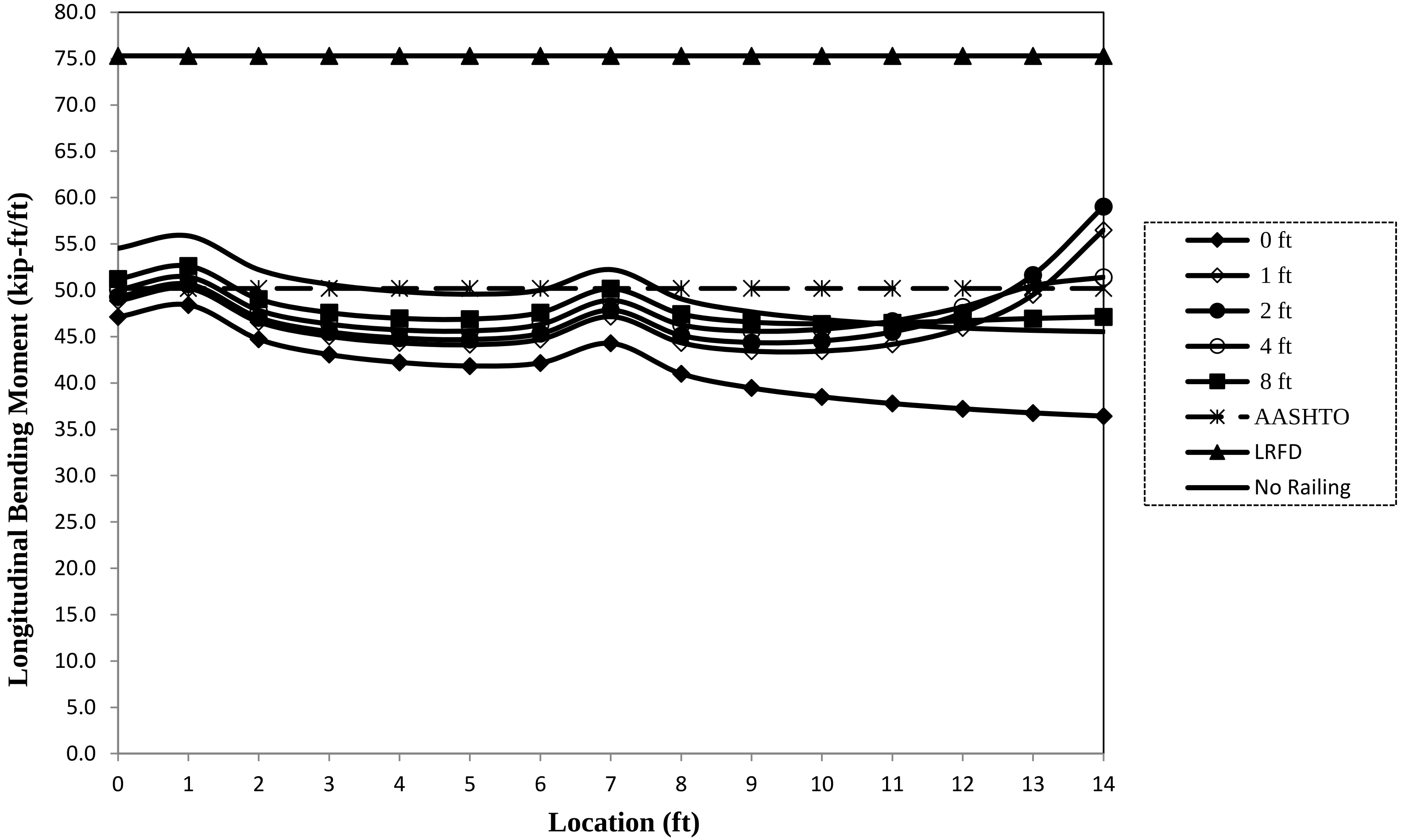


Figure 4.14: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

- **Span= 54 ft; Two Lane; One Railing; E1; Half Deterioration**

Table 4.15a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	64.5	54.0	82.5	85.9	74.8	68.3	50.2	81.7
1	65.8	56.0	73.8	76.7	74.7	69.3	50.2	81.7
2	62.1	52.8	64.9	67.0	67.6	65.0	50.2	81.7
3	60.5	51.7	60.3	62.1	63.4	62.6	50.2	81.7
4	59.7	51.3	57.8	59.2	60.7	61.0	50.2	81.7
5	59.4	51.4	56.5	57.7	59.1	59.9	50.2	81.7
6	60.0	52.2	56.4	57.3	58.6	59.7	50.2	81.7
7	62.3	54.8	58.3	59.1	60.3	61.5	50.2	81.7
8	59.5	52.2	55.2	55.9	56.9	58.2	50.2	81.7
9	58.5	51.4	54.0	54.6	55.6	56.8	50.2	81.7
10	58.7	51.8	54.1	54.6	55.5	56.7	50.2	81.7
11	60.8	54.1	56.2	56.6	57.5	58.6	50.2	81.7
12	57.7	51.1	52.9	53.4	54.1	55.2	50.2	81.7
13	56.3	49.9	51.6	52.0	52.7	53.7	50.2	81.7
14	55.7	49.4	51.0	51.4	52.0	53.0	50.2	81.7
15	55.5	49.4	50.8	51.2	51.8	52.7	50.2	81.7
16	56.0	50.0	51.3	51.7	52.3	53.2	50.2	81.7
17	58.3	52.4	53.7	54.0	54.5	55.4	50.2	81.7
18	55.2	49.4	50.6	50.9	51.4	52.3	50.2	81.7
19	53.8	48.1	49.3	49.6	50.1	50.9	50.2	81.7
20	53.1	47.4	48.6	48.9	49.3	50.1	50.2	81.7
21	52.6	47.0	48.1	48.4	48.9	49.6	50.2	81.7
22	52.3	46.8	47.8	48.1	48.6	49.3	50.2	81.7
23	52.1	46.6	47.7	47.9	48.4	49.1	50.2	81.7
24	52.0	46.6	47.6	47.9	48.3	49.1	50.2	81.7

Table 4.15b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	65.8	56.0	82.5	85.9	74.8	69.3
	At Deterioration	65.8	56.0	82.5	75.7	70.2	65.4
Maximum Longitudinal Moment (Kip-ft/ft)		62.3	54.8	58.3	59.1	60.3	61.5
Maximum Live Load Deflection (in)		0.423	0.357	0.372	0.375	0.380	0.389

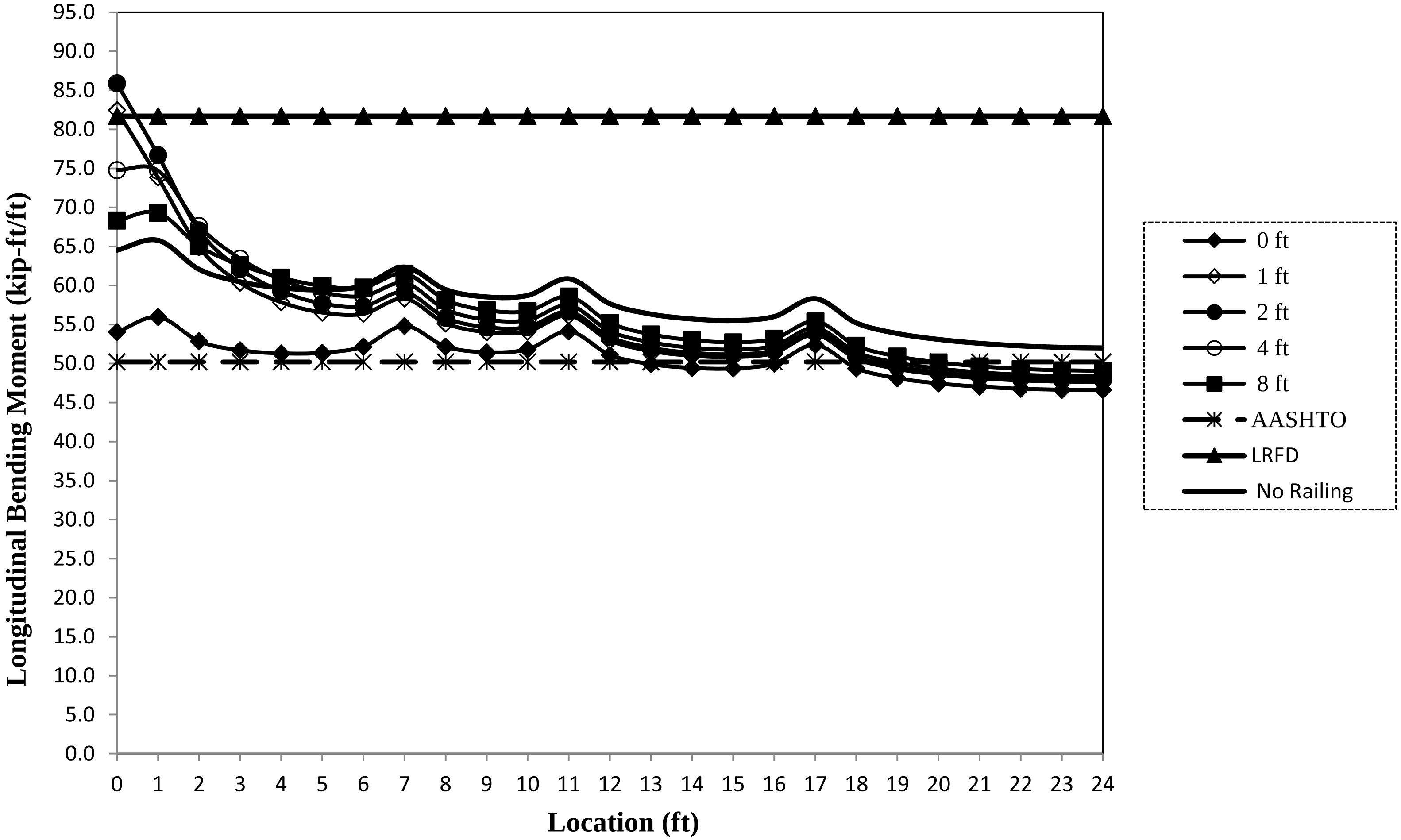


Figure 4.15: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E1, Half Depth Railing Deterioration

- **Span= 54 ft; Two Lane; One Railing; E2; Half Deterioration**

Table 4.16a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	64.5	59.8	60.7	60.9	61.3	61.9	50.2	81.7
1	65.8	61.0	61.9	62.1	62.5	63.2	50.2	81.7
2	62.1	57.2	58.2	58.4	58.8	59.5	50.2	81.7
3	60.5	55.6	56.5	56.8	57.2	57.8	50.2	81.7
4	59.7	54.7	55.7	56.0	56.4	57.1	50.2	81.7
5	59.4	54.4	55.4	55.7	56.1	56.8	50.2	81.7
6	60.0	54.8	55.9	56.2	56.6	57.4	50.2	81.7
7	62.3	57.2	58.3	58.5	59.0	59.8	50.2	81.7
8	59.5	54.2	55.4	55.6	56.2	56.9	50.2	81.7
9	58.5	53.1	54.4	54.7	55.2	56.1	50.2	81.7
10	58.7	53.2	54.6	54.9	55.5	56.3	50.2	81.7
11	60.8	55.2	56.7	57.0	57.6	58.5	50.2	81.7
12	57.7	51.9	53.5	53.9	54.5	55.5	50.2	81.7
13	56.3	50.4	52.2	52.6	53.3	54.3	50.2	81.7
14	55.7	49.7	51.6	52.1	52.9	53.9	50.2	81.7
15	55.5	49.3	51.6	52.1	52.9	54.0	50.2	81.7
16	56.0	49.7	52.2	52.8	53.8	54.9	50.2	81.7
17	58.3	51.8	54.8	55.5	56.5	57.6	50.2	81.7
18	55.2	48.5	52.1	52.9	54.1	55.0	50.2	81.7
19	53.8	47.0	51.4	52.4	53.6	54.3	50.2	81.7
20	53.1	46.0	51.6	52.8	54.1	54.3	50.2	81.7
21	52.6	45.3	52.7	54.2	55.4	54.7	50.2	81.7
22	52.3	44.7	55.0	56.9	57.4	55.1	50.2	81.7
23	52.1	44.3	59.6	62.0	60.3	55.5	50.2	81.7
24	52.0	43.9	68.3	71.3	61.5	55.8	50.2	81.7

Table 4.16b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	65.8	61.0	68.3	71.3	62.5	63.2
	At Deterioration	65.8	61.0	68.7	63.7	60.7	57.9
Maximum Longitudinal Moment (Kip-ft/ft)		62.3	57.2	58.3	58.5	59.0	59.8
Maximum Live Load Deflection (in)		0.423	0.389	0.394	0.395	0.398	0.402

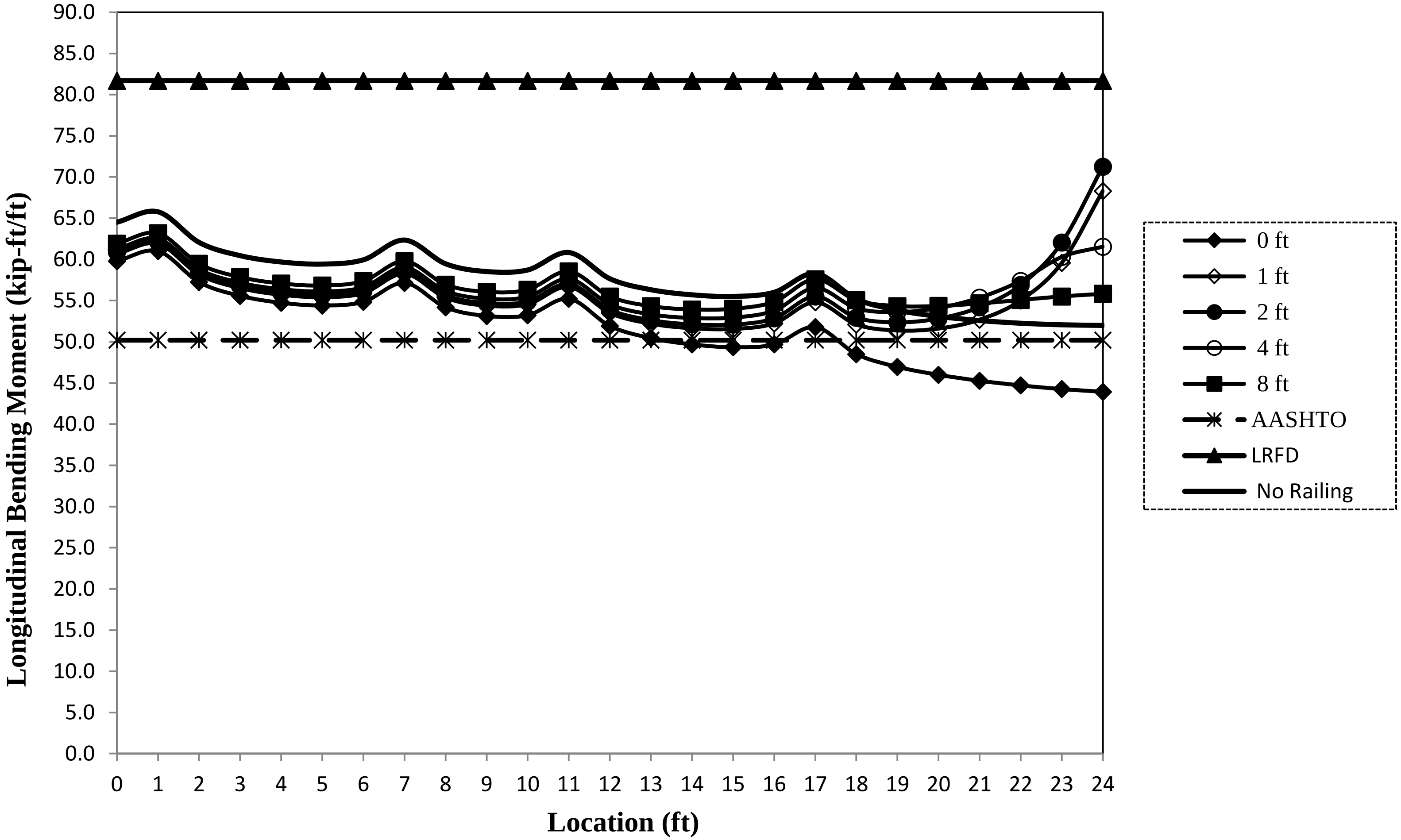


Figure 4.16: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, One Railing with Edge Loading E2, Half Depth Railing Deterioration

4.2.3 FEA Longitudinal Moments and Deflections for Categories “R2-Full”, “R2-Ref”, and “R0-Ref” with AASHTO Moments

- **Span= 36 ft; One Lane; Two Railings; E1; Full Deterioration**
- **Span= 36 ft; Two Lane; Two Railings; E1; Full Deterioration**
- **Span= 54 ft; One Lane; Two Railings; E1; Full Deterioration**
- **Span= 54 ft; Two Lane; Two Railings; E1; Full Deterioration**

- **Span= 36 ft; One Lane; Two Railings; E1; Full Deterioration**

Table 4.17a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	31.4	17.0	43.1	33.4	30.5	27.7	32.4	47.2
1	32.7	19.2	36.4	35.5	31.4	29.0	32.4	47.2
2	29.1	16.3	27.9	28.3	27.0	25.1	32.4	47.2
3	27.5	15.2	23.8	24.4	24.3	23.2	32.4	47.2
4	26.7	14.8	21.5	22.1	22.4	22.0	32.4	47.2
5	26.4	14.8	20.1	20.7	21.2	21.3	32.4	47.2
6	26.9	15.4	19.8	20.3	20.9	21.2	32.4	47.2
7	29.1	17.7	21.5	22.0	22.6	23.0	32.4	47.2
8	25.9	14.6	17.9	18.3	18.9	19.5	32.4	47.2
9	24.5	13.2	16.2	16.5	17.1	17.7	32.4	47.2
10	23.7	12.4	15.0	15.4	15.9	16.5	32.4	47.2
11	23.2	11.8	14.2	14.5	14.9	15.5	32.4	47.2
12	22.8	11.3	13.5	13.7	14.2	14.8	32.4	47.2
13	22.5	10.8	12.9	13.1	13.6	14.2	32.4	47.2
14	22.4	10.5	12.4	12.7	13.1	13.6	32.4	47.2

Table 4.17b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	32.7	19.2	43.1	35.5	31.4	29.0
	At Deterioration	32.7	22.3	43.1	39.6	34.6	29.5
Maximum Longitudinal Moment (Kip-ft/ft)		29.1	17.7	21.5	22.0	22.6	23.0
Maximum Live Load Deflection (in)		0.180	0.095	0.120	0.123	0.128	0.134

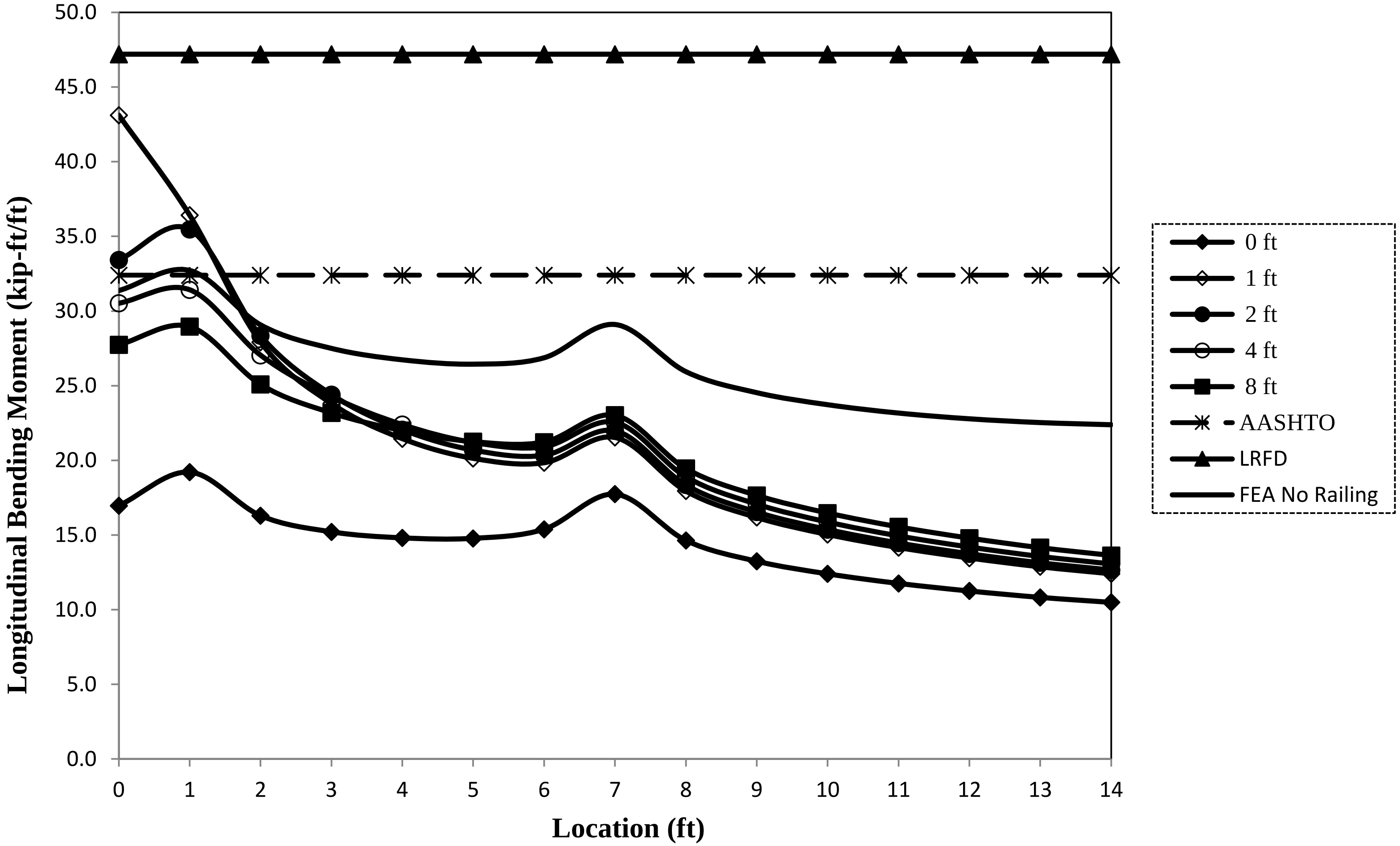


Figure 4.17: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

- **Span= 36 ft; Two Lane; Two Railings; E1; Full Deterioration**

Table 4.18a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	37.2	23.2	59.4	45.6	41.3	37.1	32.4	45.6
1	38.5	25.5	49.4	47.9	42.0	38.2	32.4	45.6
2	34.8	22.8	38.9	39.4	37.3	34.3	32.4	45.6
3	33.3	21.9	33.7	34.5	34.1	32.3	32.4	45.6
4	32.6	21.7	30.8	31.5	31.9	31.1	32.4	45.6
5	32.3	21.9	29.2	29.9	30.5	30.3	32.4	45.6
6	32.9	22.9	28.8	29.4	30.1	30.4	32.4	45.6
7	35.3	25.6	30.6	31.1	31.8	32.3	32.4	45.6
8	32.5	23.0	27.3	27.8	28.4	29.0	32.4	45.6
9	31.6	22.3	26.0	26.5	27.1	27.7	32.4	45.6
10	31.8	22.7	26.0	26.4	26.9	27.6	32.4	45.6
11	33.9	24.9	27.9	28.2	28.8	29.4	32.4	45.6
12	30.8	21.9	24.5	24.8	25.3	26.0	32.4	45.6
13	29.4	20.6	23.0	23.3	23.7	24.4	32.4	45.6
14	28.8	20.0	22.2	22.4	22.9	23.5	32.4	45.6
15	28.6	19.8	21.8	22.0	22.5	23.0	32.4	45.6
16	29.1	20.2	22.1	22.3	22.7	23.3	32.4	45.6
17	31.4	22.4	24.2	24.4	24.7	25.3	32.4	45.6
18	28.3	19.2	20.8	21.0	21.4	21.9	32.4	45.6
19	26.9	17.7	19.2	19.4	19.7	20.2	32.4	45.6
20	26.1	16.7	18.1	18.3	18.6	19.1	32.4	45.6
21	25.6	16.0	17.3	17.5	17.8	18.2	32.4	45.6
22	25.3	15.4	16.6	16.8	17.1	17.5	32.4	45.6
23	25.1	14.8	16.1	16.2	16.5	16.9	32.4	45.6
24	25.0	14.5	15.6	15.8	16.0	16.4	32.4	45.6

Table 4.18b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	38.5	25.5	59.4	47.9	42.0	38.2
	At Deterioration	38.5	25.5	59.4	55.3	49.0	42.3
Maximum Longitudinal Moment (Kip-ft/ft)		35.3	25.6	30.6	31.1	31.8	32.3
Maximum Live Load Deflection (in)		0.219	0.137	0.171	0.175	0.180	0.188

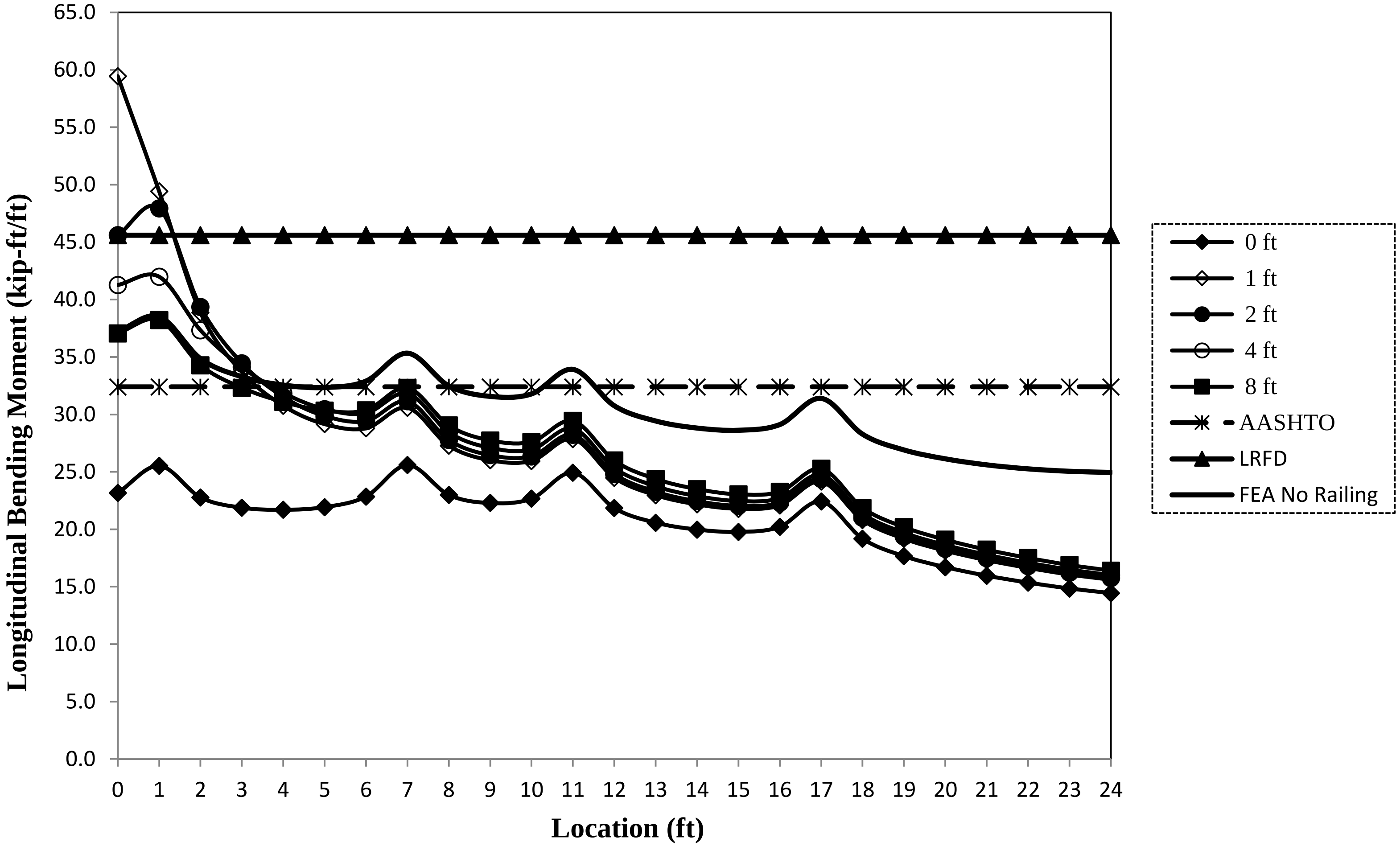


Figure 4.18: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration.

- **Span= 54 ft; One Lane; Two Railings; E1; Full Deterioration**

Table 4.19a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	54.5	37.7	68.3	57.0	53.4	49.7	50.2	75.3
1	55.9	39.6	60.4	59.2	54.2	51.0	50.2	75.3
2	52.2	36.4	50.8	51.2	49.6	47.0	50.2	75.3
3	50.6	35.1	45.9	46.6	46.5	45.0	50.2	75.3
4	49.9	34.6	42.9	43.7	44.2	43.7	50.2	75.3
5	49.6	34.4	41.2	41.9	42.6	42.8	50.2	75.3
6	50.0	35.0	40.7	41.3	42.1	42.6	50.2	75.3
7	52.2	37.3	42.1	42.7	43.5	44.3	50.2	75.3
8	49.1	34.1	38.4	39.0	39.7	40.6	50.2	75.3
9	47.7	32.7	36.6	37.1	37.8	38.8	50.2	75.3
10	46.9	31.9	35.4	35.8	36.6	37.5	50.2	75.3
11	46.3	31.3	34.5	34.9	35.6	36.6	50.2	75.3
12	45.9	30.8	33.8	34.2	34.9	35.9	50.2	75.3
13	45.7	30.4	33.3	33.7	34.3	35.3	50.2	75.3
14	45.5	30.2	32.9	33.3	33.9	34.9	50.2	75.3

Table 4.19b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	55.9	39.6	68.3	59.2	54.2	51.0
	At Deterioration	55.9	39.6	68.3	65.2	60.4	56.2
Maximum Longitudinal Moment (Kip-ft/ft)		52.2	37.3	42.1	42.7	43.5	44.3
Maximum Live Load Deflection (in)		0.352	0.241	0.262	0.264	0.269	0.276

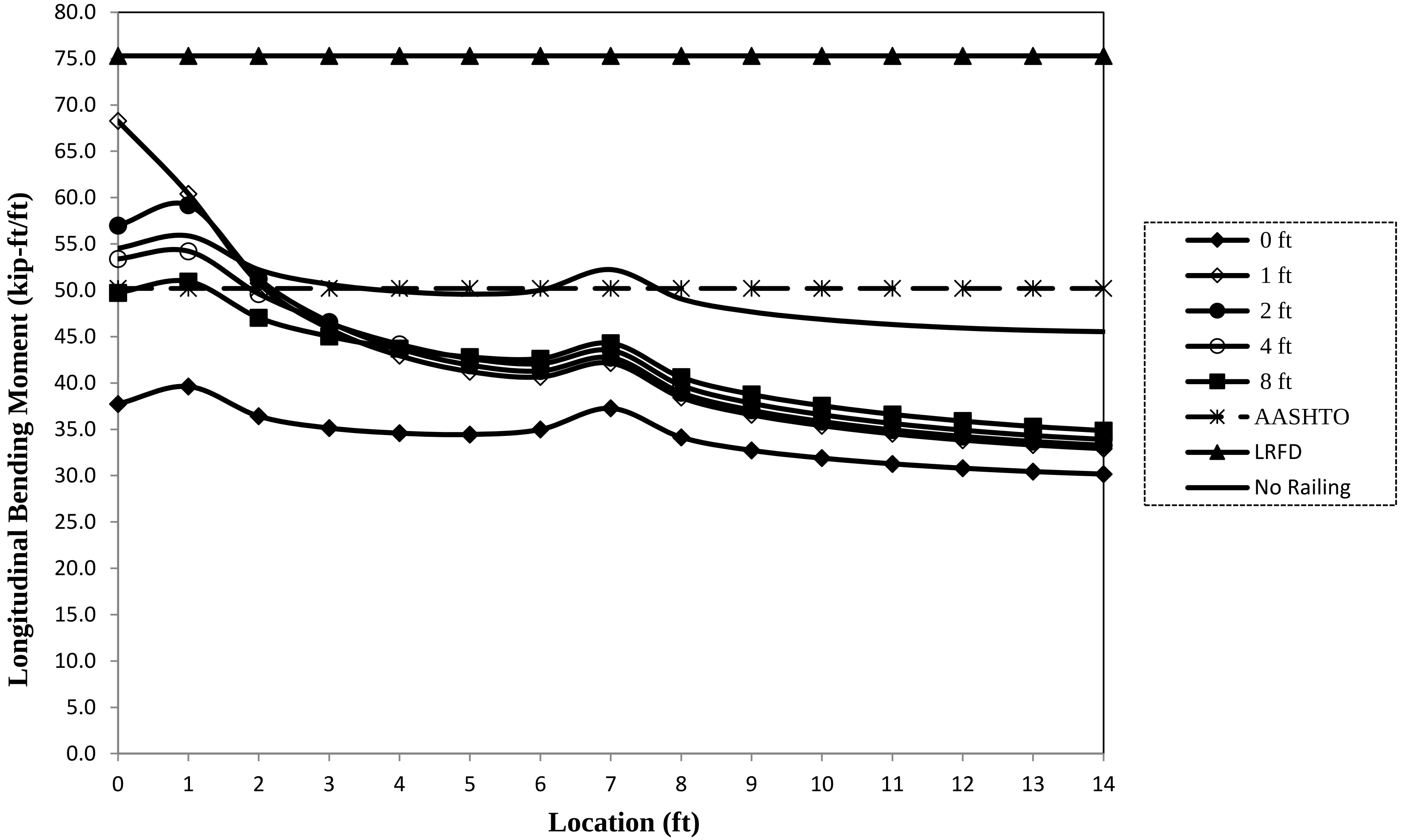


Figure 4.19: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

- **Span= 54 ft; Two Lane; Two Railings; E1; Full Deterioration**

Table 4.20a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	64.5	50.1	90.4	75.0	69.8	64.4	50.2	81.7
1	65.8	51.9	79.2	77.4	70.4	65.5	50.2	81.7
2	62.1	48.8	67.4	67.9	65.4	61.5	50.2	81.7
3	60.5	47.5	61.3	62.1	61.7	59.3	50.2	81.7
4	59.7	47.1	57.6	58.5	58.9	57.8	50.2	81.7
5	59.4	47.1	55.5	56.3	57.0	56.8	50.2	81.7
6	60.0	47.8	54.7	55.4	56.2	56.5	50.2	81.7
7	62.3	50.4	56.1	56.8	57.6	58.2	50.2	81.7
8	59.5	47.6	52.5	53.1	53.9	54.7	50.2	81.7
9	58.5	46.8	51.1	51.6	52.3	53.2	50.2	81.7
10	58.7	47.0	50.8	51.3	52.0	52.9	50.2	81.7
11	60.8	49.2	52.6	53.0	53.7	54.6	50.2	81.7
12	57.7	46.1	49.1	49.5	50.2	51.0	50.2	81.7
13	56.3	44.7	47.5	47.9	48.5	49.4	50.2	81.7
14	55.7	44.1	46.7	47.0	47.6	48.4	50.2	81.7
15	55.5	43.9	46.3	46.6	47.2	48.0	50.2	81.7
16	56.0	44.4	46.6	46.9	47.4	48.2	50.2	81.7
17	58.3	46.6	48.7	49.0	49.5	50.2	50.2	81.7
18	55.2	43.4	45.4	45.7	46.2	46.9	50.2	81.7
19	53.8	42.0	43.9	44.1	44.6	45.3	50.2	81.7
20	53.1	41.1	42.9	43.2	43.6	44.3	50.2	81.7
21	52.6	40.5	42.2	42.5	42.9	43.6	50.2	81.7
22	52.3	40.0	41.7	41.9	42.3	43.0	50.2	81.7
23	52.1	39.6	41.3	41.5	41.9	42.5	50.2	81.7
24	52.0	39.4	41.0	41.2	41.6	42.2	50.2	81.7

Table 4.20b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	65.8	51.9	90.4	77.4	70.4	65.5
	At Deterioration	65.8	51.9	90.4	86.9	81.0	75.5
Maximum Longitudinal Moment (Kip-ft/ft)		62.3	50.4	56.1	56.8	57.6	58.2
Maximum Live Load Deflection (in)		0.423	0.329	0.353	0.355	0.360	0.368

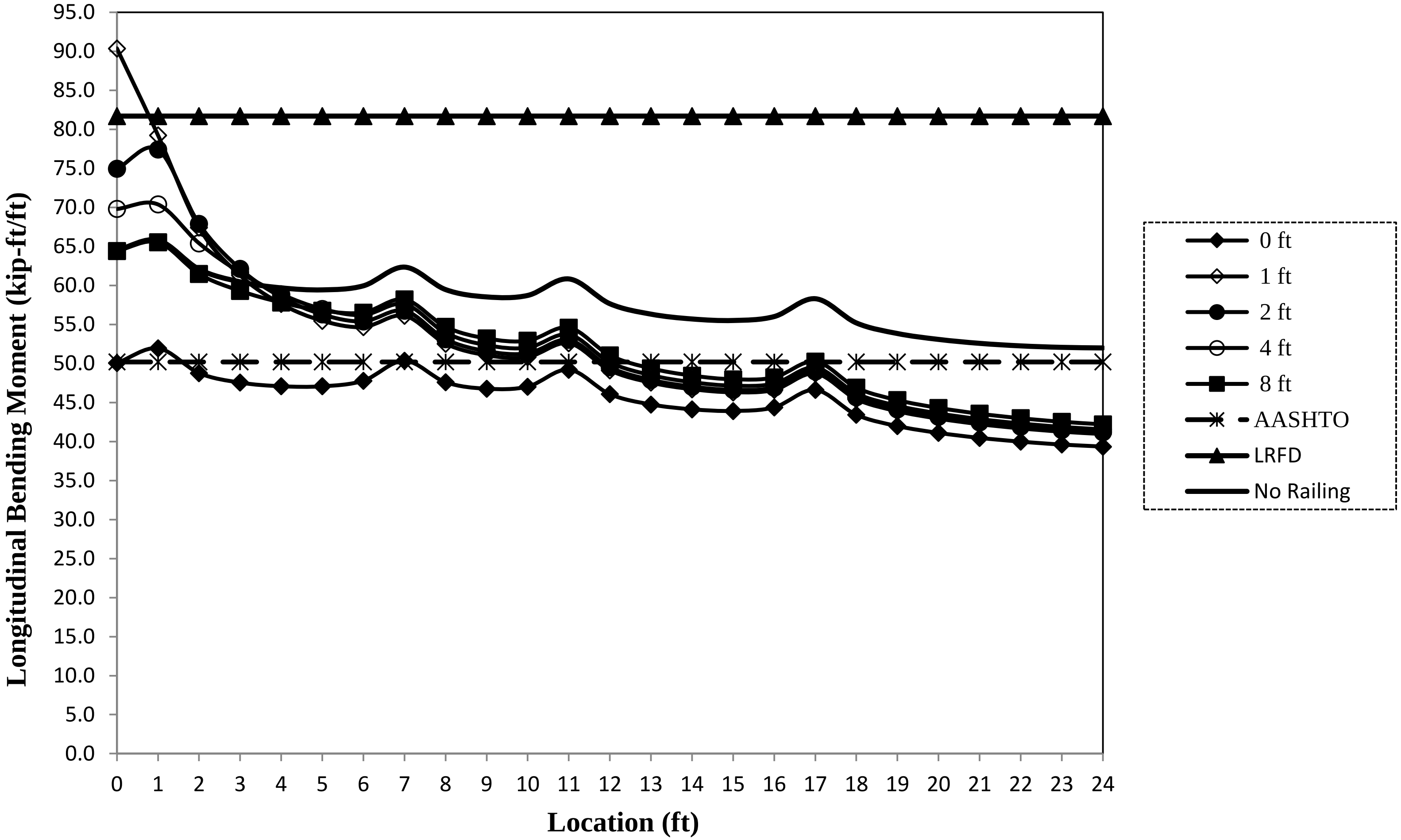


Figure 4.20: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Full Depth Railing Deterioration

4.2.4 FEA Longitudinal Moments and Deflections for Categories "R2-Half", "R2-Ref", and "R0-Ref" with AASHTO Moments

- **Span= 36 ft; One Lane; Two Railings; E1; Half Deterioration**
- **Span= 36 ft; Two Lane; Two Railings; E1; Half Deterioration**
- **Span= 54 ft; One Lane; Two Railings; E1; Half Deterioration**
- **Span= 54 ft; Two Lane; Two Railings; E1; Half Deterioration**

- **Span= 36 ft; One Lane; Two Railings; E1; Half Deterioration**

Table 4.21a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	31.4	17.0	32.4	34.6	28.6	25.7	32.4	47.2
1	32.7	19.2	28.7	30.5	29.6	27.1	32.4	47.2
2	29.1	16.3	22.7	24.0	24.4	23.4	32.4	47.2
3	27.5	15.2	19.8	20.8	21.7	21.5	32.4	47.2
4	26.7	14.8	18.3	19.1	20.0	20.3	32.4	47.2
5	26.4	14.8	17.6	18.2	19.1	19.7	32.4	47.2
6	26.9	15.4	17.7	18.2	19.0	19.7	32.4	47.2
7	29.1	17.7	19.7	20.2	20.9	21.6	32.4	47.2
8	25.9	14.6	16.3	16.7	17.4	18.1	32.4	47.2
9	24.5	13.2	14.7	15.1	15.7	16.4	32.4	47.2
10	23.7	12.4	13.7	14.0	14.6	15.3	32.4	47.2
11	23.2	11.8	12.9	13.3	13.7	14.4	32.4	47.2
12	22.8	11.3	12.3	12.6	13.1	13.7	32.4	47.2
13	22.5	10.8	11.8	12.1	12.5	13.1	32.4	47.2
14	22.4	10.5	11.4	11.7	12.1	12.6	32.4	47.2

Table 4.21b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	32.7	19.2	32.4	34.6	29.6	27.1
	At Deterioration	32.7	19.2	32.4	27.9	24.1	20.5
Maximum Longitudinal Moment (Kip-ft/ft)		29.1	17.7	19.7	20.2	20.9	21.6
Maximum Live Load Deflection (in)		0.180	0.095	0.107	0.111	0.116	0.122

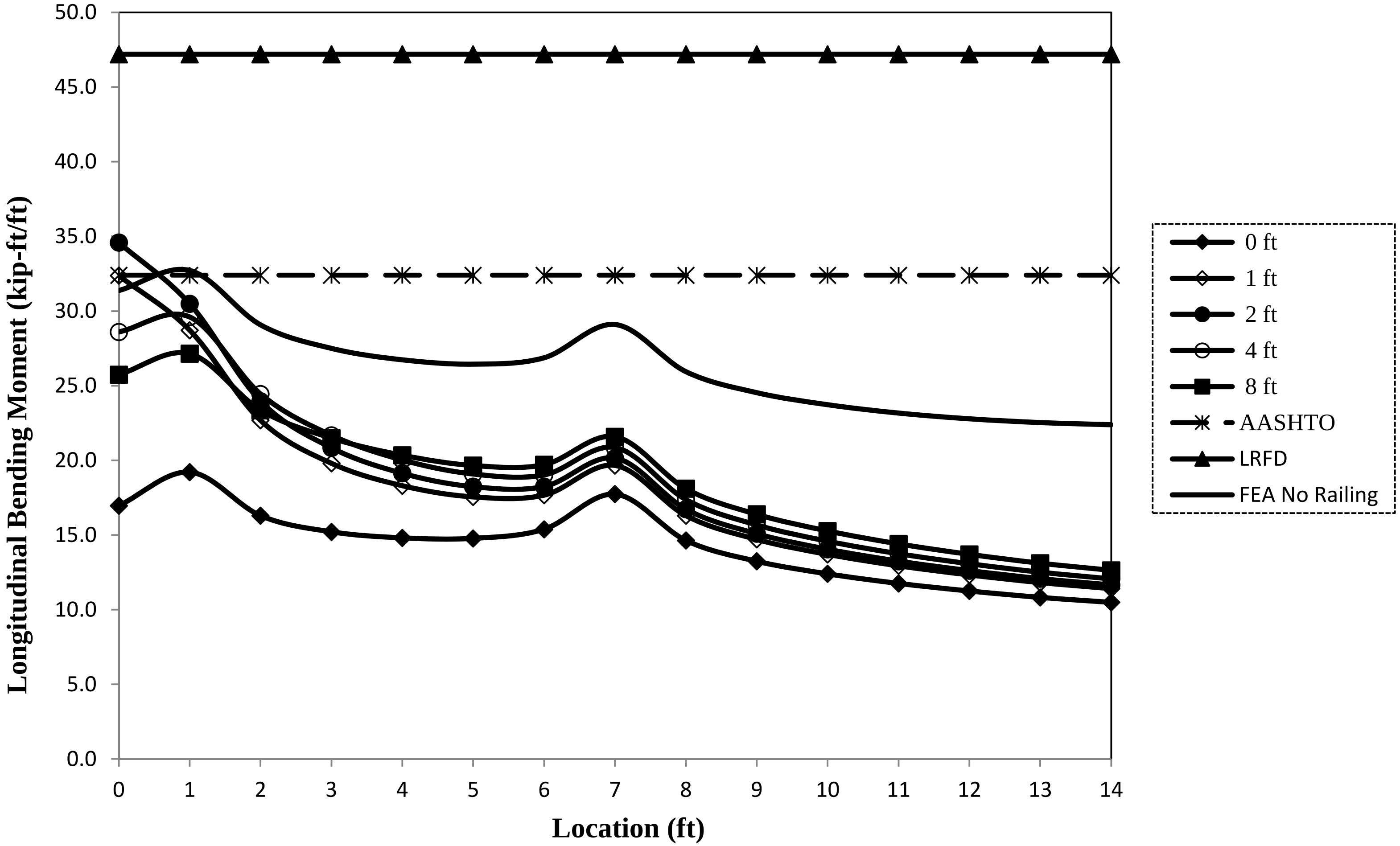


Figure 4.21: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

- **Span= 36 ft; Two Lane; Two Railings; E1; Half Deterioration**

Table 4.22a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	37.2	23.2	45.0	48.1	39.3	35.0	32.4	45.6
1	38.5	25.5	39.0	41.4	40.0	36.3	32.4	45.6
2	34.8	22.8	31.8	33.5	34.1	32.4	32.4	45.6
3	33.3	21.9	28.3	29.7	30.8	30.4	32.4	45.6
4	32.6	21.7	26.5	27.7	28.9	29.2	32.4	45.6
5	32.3	21.9	25.7	26.7	27.8	28.5	32.4	45.6
6	32.9	22.9	26.0	26.7	27.7	28.6	32.4	45.6
7	35.3	25.6	28.2	28.8	29.7	30.6	32.4	45.6
8	32.5	23.0	25.2	25.8	26.6	27.5	32.4	45.6
9	31.6	22.3	24.2	24.7	25.4	26.3	32.4	45.6
10	31.8	22.7	24.3	24.8	25.4	26.3	32.4	45.6
11	33.9	24.9	26.4	26.8	27.4	28.2	32.4	45.6
12	30.8	21.9	23.2	23.5	24.1	24.8	32.4	45.6
13	29.4	20.6	21.8	22.1	22.6	23.3	32.4	45.6
14	28.8	20.0	21.1	21.4	21.8	22.5	32.4	45.6
15	28.6	19.8	20.8	21.0	21.5	22.1	32.4	45.6
16	29.1	20.2	21.1	21.4	21.8	22.4	32.4	45.6
17	31.4	22.4	23.3	23.5	23.9	24.4	32.4	45.6
18	28.3	19.2	20.0	20.2	20.5	21.1	32.4	45.6
19	26.9	17.7	18.4	18.6	18.9	19.4	32.4	45.6
20	26.1	16.7	17.4	17.6	17.9	18.4	32.4	45.6
21	25.6	16.0	16.6	16.8	17.1	17.5	32.4	45.6
22	25.3	15.4	16.0	16.2	16.4	16.8	32.4	45.6
23	25.1	14.8	15.4	15.6	15.9	16.3	32.4	45.6
24	25.0	14.5	15.0	15.2	15.4	15.8	32.4	45.6

Table 4.22b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	38.5	25.5	45.0	48.1	40.0	36.3
	At Deterioration	38.5	25.5	45.0	39.1	34.4	29.8
Maximum Longitudinal Moment (Kip-ft/ft)		35.3	25.6	28.2	28.8	29.7	30.6
Maximum Live Load Deflection (in)		0.219	0.137	0.154	0.158	0.165	0.174

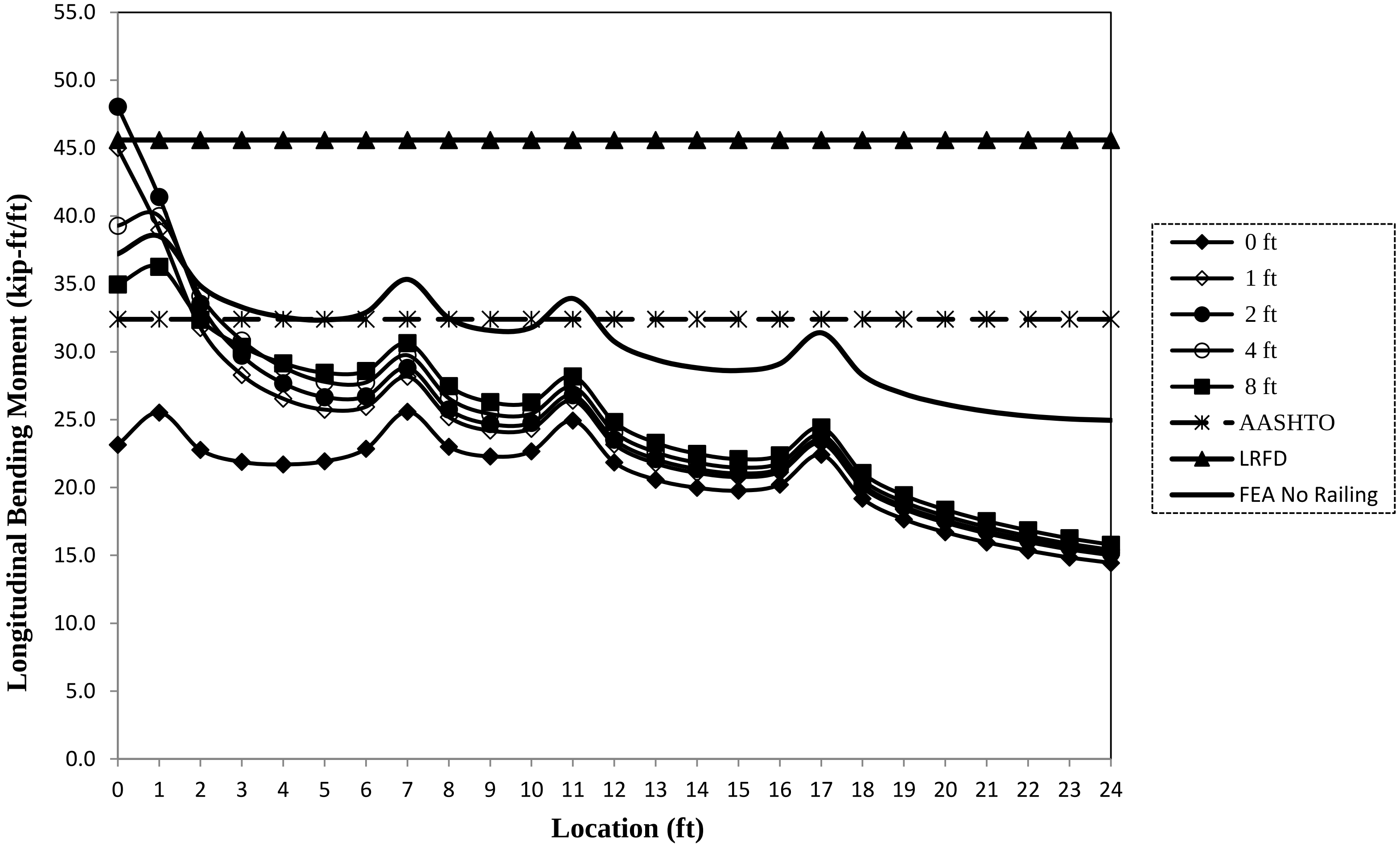


Figure 4.22: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 36 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration.

- **Span= 54 ft; One Lane; Two Railings; E1; Half Deterioration**

Table 4.23a: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	54.5	37.7	57.7	60.2	52.7	48.5	50.2	75.3
1	55.9	39.6	52.2	54.3	53.1	49.7	50.2	75.3
2	52.2	36.4	45.0	46.6	47.1	45.6	50.2	75.3
3	50.6	35.1	41.4	42.7	43.7	43.4	50.2	75.3
4	49.9	34.6	39.4	40.4	41.6	42.0	50.2	75.3
5	49.6	34.4	38.3	39.1	40.3	41.1	50.2	75.3
6	50.0	35.0	38.1	38.9	39.9	40.9	50.2	75.3
7	52.2	37.3	40.0	40.6	41.6	42.6	50.2	75.3
8	49.1	34.1	36.5	37.1	37.9	39.0	50.2	75.3
9	47.7	32.7	34.8	35.3	36.1	37.2	50.2	75.3
10	46.9	31.9	33.8	34.3	35.0	36.1	50.2	75.3
11	46.3	31.3	33.0	33.4	34.2	35.2	50.2	75.3
12	45.9	30.8	32.4	32.8	33.5	34.5	50.2	75.3
13	45.7	30.4	32.0	32.3	33.0	34.0	50.2	75.3
14	45.5	30.2	31.6	32.0	32.6	33.5	50.2	75.3

Table 4.23b: FEA Summary Results for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	55.9	39.6	57.7	60.2	53.1	49.7
	At Deterioration	55.9	39.6	57.7	52.6	48.2	44.6
Maximum Longitudinal Moment (Kip-ft/ft)		52.2	37.3	40.0	40.6	41.6	42.6
Maximum Live Load Deflection (in)		0.352	0.241	0.252	0.255	0.259	0.266

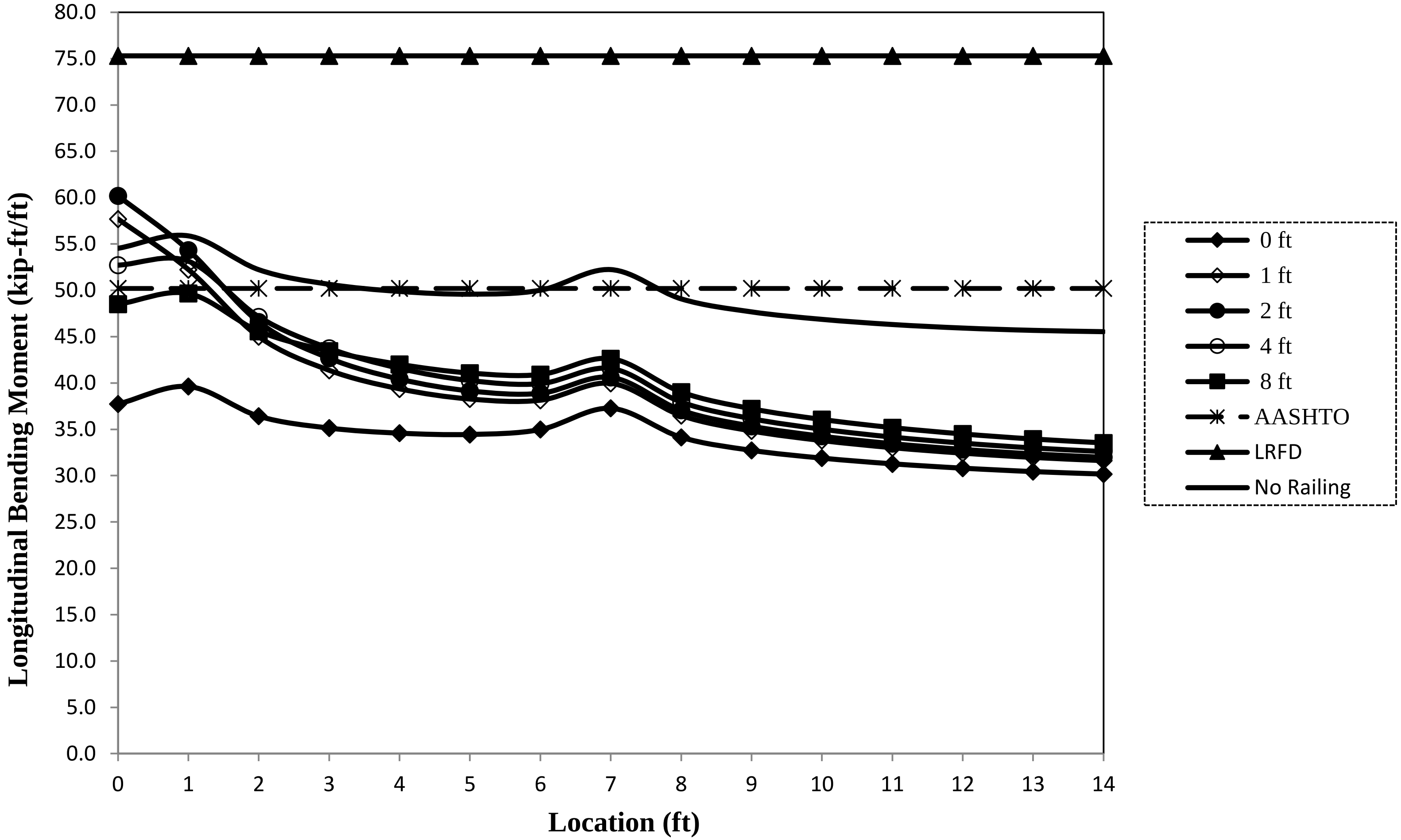


Figure 4.23: Longitudinal Moment Distribution at Critical Section for One-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 14 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

- **Span= 54 ft; Two Lane; Two Railings; E1; Half Deterioration**

Table 4.24a: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

Location (ft)	FEA Longitudinal Moment at Critical Section (Kip-ft/ft)						AASHTO Specs Moment (Kip-ft/ft)	AASHTO LRFD Moment (Kip-ft/ft)
	No Railing	Railing Deterioration Width						
		0 ft	1 ft	2 ft	4 ft	8 ft		
0	64.5	50.1	76.9	80.1	69.6	63.5	50.2	81.7
1	65.8	51.9	68.8	71.5	69.6	64.5	50.2	81.7
2	62.1	48.8	60.1	62.1	62.7	60.2	50.2	81.7
3	60.5	47.5	55.7	57.3	58.5	57.8	50.2	81.7
4	59.7	47.1	53.2	54.5	55.9	56.1	50.2	81.7
5	59.4	47.1	51.9	53.0	54.3	55.0	50.2	81.7
6	60.0	47.8	51.7	52.6	53.8	54.8	50.2	81.7
7	62.3	50.4	53.6	54.4	55.4	56.5	50.2	81.7
8	59.5	47.6	50.4	51.0	52.0	53.1	50.2	81.7
9	58.5	46.8	49.1	49.7	50.6	51.7	50.2	81.7
10	58.7	47.0	49.1	49.6	50.4	51.5	50.2	81.7
11	60.8	49.2	51.1	51.5	52.3	53.3	50.2	81.7
12	57.7	46.1	47.7	48.2	48.8	49.8	50.2	81.7
13	56.3	44.7	46.3	46.6	47.3	48.2	50.2	81.7
14	55.7	44.1	45.5	45.9	46.5	47.3	50.2	81.7
15	55.5	43.9	45.2	45.5	46.1	46.9	50.2	81.7
16	56.0	44.4	45.6	45.9	46.4	47.2	50.2	81.7
17	58.3	46.6	47.7	48.0	48.5	49.3	50.2	81.7
18	55.2	43.4	44.5	44.8	45.2	46.0	50.2	81.7
19	53.8	42.0	43.0	43.3	43.7	44.4	50.2	81.7
20	53.1	41.1	42.1	42.3	42.7	43.4	50.2	81.7
21	52.6	40.5	41.4	41.6	42.0	42.7	50.2	81.7
22	52.3	40.0	40.9	41.1	41.5	42.1	50.2	81.7
23	52.1	39.6	40.5	40.7	41.1	41.7	50.2	81.7
24	52.0	39.4	40.2	40.4	40.8	41.4	50.2	81.7

Table 4.24b: FEA Summary Results for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

FEA Results		No Railing	Railing Deterioration Width				
			0 ft	1 ft	2 ft	4 ft	8 ft
Edge Beam Moment (Kip-ft/ft)	At Center	65.8	51.9	76.9	80.1	69.6	64.5
	At Deterioration	65.8	51.9	76.9	70.4	65.1	60.4
Maximum Longitudinal Moment (Kip-ft/ft)		62.3	50.4	53.6	54.4	55.4	56.5
Maximum Live Load Deflection (in)		0.423	0.329	0.342	0.345	0.350	0.358

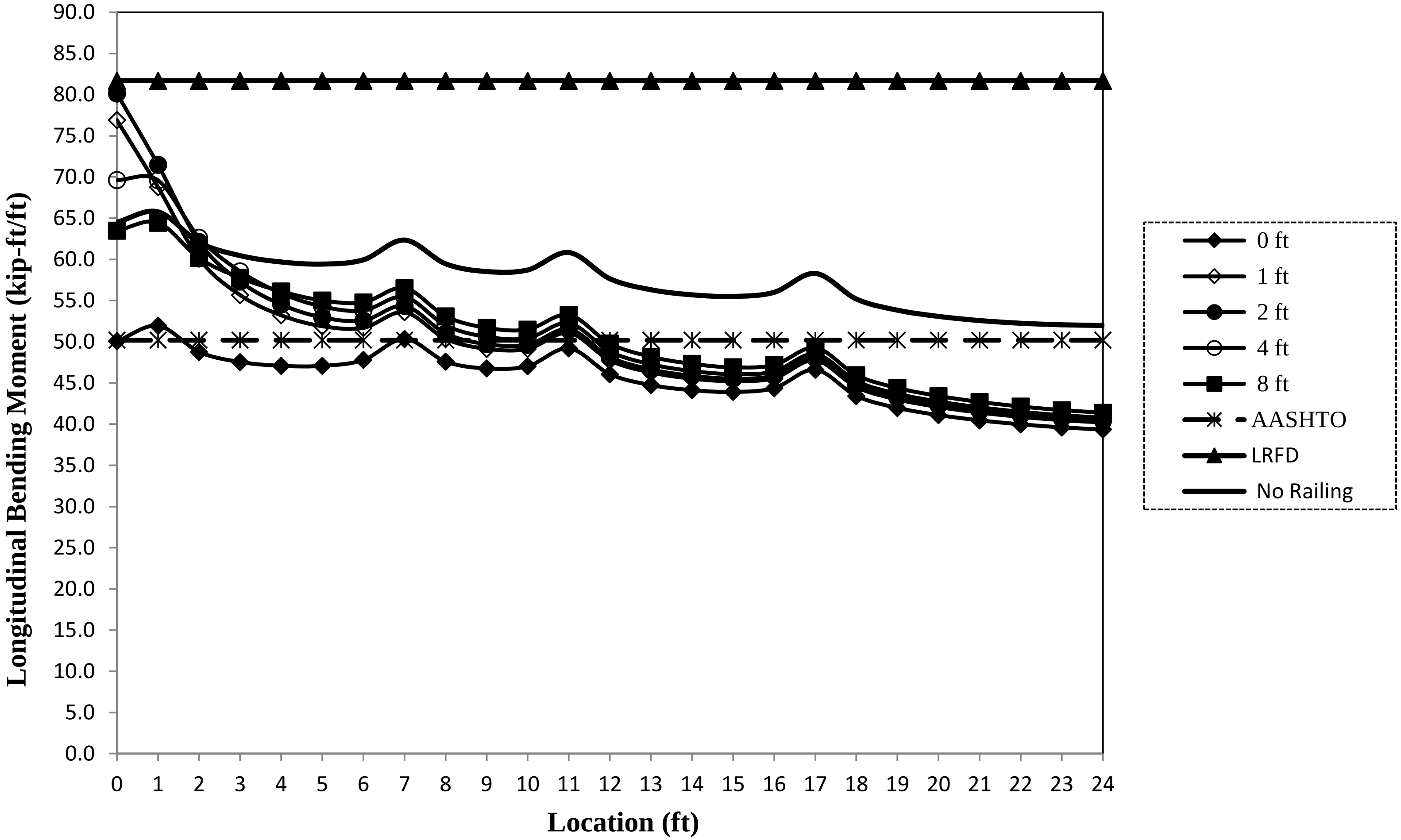


Figure 4.24: Longitudinal Moment Distribution at Critical Section for Two-Lane Single Span Bridge – Deck Span = 54 ft, Deck Width = 24 ft, Two Railings with Edge Loading E1, Half Depth Railing Deterioration

4.3 Summary and Comparison of Results with AASHTO and Reference Cases

4.3.1 Comparison of FEA Results with AASHTO

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in tables for each bridge category with its reference bridge cases of "Category R0-Ref", "Category R1-Ref" and "Category R2-Ref". The results are compared in terms of percentage difference along with the corresponding AASHTO bending moments and deflections.

For bridges with one railing the results are tabulated in Tables 4.25 to 4.29 for "Category R1-Full" and in Tables 4.30 to 4.34 for "Category R1-Half".

For bridges with two railings the results are tabulated in Tables 4.35 to 4.39 for "Category R2-Full" and in Tables 4.40 to 4.44 for "Category R2-Half".

4.3.1.1 Bridges with No Railings "Category R0-Ref"

Using Tables 4.25 and 4.26, it can be observed that, for bridge cases with no railings, AASHTO Standard Specifications generally tends to give similar results to the FEA moments, with the exception of one-lane with 36 ft span where AASHTO overestimates FEA moments by about 15%. This is more pronounced with two lanes and longer span, where AASHTO underestimates FEA moments reaching up to 20% for slab moments and 15% for edge beam moments for two lanes with 54 ft span.

Using Tables 4.27 and 4.28, it can be observed that AASHTO LRFD overestimates the FEA moments in all bridge cases with no railing. AASHTO LRFD overestimates the slab moments and the edge beam moment for one-lane bridges by about 50% and 30% and for two-lane bridges by about 30 % and 15%, respectively.

As shown in Table 4.29, the AASHTO estimated deflections range between 2 - 3 that of the FEA deflections for bridges with no railings.

4.3.1.2 Bridges with One Railing without Deterioration “Category R1-Ref”

Tables 4.25 and 4.26 show that the FEA moments for bridge cases with one railing without deterioration are similar to the cases with no railings, and therefore no significant changes in the FEA moments is observed, with the exception of one-lane bridges where the overestimation by AASHTO reaches 40% for 36 ft span down to 20% for 54 ft span.

Tables 4.27 and 4.28 show that AASHTO LRFD overestimates the slab moments by about 90% for one-lane bridges and 40% for two-lane bridges, and overestimates the edge beam moments by about 50% for one-lane and 20% for two-lane bridges.

As shown in Table 4.29, the AASHTO estimated deflections range between 2 - 4 of the FEA deflections for bridges with one railing.

4.3.1.3 Bridges with One Railing and Full Deterioration “Category R1-Full”

Using Tables 4.25 to 4.28, it can be observed that the FEA slab moments for bridge cases with one railing full-deteriorated compares similarly to AASHTO Standard Specifications as the cases with no railings, with the exception of one-lane bridges where the overestimation by AASHTO reaches 20% for 36 ft span for 1 ft deteriorated width, and decreases to about 10% as the deterioration width increases to 8 ft. However, the FEA edge beam moments changed significantly from being overestimated or similar to AASHTO in the case of no railing to being underestimated by AASHTO when the railing is deteriorated. The underestimation by AASHTO for the FEA edge beam moments reaches about 25% for one-lane and 40% for two-lane bridges when the railing deterioration width is 1 ft. The underestimation decreases as the deterioration width increases up to 8 ft where AASHTO gives similar results to

FEA edge beam moments, for one-lane, but still significantly underestimates them for two-lanes by about 15% for 36 ft span and 30% for 54 ft span.

Using Tables 4.27 and 4.28, it is observed that AASHTO LRFD continues to overestimate the FEA slab moments in bridge cases when the railing is full-deteriorated. The slab moments are overestimated by about 60 % for one-lane bridges and 35% for two-lane bridges for all deterioration widths. However, the edge beam moments changed significantly to be underestimated by AASHTO LRFD when the railing is full-deteriorated, except for the case of one-lane and 54 ft span where AASHTO LRFD gives similar results. For one-lane bridges with 36 ft span, AASHTO LRFD underestimates the edge beam moments by about 20% for deterioration width of 1 ft and gives similar results when the deterioration width is greater than 2 ft. For one-lane bridges with 54 ft span, AASHTO LRFD gives similar results when the deterioration width is less than 4 ft and overestimates the edge beam moment when it is greater than 4 ft. For two-lane bridges, AASHTO LRFD underestimates the edge beam moments by about 35% for 36 ft span and 20% for 54 ft span when the deterioration width is 1 ft, this underestimation decreases as the deterioration width increases where AASHTO LRFD gives similar results for deterioration width greater than 4 ft.

Tables 4.29 shows that the AASHTO estimated deflections still range between 2 - 4 of the FEA deflections when the railing is full-deteriorated.

Table 4.25: Comparison of FEA Maximum Longitudinal Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	11%	23.0	41%	27.0	20%	27.5	18%	28.2	15%	28.9	12%	32.4
	54	52.2	-4%	44.3	13%	49.4	2%	50.1	0%	51.0	-2%	51.9	-3%	50.2
2	36	35.3	-8%	31.9	2%	33.7	-4%	34.3	-6%	35.1	-8%	35.7	-9%	32.4
	54	62.3	-19%	57.2	-12%	61.0	-18%	61.7	-19%	62.6	-20%	63.3	-21%	50.2

Table 4.26: Comparison of FEA Edge Beam Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	17%	27.4	40%	50.7	-24%	47.0	-18%	41.6	-8%	36.2	6%	38.4
	54	55.9	3%	48.4	19%	77.4	-26%	74.3	-22%	69.3	-17%	64.9	-11%	57.6
2	36	38.5	0%	35.6	8%	63.9	-40%	59.5	-35%	52.9	-27%	45.9	-16%	38.4
	54	65.8	-12%	61.0	-6%	96.5	-40%	92.9	-38%	86.9	-34%	81.2	-29%	57.6

Table 4.27: Comparison of FEA Maximum Longitudinal Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	62%	23.0	106%	27.0	75%	27.5	72%	28.2	68%	28.9	64%	47.2
	54	52.2	44%	44.3	70%	49.4	52%	50.1	50%	51.0	48%	51.9	45%	75.3
2	36	35.3	29%	31.9	43%	33.7	35%	34.3	33%	35.1	30%	35.7	28%	45.6
	54	62.3	31%	57.2	43%	61.0	34%	61.7	32%	62.6	31%	63.3	29%	81.7

Table 4.28: Comparison of FEA Edge Beam Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	21%	27.4	44%	50.7	-22%	47.0	-16%	41.6	-5%	36.2	9%	39.5
	54	55.9	36%	48.4	57%	77.4	-2%	74.3	3%	69.3	10%	64.9	17%	76.1
2	36	38.5	7%	35.6	16%	63.9	-35%	59.5	-30%	52.9	-22%	45.9	-10%	41.4
	54	65.8	22%	61.0	31%	96.5	-17%	92.9	-14%	86.9	-8%	81.2	-2%	79.9

Table 4.29: Comparison of FEA Maximum Live Load Deflection with AASHTO Criterion

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												AASHTO Deflection (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	201%	0.143	276%	0.158	242%	0.160	238%	0.163	231%	0.169	219%	0.540
	54	0.352	130%	0.298	171%	0.313	158%	0.316	157%	0.319	154%	0.327	148%	0.810
2	36	0.219	147%	0.199	171%	0.207	160%	0.208	159%	0.210	157%	0.213	153%	0.540
	54	0.423	91%	0.389	108%	0.398	103%	0.400	103%	0.402	101%	0.407	99%	0.810

4.3.1.4 Bridges with One Railing and Half Deterioration "Category R1-Half"

Using Tables 4.30 and 4.31, it can be observed that the FEA slab moments for bridge cases with one railing half-deteriorated compares similarly to AASHTO Standard Specifications as the cases with no railing, with the exception of one-lane bridges where the overestimation by AASHTO reaches 30% for 36 ft span for 1 ft deteriorated width, and decreases to about 20% as the deterioration width increases to 8 ft. The FEA edge beam moments changed significantly from being overestimated or similar to AASHTO in the case of no railing to being underestimated by AASHTO when the railing is deteriorated except for the case of one-lane bridges with 36 ft span. The underestimation by AASHTO for the FEA edge beam moments reaches about 5% to 15% for one-lane and 30% for two-lane bridges when the railing deterioration width is 2 ft or less. The underestimation decreases as the deterioration width increases up to 8 ft where AASHTO gives similar results to FEA edge beam moments for one-lane bridges with 54 ft span and two-lane bridges with 36 ft span. For the case of one-lane with 36 ft span AASHTO overestimates the edges beam moments by up to 20% when the deterioration width reaches 8 ft, but still significantly underestimates them for two-lanes with 54 ft span by about 15%.

Using Tables 4.32 and 4.33, it can be observed that AASHTO LRFD continues to overestimate the FEA slab moments in bridge cases when the railing is half- deteriorated. The slab moments are overestimated by about 70 % for one-lane bridges and 40% for two-lane bridges almost for all deterioration widths. For the FEA edge beam moments, the overestimation by AASHTO LRFD decreases as the deterioration width increases. For one-lane bridges with 36 ft span, AASHTO LRFD gives similar results for the edge beam moments when the deterioration width is less than 4 ft and overestimates the edge beam moment by about 20% when the deterioration width is greater than 4 ft. For one-lane bridges with 54 ft span, AASHTO LRFD overestimates the edge beam moment by about 15% reaching up to 30% as the deterioration width increases up to 8 ft . For two-lane bridges, AASHTO LRFD underestimates the edge beam moments by about 20% for 36 ft span and 5% for 54 ft span when the deterioration width is less than 2 ft, this underestimation changes to overestimation when the deterioration width reaches 8 ft.

Tables 4.34 shows that the AASHTO estimated deflections still range between 2 - 4 of the FEA deflections when the railing is half-deteriorated.

Table 4.30: Comparison of FEA Maximum Longitudinal Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	11%	23.0	41%	25.0	30%	25.5	27%	26.3	23%	27.3	19%	32.4
	54	52.2	-4%	44.3	13%	47.2	6%	47.9	5%	48.9	3%	50.2	0%	50.2
2	36	35.3	-8%	31.9	2%	32.7	-1%	32.9	-2%	33.3	-3%	33.8	-4%	32.4
	54	62.3	-19%	57.2	-12%	58.3	-14%	59.1	-15%	60.3	-17%	61.5	-18%	50.2

Table 4.31: Comparison of FEA Edge Beam Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	17%	27.4	40%	38.1	1%	40.8	-6%	34.8	10%	32.1	20%	38.4
	54	55.9	3%	48.4	19%	65.8	-12%	68.6	-16%	60.7	-5%	56.9	1%	57.6
2	36	38.5	0%	35.6	8%	48.5	-21%	51.8	-26%	43.1	-11%	39.1	-2%	38.4
	54	65.8	-12%	61.0	-6%	82.5	-30%	85.9	-33%	74.8	-23%	69.3	-17%	57.6

Table 4.32: Comparison of FEA Maximum Longitudinal Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	62%	23.0	106%	25.0	89%	25.5	85%	26.3	79%	27.3	73%	47.2
	54	52.2	44%	44.3	70%	47.2	60%	47.9	57%	48.9	54%	50.2	50%	75.3
2	36	35.3	29%	31.9	43%	32.7	39%	32.9	39%	33.3	37%	33.8	35%	45.6
	54	62.3	31%	57.2	43%	58.3	40%	59.1	38%	60.3	35%	61.5	33%	81.7

Table 4.33: Comparison of FEA Edge Beam Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	21%	27.4	44%	38.1	4%	40.8	-3%	34.8	14%	32.1	23%	39.5
	54	55.9	36%	48.4	57%	65.8	16%	68.6	11%	60.7	25%	56.9	34%	76.1
2	36	38.5	7%	35.6	16%	48.5	-15%	51.8	-20%	43.1	-4%	39.1	6%	41.4
	54	65.8	22%	61.0	31%	82.5	-3%	85.9	-7%	74.8	7%	69.3	15%	79.9

Table 4.34: Comparison of FEA Maximum Live Load Deflection with AASHTO Criterion

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												AASHTO Deflection (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	201%	0.143	276%	0.151	259%	0.152	254%	0.156	246%	0.161	235%	0.540
	54	0.352	130%	0.298	171%	0.307	164%	0.309	162%	0.312	159%	0.319	154%	0.810
2	36	0.219	147%	0.199	171%	0.203	165%	0.204	164%	0.206	162%	0.209	158%	0.540
	54	0.423	91%	0.389	108%	0.394	106%	0.395	105%	0.398	104%	0.402	102%	0.810

4.3.1.5 Bridges with Two Railings without Deterioration "Category R2-Ref"

The presence of two railings provide stiffness for the bridge slab especially at its edges causing the FEA moments to be greatly overestimated by AASHTO.

As shown in Table 4.35, the FEA slab moments for bridge cases with two railings are overestimated by AASHTO standard specifications except for the case of two-lane bridges with 54 ft span. For one-lane bridges the overestimation by AASHTO reaches about 80% for 36 ft span down to 35% for 54 ft span. For two-lane bridges, AASHTO overestimates the slab moments by about 30% for 36 ft span bridges and gives similar results for 54 ft span bridges.

As shown in Table 4.36, the FEA edge beam moments decreases significantly due to the presence of two railings leading to higher overestimation by AASHTO reaching up to 100% for one-lane bridges and 50% for two-lane bridges.

Using Tables 4.37 and 4.38, it can be observed that AASHTO LRFD overestimates the slab moments by about 130% for one-lane bridges and 70% for two-lane bridges, and overestimates the edge beam moments by about 100% for one-lane and 60% for two-lane bridges.

As shown in Table 4.39, the AASHTO estimated deflections range between 2 -6 of the FEA deflections for bridges with two railings.

4.3.1.6 Bridges with Two Railings and Full Deterioration "Category R2-Full"

As shown in Tables 4.35 and 4.36, when one of the two railings is full-deteriorated the FEA slab moments increase. The overestimation by AASHTO Standard Specifications of the FEA slab moments goes down as the deterioration width increases. For one-lane bridges, the overestimation by AASHTO ranges between 50% (for 36 ft span) with 1 ft width of full-deterioration and 15% (for 54 ft span) when the deterioration width reaches up to 8 ft. For two-lane bridges, AASHTO is no more overestimating the slab moments but it gives similar results for 36 ft span and underestimates the slab moment by about 10% and 15% for 1 ft and 8 ft of deterioration widths, respectively. However, the FEA edge beam moments increased significantly and changed from being overestimated by AASHTO in the case of non-deteriorated railing to being underestimated by AASHTO when the railing is full-deteriorated. The underestimation by AASHTO for the FEA edge beam moments reaches about 15% for one-lane bridges and 35% for two-lane bridges when the railing deterioration width is 1 ft. The underestimation decreases as the deterioration width increases up to 8 ft where AASHTO gives similar results or overestimates the FEA edge beam moments, for one-lane bridges, but still significantly underestimates the edge beam moments for two-lane bridges by about 10% for 36 ft span and 25% for 54 ft span.

Using Tables 4.37 and 4.38, it can be observed that AASHTO LRFD continues to overestimate the FEA slab moments in bridge cases when one of the railings is full-deteriorated. The slab moments are overestimated by about 100 % for one-lane bridges and 45% for two-lane bridges for all deterioration widths. However, the edge beam moments changed significantly to be underestimated by AASHTO LRFD when the railing is full-deteriorated by 1 ft, except for the case of one-lane and

54 ft span. For one-lane bridges with 36 ft span, AASHTO LRFD gives similar results when the deterioration width is less than 4 ft and overestimates the edge beam moment by 15% to 35% when it is greater than 4 ft. For one-lane bridges with 54 ft span, AASHTO LRFD overestimates the edge beam moment by about 15% to 35% as the deterioration width increases from 1 ft to 8 ft. For two-lane bridges, AASHTO LRFD underestimates the edge beam moments by about 30% for 36 ft span and 15% for 54 ft span when the deterioration width is 1 ft, this underestimation decreases as the deterioration width increases where AASHTO LRFD gives similar results for deterioration width greater than 4 ft.

Table 4.39 shows that the AASHTO estimated deflections range between 2 - 5 of the FEA deflections when the railing is full-deteriorated.

Table 4.35: Comparison of FEA Maximum Longitudinal Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	11%	17.7	83%	21.5	50%	22.0	47%	22.6	44%	23.0	41%	32.4
	54	52.2	-4%	37.3	35%	42.1	19%	42.7	17%	43.5	15%	44.3	13%	50.2
2	36	35.3	-8%	25.6	27%	30.6	6%	31.1	4%	31.8	2%	32.3	0%	32.4
	54	62.3	-19%	50.4	0%	56.1	-11%	56.8	-12%	57.6	-13%	58.2	-14%	50.2

Table 4.36: Comparison of FEA Edge Beam Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	17%	19.2	100%	43.1	-11%	39.6	-3%	34.6	11%	29.5	30%	38.4
	54	55.9	3%	39.6	45%	68.3	-16%	65.2	-12%	60.4	-5%	56.2	3%	57.6
2	36	38.5	0%	25.5	50%	59.4	-35%	55.3	-31%	49.0	-22%	42.3	-9%	38.4
	54	65.8	-12%	51.9	11%	90.4	-36%	86.9	-34%	81.0	-29%	75.5	-24%	57.6

Table 4.37: Comparison of FEA Maximum Longitudinal Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	62%	17.7	166%	21.5	119%	22.0	115%	22.6	109%	23.0	105%	47.2
	54	52.2	44%	37.3	102%	42.1	79%	42.7	76%	43.5	73%	44.3	70%	75.3
2	36	35.3	29%	25.6	78%	30.6	49%	31.1	46%	31.8	43%	32.3	41%	45.6
	54	62.3	31%	50.4	62%	56.1	46%	56.8	44%	57.6	42%	58.2	40%	81.7

Table 4.38: Comparison of FEA Edge Beam Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	21%	19.2	106%	43.1	-8%	39.6	0%	34.6	14%	29.5	34%	39.5
	54	55.9	36%	39.6	92%	68.3	12%	65.2	17%	60.4	26%	56.2	35%	76.1
2	36	38.5	7%	25.5	62%	59.4	-30%	55.3	-25%	49.0	-15%	42.3	-2%	41.4
	54	65.8	22%	51.9	54%	90.4	-12%	86.9	-8%	81.0	-1%	75.5	6%	79.9

Table 4.39: Comparison of FEA Maximum Live Load Deflection with AASHTO Criterion

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												AASHTO Deflection (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	201%	0.095	470%	0.120	350%	0.123	339%	0.128	323%	0.134	304%	0.540
	54	0.352	130%	0.241	236%	0.262	209%	0.264	207%	0.269	202%	0.276	194%	0.810
2	36	0.219	147%	0.137	295%	0.171	216%	0.175	209%	0.180	199%	0.188	187%	0.540
	54	0.423	91%	0.329	146%	0.353	130%	0.355	128%	0.360	125%	0.368	120%	0.810

4.3.1.7 Bridges with Two Railings and Half Deterioration "Category R2-Half"

As shown in Tables 4.40 and 4.41, when one of the two railings is half-deteriorated the FEA slab moments increase. The overestimation by AASHTO Standard Specifications of the FEA slab moments goes down as the deterioration width increases. For one-lane bridges, the overestimation by AASHTO ranges between 65% (for 36 ft span) with 1 ft width of full-deterioration and about 20% (for 54 ft span) when the deterioration width reaches up to 8 ft. For two-lane bridges with 36 ft span AASHTO is overestimating the slab moments by about 15% when the deterioration width is less than 2 ft, and gives similar results for deterioration width greater than 2 ft. For two-lane bridges with 54 ft span AASHTO gives similar results for deterioration width less than 8 ft and underestimates the slab moments by about 10% for deterioration width of 8 ft. However, the FEA edge beam moments increased significantly when the railing is half-deteriorated and the overestimation by AASHTO changed to underestimation in the two-lane bridge cases. For one-lane bridges, AASHTO keeps on overestimating the FEA edge beam moments by about 10% to 40% for 36 ft span and almost gives similar results for 54 ft span. For two-lane bridges AASHTO significantly underestimates the edge beam moments by about 15% to 20% for 36 ft span and 25% to 30% for 54 ft span when the deterioration width is less than 4 ft. As the deterioration width increases to 8 ft, the underestimation by AASHTO decreases to 10% for 54 ft span bridges and changes to give similar results for 36 ft span bridges.

Using Tables 4.42 and 4.43, it can be observed that AASHTO LRFD continues to overestimate the FEA slab moments in bridge cases when one of the railings is half-deteriorated. The slab moments are overestimated by about 110 % for one-lane bridges and about 50% for two-lane bridges for all deterioration widths. The overestimation by AASHTO LRFD for the FEA edge beam moments decreases significantly when the railing is half-deteriorated. For one-lane bridges, AASHTO LRFD overestimates the edge beam moment by 15% to 50% for different deterioration widths. For two-lane bridges, AASHTO LRFD gives similar results to the FEA edge beam moments when the deterioration width is less than 4 ft except for the case of 36 ft span with 2 ft width of deterioration. For deterioration width greater than 4 ft, AASHTO LRFD goes back to overestimate the FEA edge beam moments by 15% to 25%.

Table 4.44 shows that the AASHTO estimated deflections still range between 2 - 5 of the FEA deflections when the railing is half-deteriorated.

Table 4.40: Comparison of FEA Maximum Longitudinal Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	11%	17.7	83%	19.7	65%	20.2	61%	20.9	55%	21.6	50%	32.4
	54	52.2	-4%	37.3	35%	40.0	26%	40.6	24%	41.6	21%	42.6	18%	50.2
2	36	35.3	-8%	25.6	27%	28.2	15%	28.8	12%	29.7	9%	30.6	6%	32.4
	54	62.3	-19%	50.4	0%	53.6	-6%	54.4	-8%	55.4	-9%	56.5	-11%	50.2

Table 4.41: Comparison of FEA Edge Beam Moment with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	17%	19.2	100%	32.4	19%	34.6	11%	29.6	30%	27.1	41%	38.4
	54	55.9	3%	39.6	45%	57.7	0%	60.2	-4%	53.1	8%	49.7	16%	57.6
2	36	38.5	0%	25.5	50%	45.0	-15%	48.1	-20%	40.0	-4%	36.3	6%	38.4
	54	65.8	-12%	51.9	11%	76.9	-25%	80.1	-28%	69.6	-17%	64.5	-11%	57.6

Table 4.42: Comparison of FEA Maximum Longitudinal Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	62%	17.7	166%	19.7	140%	20.2	134%	20.9	126%	21.6	119%	47.2
	54	52.2	44%	37.3	102%	40.0	88%	40.6	85%	41.6	81%	42.6	77%	75.3
2	36	35.3	29%	25.6	78%	28.2	62%	28.8	58%	29.7	53%	30.6	49%	45.6
	54	62.3	31%	50.4	62%	53.6	52%	54.4	50%	55.4	47%	56.5	45%	81.7

Table 4.43: Comparison of FEA Edge Beam Moment with LRFD Moment

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	21%	19.2	106%	32.4	22%	34.6	14%	29.6	34%	27.1	46%	39.5
	54	55.9	36%	39.6	92%	57.7	32%	60.2	26%	53.1	43%	49.7	53%	76.1
2	36	38.5	7%	25.5	62%	45.0	-8%	48.1	-14%	40.0	4%	36.3	14%	41.4
	54	65.8	22%	51.9	54%	76.9	4%	80.1	0%	69.6	15%	64.5	24%	79.9

Table 4.44: Comparison of FEA Maximum Live Load Deflection with AASHTO Criterion

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												AASHTO Deflection (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	201%	0.095	470%	0.107	402%	0.111	387%	0.116	367%	0.122	342%	0.540
	54	0.352	130%	0.241	236%	0.252	221%	0.255	218%	0.259	212%	0.266	204%	0.810
2	36	0.219	147%	0.137	295%	0.154	251%	0.158	241%	0.165	227%	0.174	211%	0.540
	54	0.423	91%	0.329	146%	0.342	137%	0.345	135%	0.350	131%	0.358	127%	0.810

4.3.2 Comparison of FEA Results with Reference Cases - No Railing

4.3.2.1 Bridges with One Railing and Full Deterioration "Category R1-Full"

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in terms of ratios in Tables 4.45 to 4.47 for bridges of "Category R1-Full" analyzed along with their corresponding bending moments MR0 and deflections ∆R0 of reference bridge cases with no Railing "Category R0-Ref".

Tables 4.45 to 4.47 show that the presence of one railing reduces the maximum slab moment, edge beam moment, and live-load deflection by about 15% to 20% for one-lane bridges and 10% for two-lane bridges. Once the railing is full-deteriorated by 1 ft, the reduction of the slab moments and deflections decreases to about 5% for one-lane and two-lane bridges and tends to be null when the deterioration width is greater than 4 ft; the results becomes similar to that of bridge cases with no railings. However, the full-deterioration of the railing causes a significant increase in the edge beam moments by about 45% for one-lane bridges and 55% for two-lane bridges for 1 ft of deterioration width. This increase in the edge beam moments still exist even when the deterioration width reaches 8 ft. The results shows an increase of 15% for one-lane bridges and 20% for two-lane bridges.

Table 4.45: FEA Maximum Longitudinal Bending Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.00	23.0	0.79	27.0	0.93	27.5	0.94	28.2	0.97	28.9	0.99	29.1
	54	52.2	1.00	44.3	0.85	49.4	0.95	50.1	0.96	51.0	0.98	51.9	0.99	52.2
2	36	35.3	1.00	31.9	0.90	33.7	0.95	34.3	0.97	35.1	0.99	35.7	1.01	35.3
	54	62.3	1.00	57.2	0.92	61.0	0.98	61.7	0.99	62.6	1.00	63.3	1.02	62.3

Table 4.46: FEA Edge Beam Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Beam Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	1.00	27.4	0.84	50.7	1.55	47.0	1.44	41.6	1.27	36.2	1.11	32.7
	54	55.9	1.00	48.4	0.87	77.4	1.38	74.3	1.33	69.3	1.24	64.9	1.16	55.9
2	36	38.5	1.00	35.6	0.93	63.9	1.66	59.5	1.54	52.9	1.37	45.9	1.19	38.5
	54	65.8	1.00	61.0	0.93	96.5	1.47	92.9	1.41	86.9	1.32	81.2	1.24	65.8

Table 4.47: FEA Maximum Live Load Deflection Ratios with ΔR0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔR0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.00	0.143	0.80	0.158	0.88	0.160	0.89	0.163	0.91	0.169	0.94	0.180
	54	0.352	1.00	0.298	0.85	0.313	0.89	0.316	0.90	0.319	0.91	0.327	0.93	0.352
2	36	0.219	1.00	0.199	0.91	0.207	0.95	0.208	0.95	0.210	0.96	0.213	0.97	0.219
	54	0.423	1.00	0.389	0.92	0.398	0.94	0.400	0.95	0.402	0.95	0.407	0.96	0.423

4.3.2.2 Bridges with One Railing and Half Deterioration "Category R1-Half"

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in terms of ratios in Tables 4.48 to 4.50 for bridges of "Category R1-Half" analyzed along with their corresponding bending moments MR0 and deflections ∆R0 of reference bridge cases with no Railing "Category R0-Ref".

Tables 4.48 to 4.50 show that the presence of one railing reduces the maximum slab moments, edge beam moments, and live-load deflections by about 15% to 20% for one-lane bridges and 10% for two-lane bridges. Once the railing is half-deteriorated by a width of 1 ft, the reduction of slab moments and deflections decreases to about 10% to 15% for one-lane bridges and 5% for two-lane bridges and tends to be null when the deterioration width is greater than 8 ft where the results become similar to that of bridge cases with no railings. However, the half-deterioration of the railing causes an increase in the edge beam moments by about 20% to 25% for one-lane bridges and 25% to 30% for two-lane bridges for deterioration width less than 2 ft. The edge beam moment in the bridges with half-deteriorated railing decreases as the deterioration width increases to become similar to that of no railing when the deterioration width is greater than 4 ft.

Table 4.48: FEA Maximum Longitudinal Bending Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.00	23.0	0.79	25.0	0.86	25.5	0.88	26.3	0.91	27.3	0.94	29.1
	54	52.2	1.00	44.3	0.85	47.2	0.90	47.9	0.92	48.9	0.94	50.2	0.96	52.2
2	36	35.3	1.00	31.9	0.90	32.7	0.93	32.9	0.93	33.3	0.94	33.8	0.96	35.3
	54	62.3	1.00	57.2	0.92	58.3	0.94	59.1	0.95	60.3	0.97	61.5	0.99	62.3

Table 4.49: FEA Edge Beam Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Beam Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	Th	32.7	1.00	27.4	0.84	38.1	1.16	40.8	1.25	34.8	1.06	32.1	0.98	32.7
	54	55.9	1.00	48.4	0.87	65.8	1.18	68.6	1.23	60.7	1.09	56.9	1.02	55.9
2	36	38.5	1.00	35.6	0.93	48.5	1.26	51.8	1.34	43.1	1.12	39.1	1.02	38.5
	54	65.8	1.00	61.0	0.93	82.5	1.25	85.9	1.31	74.8	1.14	69.3	1.05	65.8

Table 4.50: FEA Maximum Live Load Deflection Ratios with ΔR0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔR0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.00	0.143	0.80	0.151	0.84	0.152	0.85	0.156	0.87	0.161	0.90	0.180
	54	0.352	1.00	0.298	0.85	0.307	0.87	0.309	0.88	0.312	0.89	0.319	0.91	0.352
2	36	0.219	1.00	0.199	0.91	0.203	0.93	0.204	0.93	0.206	0.94	0.209	0.96	0.219
	54	0.423	1.00	0.389	0.92	0.394	0.93	0.395	0.93	0.398	0.94	0.402	0.95	0.423

4.3.2.3 Bridges with Two Railings and Full Deterioration "Category R2-Full"

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in terms of ratios in Tables 4.51 to 4.53, for bridges of "Category R2-Full" analyzed along with their corresponding bending moments MR0 and deflections ΔR0 of reference bridge cases with no Railing "Category R0-Ref".

Tables 4.51 to 4.53 show that the presence of two railings reduces the maximum slab moment, edge beam moment, and live-load deflection by about 30% to 40% for one-lane bridges and 20% to 30% for two-lane bridges.

In the case of bridges with two railings it is supposed that one of the railings is deteriorated and the trucks are placed to its side.

As shown in Tables 4.51 to 4.53 the full-deterioration of the railing causes the reduction of slab moments and deflections to go down to reach about 10% to 15% for one-lane bridges and 5% to 10% for two-lane bridges when the deterioration width reaches 8 ft. However, a significant increase in the edge beam moments is caused by the full-deterioration and the railing is no more providing any moment reduction. The edge beam moments increased by about 25% for one-lane bridges and 45% for two-lane bridges for 1 ft of deterioration. The edge beam moments decrease as the deterioration width increases. For one-lane bridges, the edge beam moments reach similar values to that of the cases with no railing when the deterioration width is greater than 4 ft. For two-lane bridges, the edge beam moments still show an increase of 10% to 15% even when the deterioration width reaches 8 ft.

Table 4.51: FEA Maximum Longitudinal Bending Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.00	17.7	0.61	21.5	0.74	22.0	0.76	22.6	0.77	23.0	0.79	29.1
	54	52.2	1.00	37.3	0.71	42.1	0.81	42.7	0.82	43.5	0.83	44.3	0.85	52.2
2	36	35.3	1.00	25.6	0.72	30.6	0.87	31.1	0.88	31.8	0.90	32.3	0.91	35.3
	54	62.3	1.00	50.4	0.81	56.1	0.90	56.8	0.91	57.6	0.92	58.2	0.93	62.3

Table 4.52: FEA Edge Beam Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	1.00	19.2	0.59	43.1	1.32	39.6	1.21	34.6	1.06	29.5	0.90	32.7
	54	55.9	1.00	39.6	0.71	68.3	1.22	65.2	1.17	60.4	1.08	56.2	1.01	55.9
2	36	38.5	1.00	25.5	0.66	59.4	1.54	55.3	1.44	49.0	1.27	42.3	1.10	38.5
	54	65.8	1.00	51.9	0.79	90.4	1.37	86.9	1.32	81.0	1.23	75.5	1.15	65.8

Table 4.53: FEA Maximum Live Load Deflection Ratios with ΔR0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔR0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.00	0.095	0.53	0.120	0.67	0.123	0.69	0.128	0.71	0.134	0.74	0.180
	54	0.352	1.00	0.241	0.68	0.262	0.74	0.264	0.75	0.269	0.76	0.276	0.78	0.352
2	36	0.219	1.00	0.137	0.62	0.171	0.78	0.175	0.80	0.180	0.82	0.188	0.86	0.219
	54	0.423	1.00	0.329	0.78	0.353	0.83	0.355	0.84	0.360	0.85	0.368	0.87	0.423

4.3.2.4 Bridges with Two Railings and Half Deterioration "Category R2-Half"

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in terms of ratios in Tables 4.54 to 4.56 for bridges of "Category R2-Half" analyzed along with their corresponding bending moments MR0 and deflections ΔR0 of reference bridge cases with no Railing "Category R0-Ref".

Tables 4.54 to 4.56 show that the presence of two railings reduces the maximum slab moment, edge beam moment, and live-load deflection by about 30% to 40% for one-lane bridges and 20% to 30% for two-lane bridges.

As mentioned before, it is supposed that one of the railings is deteriorated and the trucks are placed to its side.

As shown in Tables 4.54 to 4.56 the half-deterioration of the railing causes the reduction of slab moments and deflections to decrease to reach about 20% to 25% for one-lane bridges and 10% to 15% for two-lane bridges when the deterioration width reaches 8 ft. However, a significant increase in the edge beam moments is caused by the half-deterioration of the railing to exceed that of the bridge cases with no railing. The edge beam moments increased by about 10% for one-lane bridges and 25% for two-lane bridges with 2 ft width of deterioration. The edge beam moments decrease as the deterioration width increases until they reach similar values to that of no railings when the deterioration width is greater than 2 ft for one-lane bridges and greater than 4 ft for two-lane bridges.

Table 4.54: FEA Maximum Longitudinal Bending Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.00	17.7	0.61	19.7	0.68	20.2	0.69	20.9	0.72	21.6	0.74	29.1
	54	52.2	1.00	37.3	0.71	40.0	0.77	40.6	0.78	41.6	0.80	42.6	0.82	52.2
2	36	35.3	1.00	25.6	0.72	28.2	0.80	28.8	0.82	29.7	0.84	30.6	0.87	35.3
	54	62.3	1.00	50.4	0.81	53.6	0.86	54.4	0.87	55.4	0.89	56.5	0.91	62.3

Table 4.55: FEA Edge Beam Moment and Ratios with MR0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	1.00	19.2	0.59	32.4	0.99	34.6	1.06	29.6	0.90	27.1	0.83	32.7
	54	55.9	1.00	39.6	0.71	57.7	1.03	60.2	1.08	53.1	0.95	49.7	0.89	55.9
2	36	38.5	1.00	25.5	0.66	45.0	1.17	48.1	1.25	40.0	1.04	36.3	0.94	38.5
	54	65.8	1.00	51.9	0.79	76.9	1.17	80.1	1.22	69.6	1.06	64.5	0.98	65.8

Table 4.56: FEA Maximum Live Load Deflection Ratios with ΔR0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔR0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.00	0.095	0.53	0.107	0.60	0.111	0.62	0.116	0.64	0.122	0.68	0.180
	54	0.352	1.00	0.241	0.68	0.252	0.72	0.255	0.72	0.259	0.74	0.266	0.76	0.352
2	36	0.219	1.00	0.137	0.62	0.154	0.70	0.158	0.72	0.165	0.75	0.174	0.79	0.219
	54	0.423	1.00	0.329	0.78	0.342	0.81	0.345	0.82	0.350	0.83	0.358	0.85	0.423

4.3.3 Comparison of FEA Results with Reference Cases - Without Deterioration

4.3.3.1 <u>Bridges with One Railing and Full Deterioration “Category R1-Full”</u>

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in terms of ratios in Tables 4.57 to 4.59 for bridges of “Category R1-Full” analyzed along with their corresponding bending moments MD0 and deflections ΔD0 of reference bridge cases with railing without deterioration “Category R1-Ref”.

Tables 4.57 to 4.59 show that the stiffening effect provided by the railing decreases as the deterioration width increases. The slab moments and deflections increased by 10% to 15% for one-lane bridges and 5% for two-lane bridges when the railing is full-deteriorated by 1 ft. The slab moments and deflections continued to increase as the deterioration width increased to reach about 20% for one-lane bridges and 10% for two-lane bridges when the railing is full-deteriorated by 8 ft, then the results become similar to that of no railings. The edge beam moments shows drastic change when compared to that of non-deteriorated railing. The edge beam moment increases significantly by about 75% for one-lane and two-lane bridges when the railing is deteriorated by 1 ft. This increase reduces as the deterioration width increases to reach about 30% for one-lane and two-lane bridges when the railing is deteriorated by 8 ft.

Table 4.57: FEA Maximum Longitudinal Bending Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.27	23.0	1.00	27.0	1.17	27.5	1.20	28.2	1.23	28.9	1.26	23.0
	54	52.2	1.18	44.3	1.00	49.4	1.12	50.1	1.13	51.0	1.15	51.9	1.17	44.3
2	36	35.3	1.11	31.9	1.00	33.7	1.06	34.3	1.08	35.1	1.10	35.7	1.12	31.9
	54	62.3	1.09	57.2	1.00	61.0	1.07	61.7	1.08	62.6	1.10	63.3	1.11	57.2

Table 4.58: FEA Edge Beam Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Beam Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	1.20	27.4	1.00	50.7	1.85	47.0	1.72	41.6	1.52	36.2	1.32	27.4
	54	55.9	1.15	48.4	1.00	77.4	1.60	74.3	1.53	69.3	1.43	64.9	1.34	48.4
2	36	38.5	1.08	35.6	1.00	63.9	1.79	59.5	1.67	52.9	1.49	45.9	1.29	35.6
	54	65.8	1.08	61.0	1.00	96.5	1.58	92.9	1.52	86.9	1.42	81.2	1.33	61.0

Table 4.59: FEA Maximum Live Load Deflection – Ratio with ΔD0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔD0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.25	0.143	1.00	0.158	1.10	0.160	1.11	0.163	1.14	0.169	1.18	0.1
	54	0.352	1.18	0.298	1.00	0.313	1.05	0.316	1.06	0.319	1.07	0.327	1.09	0.3
2	36	0.219	1.10	0.199	1.00	0.207	1.04	0.208	1.05	0.210	1.05	0.213	1.07	0.2
	54	0.423	1.09	0.389	1.00	0.398	1.03	0.400	1.03	0.402	1.04	0.407	1.05	0.4

4.3.3.2 Bridges with One Railing and Half Deterioration "Category R1-Half"

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in terms of ratios in Tables 4.60 to 4.62 for bridges of "Category R1-Half" analyzed along with their corresponding bending moments MD0 and deflections ΔD0 of reference bridge cases with railing without deterioration "Category R1-Ref".

Tables 4.60 to 4.62 show that the stiffening effect provided to the bridge slab by the railing decreases slightly when the railing is half-deteriorated. The increase in the slab moments and deflections ranges between 5% and 20% for one-lane bridges and between 2% and 8% for two-lane bridges as the deterioration width increases form 1 ft to 8 ft. The edge beam moments show significant change when compared to that of non-deteriorated railing. The edge beam moment increases significantly by about 45% for one-lane and two-lane bridges when the railing is deteriorated by 2 ft. This increase reduces as the deterioration width increases to reach about 15% for one-lane and two-lane bridges when the railing is deteriorated by 8 ft.

Table 4.60: FEA Maximum Longitudinal Bending Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.27	23.0	1.00	25.0	1.09	25.5	1.11	26.3	1.15	27.3	1.19	23.0
	54	52.2	1.18	44.3	1.00	47.2	1.06	47.9	1.08	48.9	1.10	50.2	1.13	44.3
2	36	35.3	1.11	31.9	1.00	32.7	1.02	32.9	1.03	33.3	1.04	33.8	1.06	31.9
	54	62.3	1.09	57.2	1.00	58.3	1.02	59.1	1.03	60.3	1.06	61.5	1.08	57.2

Table 4.61: FEA Edge Beam Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Beam Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	1.20	27.4	1.00	38.1	1.39	40.8	1.49	34.8	1.27	32.1	1.17	27.4
	54	55.9	1.15	48.4	1.00	65.8	1.36	68.6	1.42	60.7	1.25	56.9	1.18	48.4
2	36	38.5	1.08	35.6	1.00	48.5	1.36	51.8	1.45	43.1	1.21	39.1	1.10	35.6
	54	65.8	1.08	61.0	1.00	82.5	1.35	85.9	1.41	74.8	1.23	69.3	1.14	61.0

Table 4.62: FEA Maximum Live Load Deflection – Ratio with ΔD0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔD0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.25	0.143	1.00	0.151	1.05	0.152	1.06	0.156	1.09	0.161	1.12	0.1
	54	0.352	1.18	0.298	1.00	0.307	1.03	0.309	1.03	0.312	1.05	0.319	1.07	0.3
2	36	0.219	1.10	0.199	1.00	0.203	1.02	0.204	1.03	0.206	1.03	0.209	1.05	0.2
	54	0.423	1.09	0.389	1.00	0.394	1.01	0.395	1.02	0.398	1.02	0.402	1.03	0.4

4.3.3.3 Bridges with Two Railings and Full Deterioration "Category R2-Full"

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections are summarized in terms of ratios in Tables 4.63 to 4.65 for bridges of "Category R2-Full" analyzed along with their corresponding bending moments MD0 and deflections ∆D0 of reference bridge cases with railing without deterioration "Category R2-Ref".

Tables 4.63 and 4.64 show that the stiffening effect provided by the railing decreases as the deterioration width increases. The slab moments and deflections increased by about 10% to 25% for one-lane and two-lane bridges when the railing is full-deteriorated by 1 ft. The slab moments and deflections continued to increase as the deterioration width increased to reach about 15% to 30% for one-lane and two-lane bridges when the railing is full-deteriorated by 8 ft.

Table 4.65 shows that the full deterioration of one of the railings causes drastic change in the edge beam moments when compared to that of non-deteriorated railing. The edge beam moment increased significantly by about 75% to 125% for one-lane and two-lane bridges when the railing is full-deteriorated by 1 ft. This increase reduces as the deterioration width increases to reach about 40% to 60% for one-lane and two-lane bridges when the railing is full-deteriorated by 8 ft.

Table 4.63: FEA Maximum Longitudinal Bending Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.64	17.7	1.00	21.5	1.21	22.0	1.24	22.6	1.27	23.0	1.30	17.7
	54	52.2	1.40	37.3	1.00	42.1	1.13	42.7	1.15	43.5	1.17	44.3	1.19	37.3
2	36	35.3	1.38	25.6	1.00	30.6	1.20	31.1	1.22	31.8	1.24	32.3	1.26	25.6
	54	62.3	1.24	50.4	1.00	56.1	1.11	56.8	1.13	57.6	1.14	58.2	1.16	50.4

Table 4.64: FEA Edge Beam Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	1.70	19.2	1.00	43.1	2.24	39.6	2.06	34.6	1.80	29.5	1.53	19.2
	54	55.9	1.41	39.6	1.00	68.3	1.72	65.2	1.65	60.4	1.53	56.2	1.42	39.6
2	36	38.5	1.51	25.5	1.00	59.4	2.33	55.3	2.17	49.0	1.92	42.3	1.66	25.5
	54	65.8	1.27	51.9	1.00	90.4	1.74	86.9	1.67	81.0	1.56	75.5	1.45	51.9

Table 4.65: FEA Maximum Live Load Deflection – Ratio with ΔD0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔD0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.89	0.095	1.00	0.120	1.27	0.123	1.30	0.128	1.35	0.134	1.41	0.095
	54	0.352	1.46	0.241	1.00	0.262	1.09	0.264	1.10	0.269	1.11	0.276	1.14	0.241
2	36	0.219	1.60	0.137	1.00	0.171	1.25	0.175	1.28	0.180	1.32	0.188	1.38	0.137
	54	0.423	1.29	0.329	1.00	0.353	1.07	0.355	1.08	0.360	1.09	0.368	1.12	0.329

4.3.3.4 Bridges with Two Railings and Half Deterioration "Category R2-Half"

The maximum slab moments, edge beam longitudinal bending moments and maximum live load deflections are summarized in terms of ratios in Tables 4.66 to 4.68 for bridges of "Category R2-Half" analyzed along with their corresponding bending moments MD0 and deflections ΔD0 of reference bridge cases with railing without deterioration "Category R2-Ref".

Tables 4.66 and 4.68 show that the stiffening effect provided by the railing decreases as the deterioration width increases. The slab moments and deflections increased by 5% to 15% for one-lane and two-lane bridges when the railing is half-deteriorated by 1 ft. The slab moments and deflections continued to increase as the deterioration width increased to reach about 10% to 25% for one-lane and two-lane bridges when the railing is half-deteriorated by 8 ft.

Table 4.67 shows that the full deterioration of one of the railings causes drastic change in the edge beam moments when compared to that of non-deteriorated railing. The edge beam moment increases significantly by about 50% to 75% for one-lane and two-lane bridges when the railing is half-deteriorated by 1 ft. This increase reduces as the deterioration width increases to reach about 25% to 40% for one-lane and two-lane bridges when the railing is half-deteriorated by 8 ft.

Table 4.66: FEA Maximum Longitudinal Bending Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	29.1	1.64	17.7	1.00	19.7	1.11	20.2	1.14	20.9	1.18	21.6	1.22	17.7
	54	52.2	1.40	37.3	1.00	40.0	1.07	40.6	1.09	41.6	1.12	42.6	1.14	37.3
2	36	35.3	1.38	25.6	1.00	28.2	1.10	28.8	1.13	29.7	1.16	30.6	1.20	25.6
	54	62.3	1.24	50.4	1.00	53.6	1.06	54.4	1.08	55.4	1.10	56.5	1.12	50.4

Table 4.67: FEA Edge Beam Moment – Ratio with MD0

Number of Lanes	Span Length (ft)	FEA Edge Beam Moment (Kip-ft/ft)												Edge Moment MD0 (Kip-ft/ft)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	32.7	1.70	19.2	1.00	32.4	1.68	34.6	1.80	29.6	1.54	27.1	1.41	19.2
	54	55.9	1.41	39.6	1.00	57.7	1.46	60.2	1.52	53.1	1.34	49.7	1.25	39.6
2	36	38.5	1.51	25.5	1.00	45.0	1.76	48.1	1.88	40.0	1.57	36.3	1.42	25.5
	54	65.8	1.27	51.9	1.00	76.9	1.48	80.1	1.54	69.6	1.34	64.5	1.24	51.9

Table 4.68: FEA Maximum Live Load Deflection – Ratio with ΔD0

Number of Lanes	Span Length (ft)	FEA Maximum Live Load Deflection (in)												Maximum Deflection ΔD0 (in)
		No Railing		Railing Deterioration Width										
				0 ft		1 ft		2 ft		4 ft		8 ft		
1	36	0.180	1.89	0.095	1.00	0.107	1.13	0.111	1.17	0.116	1.22	0.122	1.29	0.095
	54	0.352	1.46	0.241	1.00	0.252	1.05	0.255	1.06	0.259	1.08	0.266	1.10	0.241
2	36	0.219	1.60	0.137	1.00	0.154	1.13	0.158	1.16	0.165	1.21	0.174	1.27	0.137
	54	0.423	1.29	0.329	1.00	0.342	1.04	0.345	1.05	0.350	1.06	0.358	1.09	0.329

4.4 Overall Summary of Results

The maximum longitudinal bending slab moments and edge beam longitudinal bending moments are summarized for all One-Railing cases and Two-Railing cases in Tables 4.69 and 4.70, respectively, and both One- and Two-Railing cases comprehensively in Table 4.71. These tables present the ratio of the FEA moments with respect to: FEA R0-Ref, AASHTO Specs, and AASHTO LRFD moments. Discussion of those summary results are presented in the following sections.

4.4.1 One-Railing Bridge Cases

Referring to Table 4.69, the following can be deduced:

- The FEA maximum longitudinal bending slab moments are slightly reduced in the presence of one railing, when compared with the FEA reference moments with no railings (R0-Ref), and this insignificant reduction can therefore be ignored. This applies regardless of no deterioration, half deterioration, or full deterioration of the railings for all widths considered. The railing deterioration can therefore be ignored on the slab moment, as was the case with no deterioration.
- The FEA edge beam moments are slightly reduced in the presence of one railing, when compared with the FEA reference moments with no railings (R0-Ref), and this insignificant reduction can therefore be ignored. This applies specifically for the cases with no deterioration; however, this becomes significantly critical in case of half deterioration or full deterioration, when the edge beam moments increase by up to 25% and 65% when compared to the case with no railings, namely when the deterioration occurs over a short width, due to the stress concentration at the slab edge.

- For the same reasons as above, it can be deduced that the slab moments will slightly be affected in the presence of railings, in case of deterioration vs no deterioration when compared with AAHSTO Specs or LRFD slab moments. However, the edge beam moments for the deteriorated cases become critical when compared with AAHSTO Specs or LRFD edge beam moments.

4.4.2 Two-Railing Bridge Cases

Referring to Table 4.70, the following can be deduced:

- The FEA maximum longitudinal bending slab moments are significantly reduced in the presence of two railings, when compared with the FEA reference moments with no railings (R0-Ref), and this reduction varies between 40% to 20% for one- and two-lane bridge cases. This applies to the case with no deterioration, and this reduction becomes 30-25% for one lane and 15-5% for two lanes, in the case of half deterioration or full deterioration of the railings for all widths considered.
- The FEA edge beam moments are similarly reduced in the presence of two railing, when compared with the FEA reference moments with no railings (R0-Ref), and this reduction also varies between 40% to 20% for one- and two-lane bridge cases However, this applies specifically for the cases with no deterioration, and, as is the case with no railings, this becomes significantly critical in case of half deterioration or full deterioration, when the edge beam moments increase by up to 15% and 55% when compared to the case with no railings, namely when the deterioration occurs over a short width, due to the stress concentration at the slab edge.
- For the same reasons as above, it can be deduced that the slab moments become safer in case of deterioration vs no deterioration when compared with AASHTO Specs or LRFD moments. However, the edge beam moments for the deteriorated

cases become critical when compared with AASHTO Specs or LRFD edge beam moments.

4.4.3 One- and Two-Railing Bridge Cases

Referring to Table 4.71, and as per summaries of One- and Two-Railing Cases above, the following can be concluded:

- The FEA maximum longitudinal bending slab moments are generally less affected with half or full deterioration when compared with references cases no railings deterioration, and consequently when compared with AASHTO Specs and LRFD moments.
- The FEA edge beam moments are generally significantly increased with half or full deterioration when compared with references cases with no railings, or with no railings deterioration, and consequently these become critical when compared with AASHTO Specs and LRFD moments.
- It is therefore recommended to reinforce properly the edge beams at both sides of the slab deck, regardless of the presence of railings or not.

Table 4.69: ONE RAILING - Summary of FEA Longitudinal Slab and Edge Beam Moments: Ratios vs Ref-R0, AASHTO Specs and AASHTO LRFD

FEA Moment Category	ONE RAILING - FEA MOMENTS RATIOS					
	FEA/FEA-R0		FEA/AASHTO-Specs		FEA/AASHTO-LRFD	
	Slab	Edge Beam	Slab	Edge Beam	Slab	Edge Beam
R0-Ref (No Railing)	1.00	1.00	**0.90 to 1.25**	**0.85 to 1.15**	0.60 to 0.75	0.75 to 0.90
	all	all	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long
R1-Ref (No Deterioration)	0.80 to 0.90	0.85 to 0.95	**0.70 to 1.15**	0.70 to 1.05	0.50 to 0.70	0.65 to 0.85
	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long
R1-Half (Half Deterioration)	0.85 to 1.00	**1.35 to 1.05**	**0.75 to 1.20**	**1.45 to 1.20**	0.55 to 0.75	**1.15 to 0.95**
	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)
R1-Full (Full Deterioration	0.95 to 1.00	**1.65 to 1.25**	**0.85 to 1.25**	**1.65 to 1.40**	0.55 to 0.75	**1.55 to 1.10**
	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)

Table 4.70: TWO RAILINGS - Summary of FEA Longitudinal Slab and Edge Beam Moments: Ratios vs Ref-R0, AASHTO Specs and AASHTO LRFD

FEA Moment Category	TWO RAILINGS - FEA MOMENTS RATIOS					
	FEA/FEA-R0		FEA/AASHTO-Specs		FEA/AASHTO-LRFD	
	Slab	Edge Beam	Slab	Edge Beam	Slab	Edge Beam
R0-Ref (No Railing)	1.00	1.00	**0.90 to 1.25**	**0.85 to 1.15**	0.60 to 0.75	0.75 to 0.90
	all	all	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long
R2-Ref (No Deterioration)	0.60 to 0.80	0.60 to 0.80	0.55 to 1.00	0.50 to 0.90	0.40 to 0.60	0.50 to 0.65
	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long
R2-Half (Half Deterioration)	0.70 to 0.85	**1.25 to 1.00**	0.60 to 1.05	**1.35 to 1.10**	0.40 to 0.70	**1.10 to 0.90**
	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)
R2-Full (Full Deterioration	0.75 to 0.95	**1.55 to 1.15**	**0.65 to 1.15**	**1.55 to 1.30**	0.45 to 0.70	**1.45 to 1.00**
	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)

Table 4.71: ONE AND TWO RAILINGS - Summary of FEA Longitudinal Slab and Edge Beam Moments: Ratios vs Ref-R0, AASHTO Specs and AASHTO LRFD

FEA Moment Category	ONE AND TWO RAILINGS - FEA MOMENTS RATIOS					
	FEA/FEA-R0		FEA/AASHTO-Specs		FEA/AASHTO-LRFD	
	Slab	Edge Beam	Slab	Edge Beam	Slab	Edge Beam
R0-Ref (No Railing)	1.00	1.00	**0.90 to 1.25**	**0.85 to 1.15**	0.60 to 0.75	0.75 to 0.90
	all	all	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long
R1-Ref (No Deterioration)	0.80 to 0.90	0.85 to 0.95	**0.70 to 1.15**	0.70 to 1.05	0.50 to 0.70	0.65 to 0.85
R2-Ref (No Deterioration)	0.60 to 0.80	0.60 to 0.80	0.55 to 1.00	0.50 to 0.90	0.40 to 0.60	0.50 to 0.65
	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long	1lane-short to 2lane-long
R1-Half (Half Deterioration)	0.85 to 1.00	**1.35 to 1.05**	**0.75 to 1.20**	**1.45 to 1.20**	0.55 to 0.75	**1.15 to 0.95**
R2-Half (Half Deterioration)	0.70 to 0.85	**1.25 to 1.00**	0.60 to 1.05	**1.35 to 1.10**	0.40 to 0.70	**1.10 to 0.90**
	1ft (1lane-short) to 8ft (2lane-long)	2ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)
R1-Full (Full Deterioration	0.95 to 1.00	**1.65 to 1.25**	**0.85 to 1.25**	**1.65 to 1.40**	0.55 to 0.75	**1.55 to 1.10**
R2-Full (Full Deterioration	0.75 to 0.95	**1.55 to 1.15**	**0.65 to 1.15**	**1.55 to 1.30**	0.45 to 0.70	**1.45 to 1.00**
	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)	1ft (1lane-short) to 8ft (2lane-long)	1ft (2lane-short) to 8ft (2lane-long)

CHAPTER 5

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

4.1 Summary

In this research, the effect of deterioration of stiffening railings on reinforced concrete slab bridges was investigated using the finite-element method. The study considered geometrically distinct simply supported, one-span reinforced concrete slab bridge cases with various span lengths, number of lanes, transverse truck loading positions and railings configurations having different widths and depths of deterioration. The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections were generated from the finite element analysis and compared with AASHTO Standard Specifications and AASHTO Load Resistance Factor Design (LRFD) in addition to a direct comparison with the reference bridge cases, which are bridges with no railings and bridges with railings without deterioration.

A total number of one hundred twelve bridge cases was included in this study. Two span lengths are considered 36 ft and 54 ft with the slab thickness being 21 in and 27 in, respectively, each combined with one and two lane bridges with lane width of 12 ft. A railing is then placed on either or both edges of each bridge, to here, the reference cases are obtained. Then the full or half depth deterioration of railing was applied to each of the bridge cases with four different deterioration widths around the centerline of the bridge: 1 ft, 2 ft, 4 ft and 8 ft. The cross section of the standard full railing is 8 in width by 30 (2.5 ft) in depth.

The bridges were analyzed for the AASHTO HS20 design trucks that were positioned longitudinally and transversally, considering two edge loading conditions E1 and E2, in order to produce maximum bending moments.

The finite element analysis method was used to analyze the concrete slab bridges. The bridges were modeled and analyzed using SAP2000 software using square shell elements of size 1ft x 1ft for the slab and rectangular shell elements of size 1ft x 1.25ft for the railings. The deteriorated portion of the railing was modeled by assigning the corresponding shell element a zero stiffness. The supports were assigned simple supports at the piers. The wheel loads of the AASHTO HS-20 truck were applied as concentrated loads at the nodes.

The FEA longitudinal bending moments and deflections were extracted from SAP2000 output files for each of the bridge cases. The longitudinal bending moments at critical locations of the bridge slabs were tabulated and plotted along with AASHTO moments.

The bridge cases were then divided into 7 categories, including 3 categories for reference bridge cases with no railing, one railing, and two railings and 4 categories consisting of the cases with one railing and two railings each subdivided into 2 categories of half depth and full depth railing deterioration.

The maximum longitudinal slab moments, edge beam moments, and maximum live load deflections for each bridge category were summarized in tables along with the corresponding AASHTO results and reference bridge cases results. The FEA results are then compared to AASHTO Specs and AASHTO LRFD results in terms of percentage differences, and to the reference bridge cases results in terms of ratios.

4.2 Conclusions and Recommendations

This research evaluated the effect of deterioration of stiffening railings on the maximum longitudinal slab moments, edge beam moments, and maximum live load deflections of simply supported, one-span reinforced concrete slab bridges. The following final conclusions are drawn based on the results of this investigation for both One- and Two-Railing Cases above:

- The FEA maximum longitudinal bending slab moments are generally less affected with half or full deterioration when compared with references cases no railings deterioration, and consequently when compared with AASHTO Specs and LRFD moments.
- The FEA edge beam moments are generally significantly increased with half or full deterioration when compared with references cases with no railings, or with no railings deterioration, and consequently these become critical when compared with AASHTO Specs and LRFD moments.

It is therefore recommended to reinforce properly the edge beams at both sides of the slab deck, regardless of the presence of railings or not.

BIBLIOGRAPHY

1. AASHTO (2002). Standard specifications for highway bridges, 17th Ed., American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.
2. AASHTO (2012). LRFD bridge design specifications, 6th Ed., American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.
3. Akinci, N.O., Liu, J., and Bowman, M.D. (2008). "Parapet strength and contribution to live load response for super load passages." J. Bridge Eng., 13(1), 55-63.
4. Chung, W., Liu, J., and Sotelino, E.D. (2006). "Influence of secondary elements and deck cracking on the lateral load distribution of steel girder bridges." J. Bridge Eng., 11(2), 178-187.
5. Conner, S., and Huo, X.S. (2006). "Influence of parapets and aspect ratio on live-load distribution." J. Bridge Eng., 11(2), 188-196.
6. Davids, W., Poulin, T., and Goslin, K. (2013). "Finite-element analysis and load rating of flat slab concrete bridges." J. Bridge Eng., 18(10), 946-956.
7. Eamon, C., and Nowak, A. (2002). "Effects of edge-stiffening elements and diaphragms on bridge resistance and load distribution." J. Bridge Eng., 7(5), 258-266.
8. Fawaz, G., Waked, M., Mabsout, M., and Tarhini, K. (2016). "Influence of railings on load carrying capacity of concrete slab bridges." Journal of Bridge Structures: Assessment, Design, and Construction, IOS Press, accepted for publication.
9. Jensen, V. P. (1938). "A distribution procedure for the analysis of slabs continuous over flexible beams." Bulletin No. 304, Univ. of Illinois, Urbana, Illinois.
10. Jensen, V. P. (1939). "Moments in simple span bridge slabs with stiffened edges." Bulletin No. 315, Univ. of Illinois, Urbana, Illinois.
11. Mabsout, M., Tarhini, K., Frederick, G., and Kobrosly M. (1997). "Influence of sidewalks and railings on wheel load distribution in steel girder highway bridges." J. Bridge Eng., 2(3), 88-96.
12. Mabsout, M., Tarhini, K., Jabakhanji, R., and Awwad, E. (2004). "Wheel load distribution in simply supported concrete slab bridges." J. Bridge Eng., 9(2), 147-155.
13. Newmark, N. M. (1938). "A distribution procedure for the analysis of slabs continuous over flexible beams." Eng. Experiment Station Bull. No. 304, Univ. Illinois at Urbana Champaign, College of Engineering. Engineering Experiment Station.
14. SAP2000 (2012). Computers and Structures Inc., Berkeley, California
15. Westergaard, H. M. (1926). ''Stresses in concrete pavements computed by theoretical analysis.'' Public Roads, 7(2), pp. 25--35.
16. Westergaard, H. M. (1930). "Computation of stresses in slabs due to wheel loads." Public Roads, 11 (1), pp. 1--23.