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EFFECT OF DETERIORATION OF INTEGRAL RAILINGS ON ONE-SPAN MULTILANE CONCRETE SLAB BRIDGES

by
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ABSTRACT

Hazem Ali El Sankari for Master of Engineering
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Title: Effect of Deterioration of Integral Railings on One-Span Multilane 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 fail to take into consideration the effect of railings as being integral parts of the bridges. In addition, the effect of railing stiffness is overlooked in the design stage. Such integrally-built railings possess the effect of stiffening and attracting the load to the slab edge thus inducing an alteration of the wheel-load distribution on concrete slab highway bridges. Past research has presented and computed the increase in the load-carrying capacities of bridges with integrally-built railings, and this increase is significant and varies with the railing size and bridge geometry. Preliminary studies have shown that high stress or moment concentration in the slab edges may be induced due to accidental or long-term deterioration of the railings, and such concentrations may even, at times, exceed the moments in cases whereby no railings were present at all. This research will aim at studying and quantifying the effect of railing deterioration, taking into account various levels of full breakage at multiple locations along the bridge railing's extent. Typical one-span, simply-supported, multilane (three or four lanes) straight reinforced concrete slab bridges with standard railings on either or both slab edges are taken into account. The finite-element method is utilized to investigate the railing deterioration's effect occurring on one side of the slab edge. The railing's deterioration is investigated parametrically through the variation of the location and length, and the extent is modelled by assuming distinct full breakage or deterioration of the railings. Having selected bridges with no railings and bridges with full non-deteriorated railings as reference cases, the wheel-load distribution and bridge moments at the critical sections are evaluated. AASHTO design truck loads are inserted transversely and longitudinally to maximize the critical section moments in the slab. The wheel load distribution and slab moments, in addition to the deflections in the bridge slabs at critical sections are computed and compared with the reference bridge cases in addition to the AASHTO procedures. This study will aid structural engineers in comprehending the effect of railing deterioration and to better judge and design straight concrete slab bridges with integrated railings. Furthermore, recommendations will also be offered to prevent sudden bridge damage resulting from deterioration of railings.

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CONTENTS

ILLUSTRATIONS

TABLES

CHAPTER 1

INTRODUCTION

1.1 Background

The early 1900s and beyond witnessed an improvement in the design and construction practices of bridges. The conception of new bridge types gave forth new attempts by bridge engineers to conceptualize and manifest contemporary analysis, design, and construction methodologies. Multitudes of bridge types, ranging from short-span to suspension structures, are being employed today, with the common bridge component being the superstructure, or the bridge deck.

Reinforced concrete slab bridges provide alternatives for short-span bridges which are more economic. There are two major types of specifications to which concrete bridges, and all other ones too, must conform; namely, the American Association of State Highway and Transportation Officials (AASHTO) Specs of 2002 and the AASHTO Load Resistance Factor Design (LRFD) 2012 procedures.

1.2 Design Procedures

AASHTO design procedures originated in the 1940s under the research work of Westergaard along with Newmark and Jensen on moment and stress distribution in reinforced concrete slabs.

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1.3 Research Objectives

Bridge railings cause the alteration of the lateral wheel load distribution on concrete slab highway bridges through the attraction of the load to the slab edges. Long-term or accidental deteriorations within these railings result in loss of stiffness and the formations of high moment concentrations.

Using the finite-element method, this research will aim at investigating the effects of the full deterioration of integral railings in concrete slab bridges on the wheel-load distribution and the carrying capacity of multilane three-lane and four-lane bridges. The analysis includes the proper longitudinal and transverse placement of the AASHTO HS-20 design trucks to produce the maximum bending moments. The case of no railings as well as the case of having two non-deteriorated railings will serve as the reference cases with which the other results from FEA will be compared.

The maximum longitudinal moments, maximum edge moments, and maximum live load deflections are compared with the reference cases and with the AASHTO Standard Specs of 2002 and AASHTO LRFD 2012 procedures.

1.4 Scope and Methodology

This research will present the finite-element results of concrete slab highway bridges analyzed with no deterioration and full deterioration of the integral railings.

Typical one-span, simply-supported, three- and four-lane, straight reinforced concrete slab bridges with railings on either or both sides will be considered. Deterioration lengths are varied to evaluate behavior of the bridges prior to and beyond the deterioration.

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In the finite-element method, discretization of the bridge slab into a convenient number of elements is performed, assuming the interconnection of these elements at nodal points and with each element retaining the original structure's properties. In this research, the bridge's FEA model consists of square shell elements of 1ft x 1ft (0.3m x 0.3m) for the slab, rectangular shell elements of 1ft x 1.25ft (0.3m x 0.38m) for the railings, and hinged/roller supports for the piers. The commercial software package chosen for analysis is SAP2000 version 21.

Two span lengths are considered: 36ft (10.8m) and 54ft (16.2m). The slab lengths are assumed to be: 36ft (10.8m) for three lanes and 48ft (14.4m) for four lanes. One typical rectangular section presents the railing as an 8in x 30in (20cm x 76cm) cross-sectional area. There are five total deterioration lengths considered for the railings: 0, 1, 2, 4, and 8ft (0, 0.3, 0.6, 1.2, and 2.4m), with the 0-ft case corresponding to full railing without any deterioration. Full deterioration depth will be considered to evaluate the extreme case of fully damaged (and hence fully-lost stiffness) railing. The critical location of deterioration will be near the critical sections of the slab (near the centerline of the span).

HS-20 trucks are taken to be travelling in the same direction. The edge loading is considered as it is more critical than the centered loading. In this loading case, the trucks are placed side-by-side close to the edge of the slab, such that the leftmost truck is positioned at 1ft (0.3m) away from the left edge of the slab, and the trucks are placed at 4ft (1.2m) away from each other. Cases with no railings and cases with two railings without deterioration will serve as reference cases. The railing deteriorations produce new cases which are analyzed and compared with the reference cases and with the AASHTO procedures. The longitudinal and edge moments are evaluated at the critical section, and the maximum deflection for each case is obtained, and all of

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them are compared with the reference cases and with the AASHTO 2002 and AASHTO LRFD 2012 procedures.

1.5 Thesis Organization

There are five total chapters with the first one being this introduction chapter. Chapter 2 presents a description of the research work, as well as a presentation of AASHTO Standard Specifications and LRFD design procedures. Chapter 3 presents a description of the considered bridge cases and the FEA models utilized. Chapter 4 involves the effects of railing deterioration on the various bridge models along with the various resultant tables and graphs showing the FEA results and their comparison with AASHTO procedures and the reference cases. Chapter 5, finally, includes proposed recommendations based on the presented conclusions within.

CHAPTER 2

BACKGROUND AND AASHTO DESIGN PROCEDURES

2.1 Introduction

In this chapter, background information is presented in the form of a literature review of the topic at hand, followed by a concise summary of the AASHTO Standard Specifications (2002) and the AASHTO LRFD (2012) design procedures.

2.2 Literature Review

A concrete slab bridge should be designed according to the provisions pertaining to the main reinforcement which is parallel to traffic. AASHTO design procedures were conceived in the 1940s and are presented in the AASHTO Standard Specifications for Highway Bridges (2002) as per section 3.24: “Distribution of Loads and Design of Concrete Slabs”.

One study by Mabsout et al. (2004) evaluated one-span concrete bridges of up to 4 lanes with span lengths ranging from 7.2m to 16.2m (14ft to 54ft) but didn’t consider the effects railings had on the load-carrying capacity in the bridges. It was found that for more than one lane, AASHTO longitudinal moments agreed with FEA results for short spans (less than 35ft or 10.5m) but underestimated the moments for longer spans by 15% to 30%. The AASHTO LRFD procedures yielded moment values which exceeded the AASHTO 2002 Specs and converged more towards the FEA results. As for the edge beam moments, AASHTO overestimated the FEA results by 20% for short spans but agreed with FEA results for longer spans.

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Fawaz et al. (2017) found that moment value reductions, as compared to the presence of two railings, after the implementation of one railing, were insignificant. The implementation of two railings, however, yielded reduced moments and deflections by up to 50% in one- and two-lane bridges, and by up to 15% for three- and four-lane bridges with relatively long spans. AASHTO Standard Specs underestimated moments for bridges with one or no railings with long spans and more than two lanes by 25% mostly, while it overestimated FEA results by as much as 50% for one-lane bridges with short span lengths. Moreover, AASHTO LRFD agreed with FEA results even for long spans and three- and four-lane models. For short spans, the AASHTO LRFD overestimation was maximal, reaching 100% for one lane, and 40% for three and four lanes. For two railings, AASHTO LRFD exceeded FEA moments by 150% for one-lane cases and by 30% in three- and four-lane bridge cases.

Abou Nouh et al. (2017) and Fawaz et al. (2019) extended the previous study to consider railings with various stiffnesses, and found that the presence of two railings reduced the maximum slab moments, whereby this reduction decreased with an increase in the deck width, and increased with elevated railing stiffness. They concluded that through the modelling of five different railing cross-sections, whose moments of inertia reached four times the moment of inertia of the base case. Jaber et al. (2019) extended this work to two-equal-span continuous bridges.

Akinci et al. (2008) tested the presence of railings or “parapets”, as they referred to them, on super-load passages. They found that parapet presence reduced the Girder Distribution Factors (GDFs) by about 30%, thus allowing the passage an extra heavy-weight, or super-load, vehicle. They also evaluated breakages in the parapets and noticed that discontinuities within the parapets yielded elevated tensile deck stresses.

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Chung et al. (2006) investigated the presence of secondary elements such as diaphragms and parapets. They found that their presence yielded load distribution factors up to 40% lower than the AASHTO LRFD values. Instead of the elastic linear analysis adopted by AASHTO provisions to produce the codes, the researchers modelled their elements with non-linear finite element analyses. They also found that LRFD always overestimated moment values in the presence of secondary elements.

Darwich et al. (2019) evaluated the effect of railing deteriorations at critical locations on the maximum longitudinal slab moments, edge beam moments, and maximum live load deflections of simply supported, one-span, one- and two-lane reinforced concrete slab bridges. The parametric study was conducted with variable railings deterioration depth, length, and location along the concrete bridge. The overall conclusions are drawn based on the FEA results of this parametric investigation due to the presence of deteriorations in both one- and two-railing bridge cases.

The maximum FEA longitudinal bending moments in the concrete slabs were generally less affected with railing deterioration when compared with reference bridge cases with no railings deterioration, and consequently when compared with AASHTO Standard Specifications and LRFD moments. Furthermore, the FEA edge beam longitudinal bending moments in the concrete slabs were generally significantly increased with railing (half or full) deterioration when compared with reference bridge cases with no railings, or with bridge cases with no railing deteriorations. These FEA edge moments become critical when compared with AASHTO Standard Specifications and LRFD moments. Therefore, it is recommended to properly reinforce the edge beams at both sides of the slab deck, regardless of the presence of railings, and repair any deterioration in railings to minimize the effect of stress concentration on bending moments in the concrete slab bridges.

The study by Darwich et al. (2019) will form the basis for this research, which will be extended to three and four lanes, so that the whole "matrix" of bridge models has been duly considered and analyzed.

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 aims at reducing the two-way bending problem into a beam (one-way) bending problem. So, reinforced concrete slab bridges are designed as a series of beam strips.

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.

$$M_{AASHTO}\ (Kip - ft/ft) = 0.9S \qquad for\ S < 50ft$$

$$M_{AASHTO}\ (Kip - ft/ft) = 1.30S - 20 \qquad for\ 50ft < S < 100ft$$

where S is the span length in feet.

For short-span structures, the truck loading governs the design process, so lane loading is neglected here.

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

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2.3.2 Edge Beam

Assuming an edge beam width of 1.5ft, which is used by some transportation departments, the edge beam moment allowable by the AASHTO 2002 Specs is:

$$M_{Edge\_AASHTO}\ (Kip - ft/ft) = \frac{0.1PS}{1.5}$$

where P = 16kips for AASHTO HS-20 Design Truck

and S is the span length in units of feet

2.3.3 Live Load Deflection

AASHTO offers a maximum live load deflection D which shall not be exceeded as:

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

2.4 AASHTO Load Resistance Factor Design (LRFD)

The following section presents the AASHTO LRFD procedures and formulas.

2.4.1 Slab Design

AASHTO LRFD section 4.6.2.3 provides an equivalent strip length to design bridges. It consists of dividing the total statical moment M_0 by the bridge equivalent E to achieve a moment per unit width for design. The equivalent length E is determined using:

$$E = 10 + 5\sqrt{L_1 W_1}\ in\ inches \qquad (LRFD\ Equation\ 4.6.2.3 - 1)$$

$$E = 84 + 1.44\sqrt{L_1 W_1} \le \frac{W}{N_L}\ in\ inches \qquad (LRFD\ Equation\ 4.6.2.3 - 2)$$

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where:

E is the equivalent length in inches

L_1 is the span length in feet taken equal to the lesser of the actual span or 60ft

W_1 is the modified edge-to-edge length of the bridge taken to be the lesser of the actual length or 60ft for multi-lane loading or 30ft for single-lane loading

W is the physical edge-to-edge length of the bridge

N_L is the number of design lanes

Finally, the live load longitudinal moment M can be obtained using:

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

2.4.2 Edge Beam

AASHTO LRFD edge beam moment (article 4.6.2.1.4b) is assumed to support one line of wheel load and a tributary portion of the design lane load, with the effective length being the sum of distance between the edge of the deck and the inside face of the barrier (taken to be 1ft), plus 1ft, plus one-quarter of the strip length E which was computed earlier on, with the constraint of not exceeding either one-half the full strip length or 6ft.

2.4.3 Live Load Deflection

Similar to AASHTO Standard Specs (2002), the AASHTO LRFD procedures specify a maximum deflection D for simple or continuous spans, which shall not be exceeded, as:

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

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CHAPTER 3

BRIDGE CASE DESCRIPTION, MODELLING AND ANALYSIS

3.1 Introduction

This chapter presents the study carried out in the form of FEA of concrete slab bridges. The different geometric and physical variations of the bridge models are presented and discussed. It also involves an explanation of the 3D FEA modelling technique followed and summarizes all the bridge cases considered.

3.2 Bridge Cases Description

3.2.1 Geometry and Dimensions

A total of ninety-six bridge models were analyzed, all of which were simply-supported one-span reinforced concrete slab bridges with the following geometrical properties varied:

- Span Length (36ft or 54ft)
- Number of Lanes (3 lanes or 4 lanes)
- Presence of Railings
- Transverse Loading Position
- Deterioration of Railings (0, 1, 2, 4, and 8ft)

The corresponding slab thicknesses chosen are as follows:

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

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

A typical lane has a fixed length of 12ft. Based on which, for the number of lanes considered, that is, three and four, the equivalent slab lengths are as follows:

- 36ft for three lanes
- 48ft for four lanes

Other parameters include the depth and length of the railing deterioration. The lengths varied between 0, 1, 2, 4, and 8ft, with the 0-foot case being the case in which no deterioration occurred. The depth of the railing was either 0 (undamaged railing) or 30in (fully damaged railing). The breakages were located near the span centerline.

As presented before, the railing cross-sectional area is 8in in length by 30in in depth above the slab. They can be on neither, either, or both sides of the slab as shown in figure 3.1 below. The deteriorated railings are schematized in figure 3.2 below too.

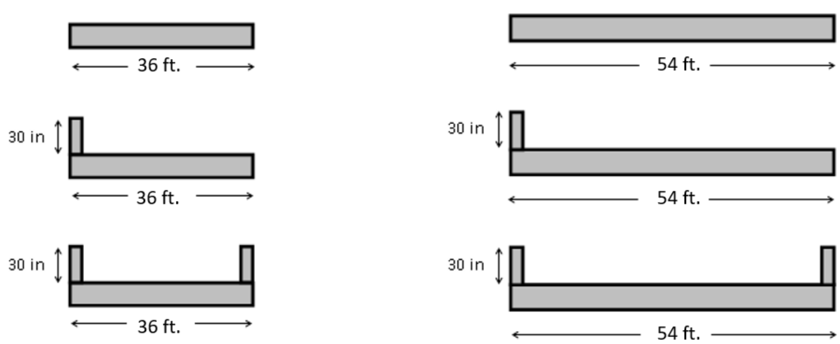

Figure 3.1: Typical Cross Sections for Three- and Four-Lane Bridge Cases with/without Railings

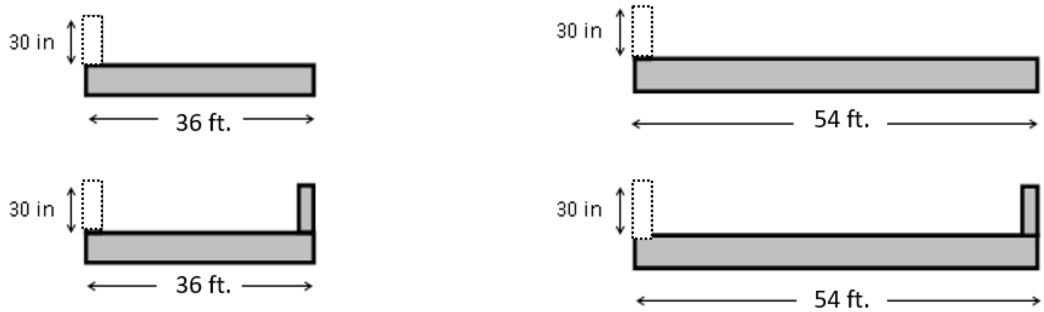

Figure 3.2: Typical Cross Sections for Three- and Four-Lane Bridge Cases with Deteriorations

Table 3.1: Summary of the Geometric Properties of the Modelled Bridges

Number of Lanes	Span Length	Slab Thickness	Slab Width	Depth of Deterioration	Length of Deterioration
	(ft)	(in)	(ft)	(in)	(ft)
3	36	21	36	30	0, 1, 2, 4 and 8
	54	27			
4	36	21	48	30	0, 1, 2, 4 and 8
	54	27			

3.2.2 Physical Properties

Normal-strength concrete was adopted in this research having the following material properties:

- Compressive Strength: f'_c (28days) = 4,000psi = 4ksi = 28MPa
- Modulus of Elasticity: E = 3.60 x 10^6psi
- Poisson's Ratio: ν = 0.2

3.2.3 AASHTO Design Truck

As mentioned before, truck loading governs the design of short-span structures based on AASHTO Standard Specifications (2002). Therefore, the bridges are analyzed based on the HS20-44 truck shown in figure 3.3 below as given by AASHTO. The weight of this truck is 72kips distributed as follows over two rear axles and one front axle:

- 32kips per rear axle or 16kips per each rear wheel
- 8kips for the front axle or 4kips per front wheel

The three axles are spaced 14ft apart to maximize the moment in the slab.

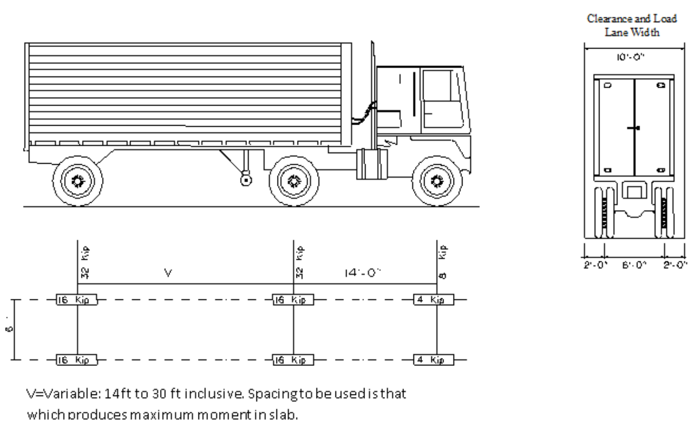

Figure 3.3: AASHTO HS-20 Design Truck (AASHTO Standard Specifications for Highway Bridges, 2002

3.2.4 Longitudinal Loading Position of Design Trucks

Adopting the results of Mabsout et al. (2004), the maximum positive moment is shown to occur if the truck is positioned with its center load coinciding with the mid-span of the bridge. This is reflected in both figures 3.4 and 3.5 for the cases of 36-ft and 54-ft spans, respectively.

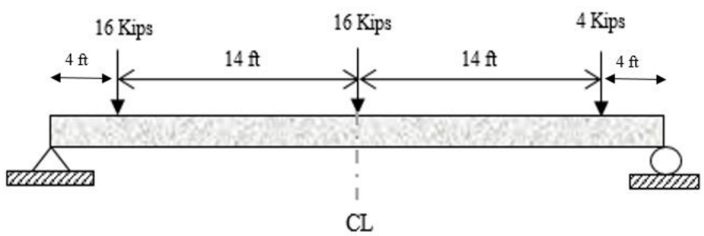

Figure 3.4: Truck Positioning for One Span of 36-ft Length

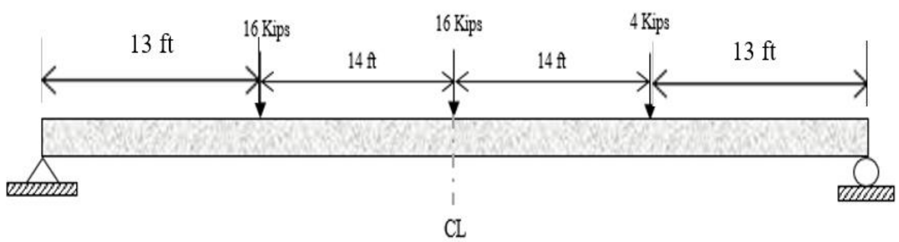

Figure 3.5: Truck Positioning for One Span of 54-ft Length

3.2.5 Transverse Loading Position of Design Trucks

The trucks are assumed to be travelling in the same direction on the bridge. The edge loading condition is adopted as the critical case after previous research including that by Mabsout

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et al. (2004) and Fawaz et al. (2016) whereby the leftmost truck's left wheel is positioned at 1ft away from the edge of the slab, and the truck-to-truck distance is 4ft. This produces the worst loading condition on the bridge as shown in figure 3.6 below.

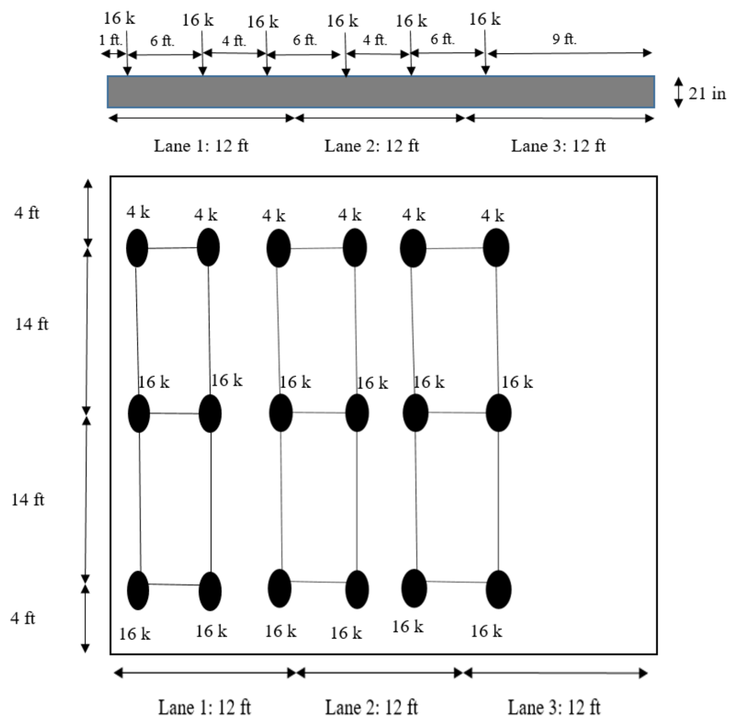

Figure 3.6: Transverse Truck Loading for 36-ft Span

There are two edge loading conditions when one railing is present, E1 and E2. E1 is where the truck is positioned next to this one railing in order to maximize the railing moment, and E2 is where the truck is positioned close to the opposite side which has no railing in order to maximize the slab moment.

3.2.6 *Modelling the Deteriorated Railings*

The railings are built integrally with the slab to provide additional stiffness. However, sudden damage or deterioration to the railings will cause the alteration of the wheel load distribution and the loss of stiffness at the deterioration location. Bridges with no railings, as well as bridges with two non-deteriorated railings, are taken as the reference cases. Railing deteriorations are then applied near the span centerline with varying lengths from 0 to 8ft as discussed before. Figures 3.7 to 3.12 show typical cross-sections and plans of various bridges with and without railing deterioration.

The critical moments at the slab centerline are extracted from SAP2000 through FEA, and are then compared with AASHTO Standard Specifications (2002) and AASHTO LRFD procedures (2012).

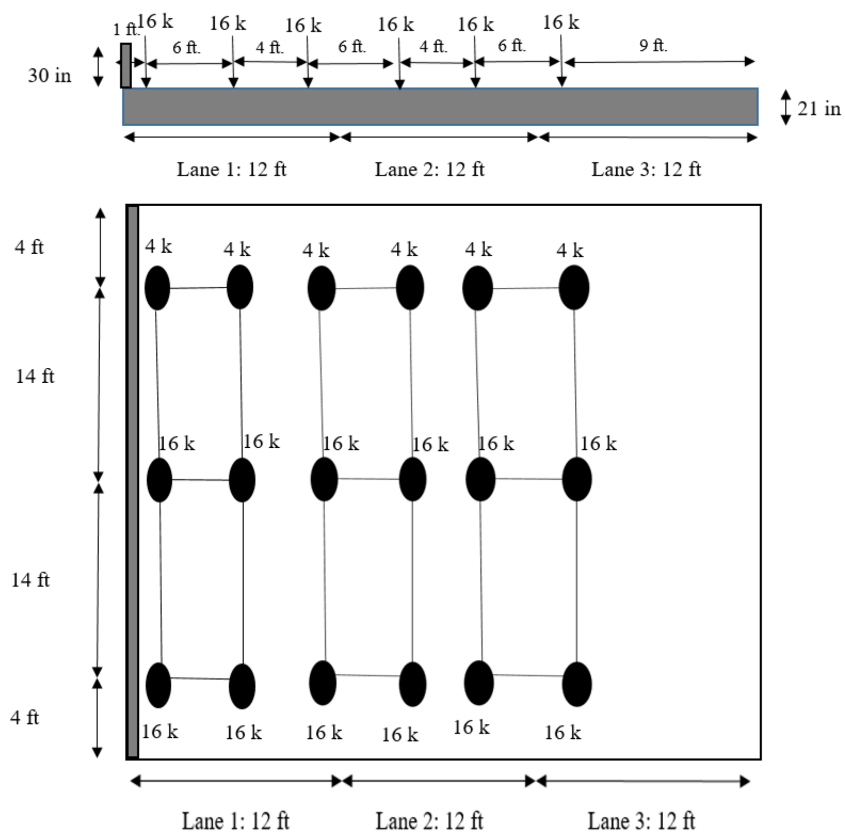

Figure 3.7: Centerline Cross-Section and Plan for 3-Lane 36-ft Span with One Railing Under E1 Loading without Deterioration

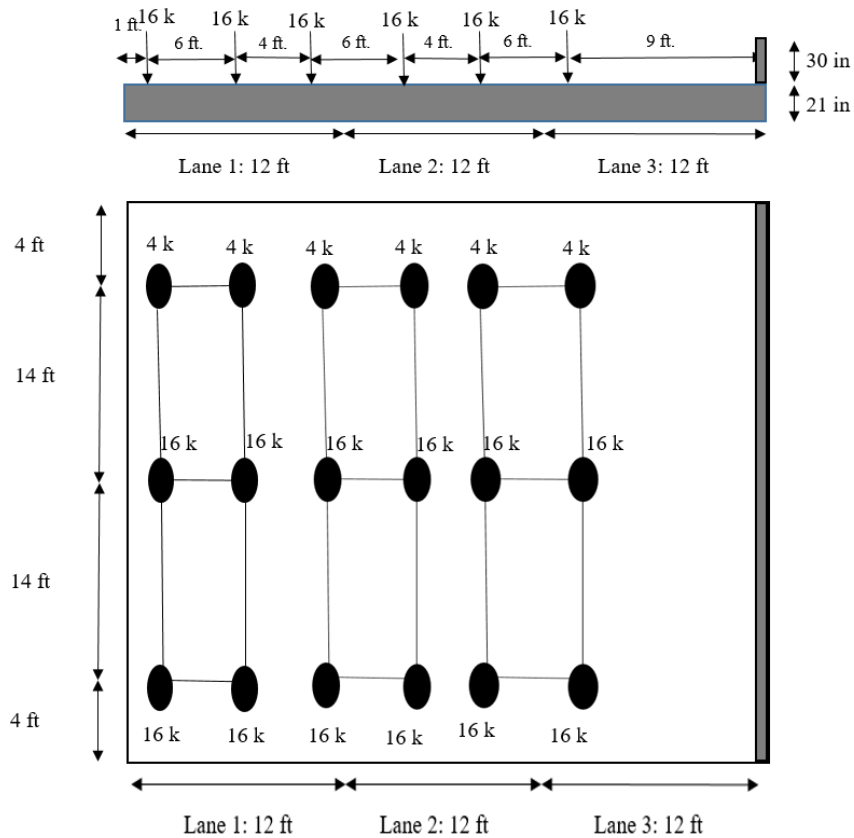

Figure 3.8: Centerline Cross-Section and Plan for 3-Lane 36-ft Span with One Railing Under E2 Loading without Deterioration

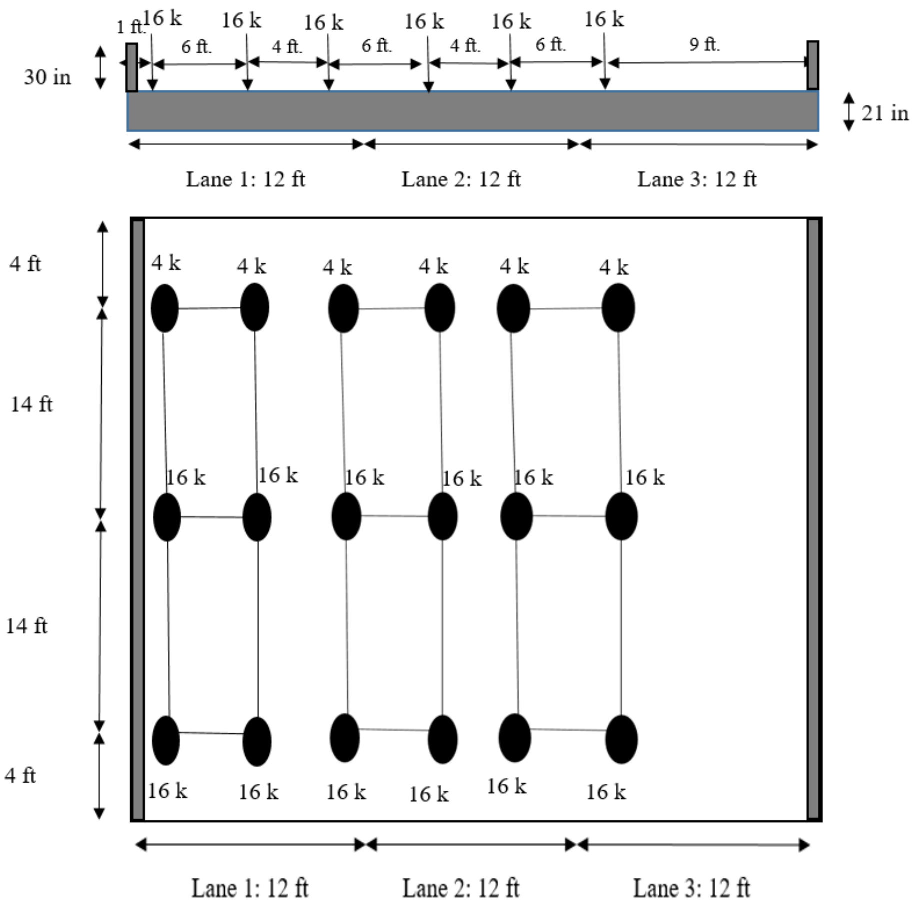

Figure 3.9: Centerline Cross-Section and Plan for 3-Lane 36-ft Span with Two Railings under E1 Loading without Deterioration

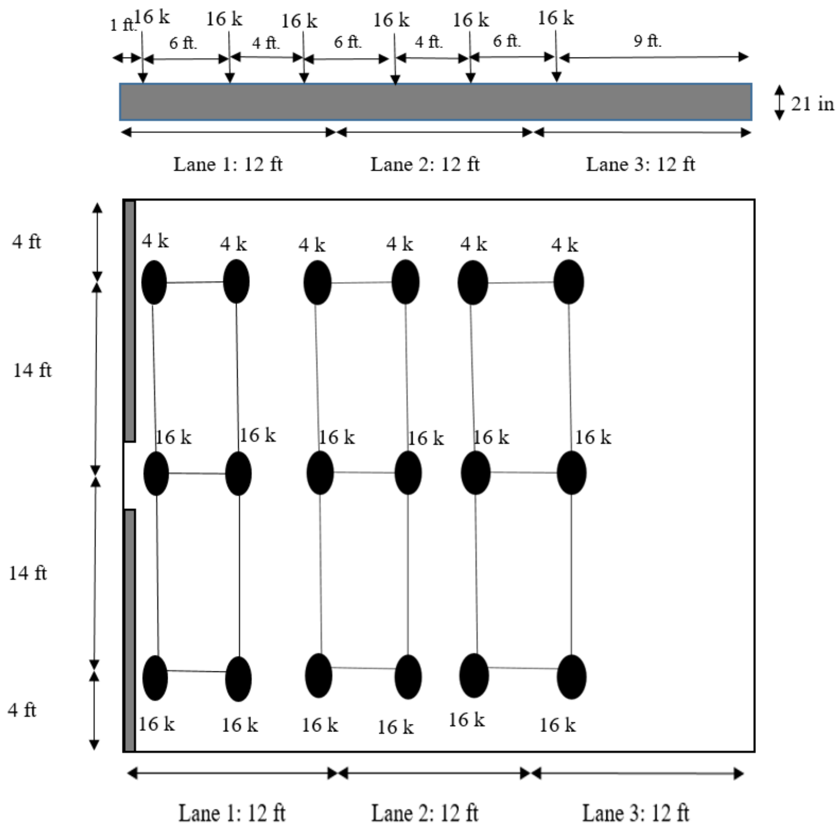

Figure 3.10: Centerline Cross-Section and Plan for 3-Lane 36-ft Span with One Railing under E1 Loading with 2-ft Deterioration

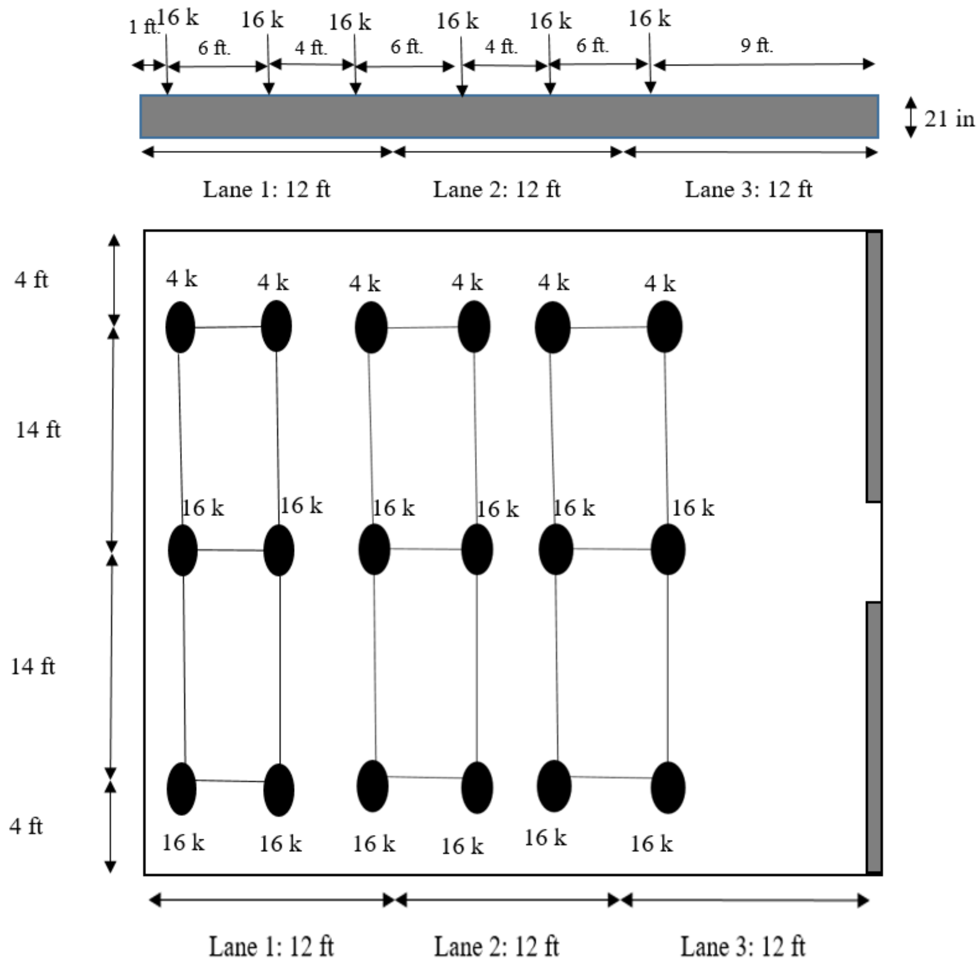

Figure 3.11: Centerline Cross-Section and Plan for 3-Lane 36-ft Span with One Railing under E2 Loading with 2-ft Deterioration

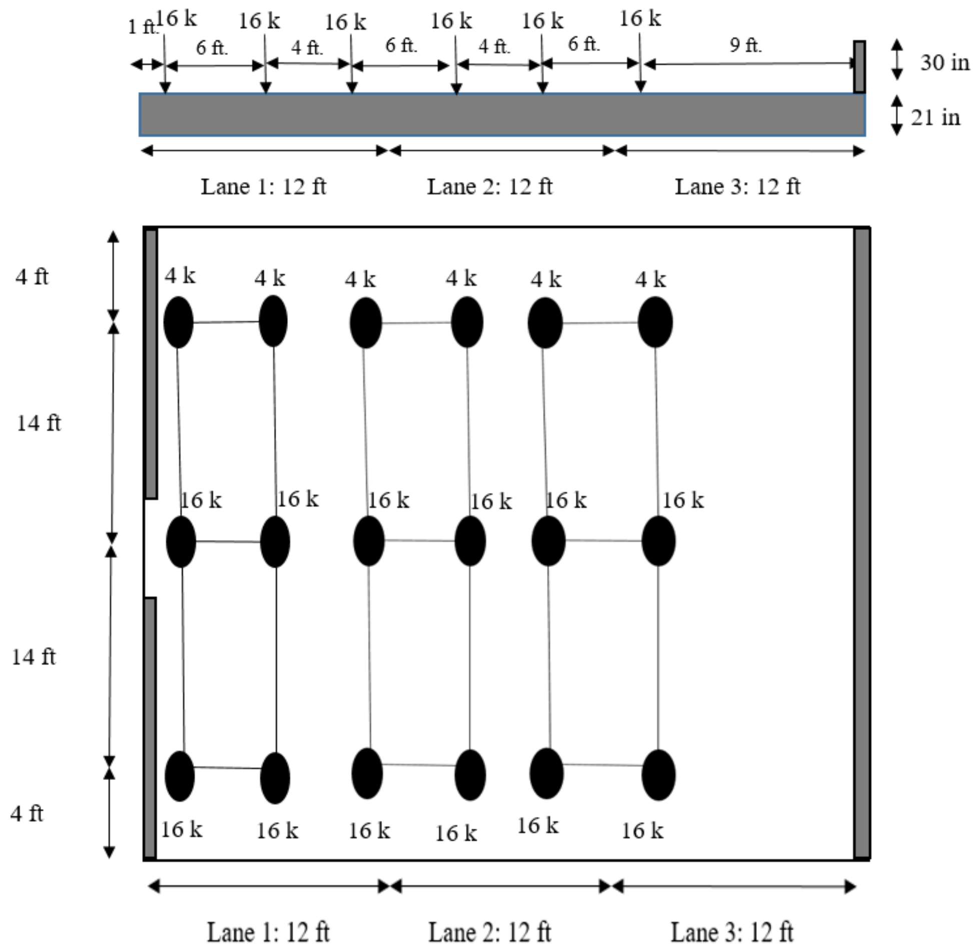

Figure 3.12: Centerline Cross-Section and Plan for 3-Lane 36-ft Span with Two Railings under E1 Loading with 2-ft Deterioration

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3.3 Finite Element Modelling and Analysis

The finite element method was utilized in the analysis of one-span three- and four-lane simply-supported reinforced concrete slab bridges. Using SAP2000 (2019), the bridge slab was discretized into a convenient number of shell elements with six degrees of freedom per node, and this shell element is capable of modelling plate behavior and membrane behavior. Linear elastic analysis was assumed, with small deformations and deflections, while having shear deformations completely neglected. Previous studies compared the use of different sizes of shell elements of 0.5ft x 0.5ft, 1ft x 1ft, and 2ft x 2ft. The results were nearly identical for all three shell element sizes. Therefore, the 1ft x 1ft shell element sizing was adopted as it was considered sufficient enough for the analysis. It was also chosen based on the convenience it offered when it came to placing truck loads at 1ft away from the slab edge to model the E1 and E2 loading cases.

Modelling deteriorated railings is not possible using frame elements which were used lastly by Fawaz et al. (2016), so shell elements were also utilized for railing modelling.

A broken railing is modelled by assigning a zero-stiffness value to the deteriorated portion of the railing, or by removing the unwanted portion of the railing after discretizing.

Piers were modelled as hinges and roller supports, with a series of hinges at one end of the span, and another series of rollers at the opposite end.

The longitudinal and edge bending moments and deflections are extracted from SAP2000 in this study, and are then investigated and reported.

Following are three figures 3.13, 3.14, and 3.15 which show geometry and loading conditions, deflection, and moment contour plot, respectively, of a 36-ft one-span three-lane simply supported reinforced concrete slab bridge subjected to E1 loading condition with two railings and one of them being deteriorated by 2ft.

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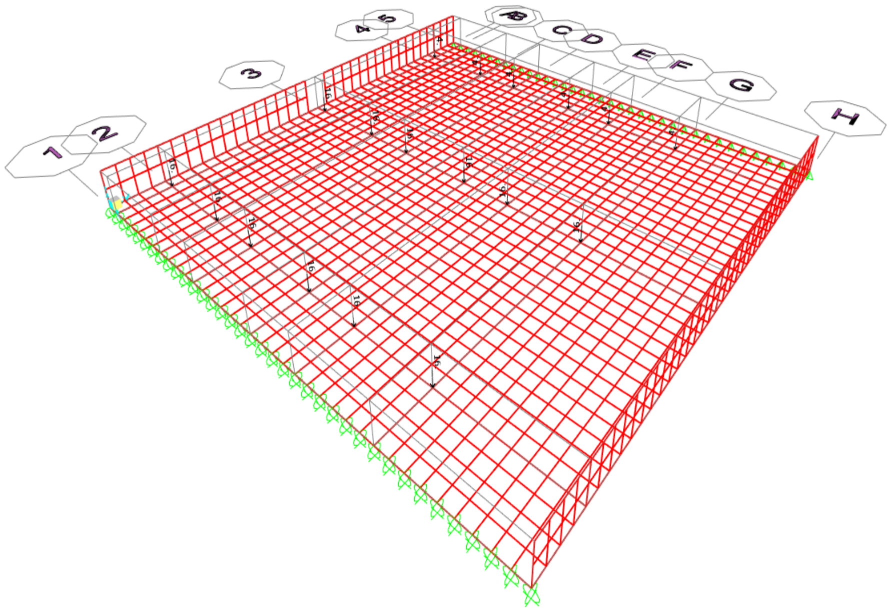

Figure 3.13: Geometry and Loading Condition of a 36-ft One-Span Two-Railing Bridge with E1 Loading and 2-ft Railing Deterioration

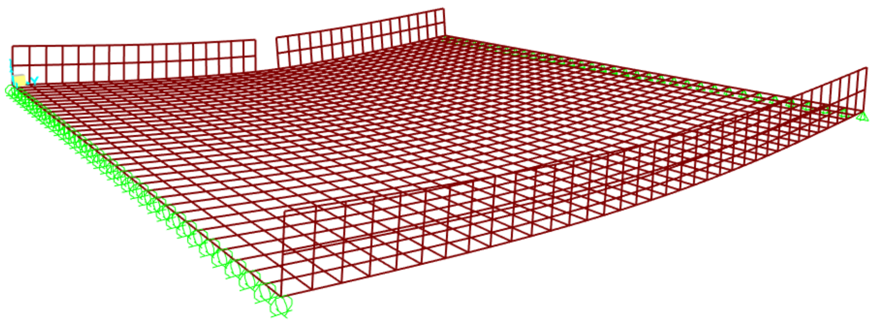

Figure 3.14: Deflected Shape of a 36-ft One-Span Two-Railing Bridge with E1 Loading and 2-ft Railing Deterioration

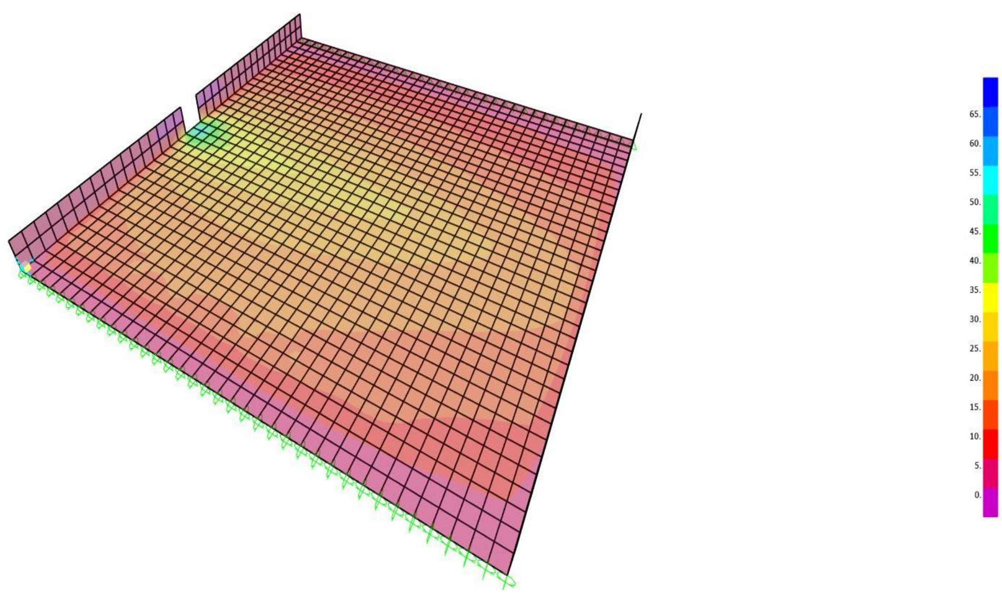

Figure 3.15: Moment Contour Plot of a 36-ft One-Span Two-Railing Bridge with E1 Loading and 2-ft Railing Deterioration

3.4 Summary

A total of ninety-six different bridge cases were analyzed with variations in geometric parameters, loading distribution, and absence/presence of railings with/without deterioration.

Two different span lengths (36ft and 54ft) were chosen with a total number of two span widths (36ft for three lanes and 48ft for four lanes). Five lengths of railing deteriorations are considered.

Cases with no railings and cases with two fully non-deteriorated railings will serve as reference cases for comparison with other analyzed cases.

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CHAPTER 4

ANALYSIS RESULTS AND DISCUSSION

4.1 Introduction

Using SAP2000, this chapter aims at presenting the results from FEA of bridge cases discussed earlier on in chapter 3.

The division of this chapter will fall under four major categories as follows:

- Comparison of FEA Results with AASHTO and LRFD (FEA Tabulated Results and Graphs)
- FEA-to-AASHTO Percentages (Tabulated and Summarized Results)
- Comparison of FEA Results with No-Railing (NR) Case (Tabulated Ratios)
- Comparison of FEA Results with Non-Deteriorated Two-Railing Case (R2-0) (Tabulated Ratios)

Note the abbreviations below as follows:

- 36 or 54 : Span Length
- 3L or 4L : Number of Lanes
- R1 or R2 : One or Two Railings
- E1 or E2: Edge Loading 1 or Edge Loading 2 Condition

For example, a (36-3L-R1-E1) case means a bridge with 36-ft span length, 3 lanes, one railing, and E1 loading condition.

Moreover, each major category above will be subdivided into subcategories as follows below:

- Comparison of FEA with AASHTO and LRFD
  - 3 Lanes; One Railing
    - 36-3L-R1-E1
    - 36-3L-R1-E2
    - 54-3L-R1-E1
    - 54-3L-R1-E2
  - 4 Lanes; One Railing
    - 36-4L-R1-E1
    - 36-4L-R1-E2
    - 54-4L-R1-E1
    - 54-4L-R1-E2
  - 3 Lanes; Two Railings
    - 36-3L-R2-E1
    - 54-3L-R2-E1
  - 4 Lanes; Two Railings
    - 36-4L-R2-E1
    - 54-4L-R2-E1
- FEA-to-AASHTO Percentages
  - R1-E1
  - R1-E2
  - R2-E1
- Comparison of FEA with No-Railing (NR) case
  - R1-E1
  - R1-E2
  - R2-E1
- Comparison of FEA with Non-Deteriorated Two-Railing Case (R2-0)
  - R1-E1
  - R1-E2
  - R2-E1

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4.2 Presentation of Results

The FEA results obtained contain longitudinal bending moments and edge beam moments from the critical locations of the bridge slabs, as well as the maximum live load deflections.

The maximum longitudinal bending moment is the first peak after the left edge peak moment, with the maximum edge peak moment being resisted by an edge beam.

The edge beam moment is the maximum moment at the slab edge along the critical cross-section of the slab. In the absence of railings, it is taken as the maximum of the two edge beam moments.

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

The following sections and subsections present the FEA longitudinal bending moments per unit foot along the critical cross-sections for all bridges and reference bridges, and are tabulated and plotted with the corresponding AASHTO Standard Specs and LRFD moments in Tables 4.1a to 4.12a and Figures 4.1 to 4.12, respectively. A summary of the FEA results, maximum longitudinal bending moments, edge beam moments, and maximum live load deflections is presented in Tables 4.1b to 4.12b.

4.2.1 FEA Longitudinal Moments and Deflections for 3 Lanes with One Railing

- 36-3L-R1-E1
- 36-3L-R1-E2
- 54-3L-R1-E1

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- 54-3L-R1-E2

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➢ Span =36ft - Three Lanes - One Railing - E1

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

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	39.3	26.9	69.5	52.6	47.5	42.5	32.4	32.6
1	40.6	29.4	52.0	55.1	48.2	43.7	32.4	32.6
2	37.0	26.8	44.5	45.8	43.4	39.8	32.4	32.6
3	35.4	26.0	39.3	40.5	40.0	37.9	32.4	32.6
4	34.7	26.0	36.2	37.3	37.6	36.7	32.4	32.6
5	34.5	26.4	34.4	35.5	36.1	35.9	32.4	32.6
6	35.0	27.5	33.4	35.0	35.8	36.0	32.4	32.6
7	37.5	30.5	31.9	36.8	37.5	38.1	32.4	32.6
8	34.7	28.1	32.0	33.5	34.2	34.9	32.4	32.6
9	33.8	27.6	31.4	32.3	33.0	33.7	32.4	32.6
10	34.0	28.2	31.0	32.4	33.0	33.8	32.4	32.6
11	36.2	30.8	29.8	34.4	35.0	35.8	32.4	32.6
12	33.1	28.0	30.0	31.3	31.8	32.5	32.4	32.6
13	31.8	27.0	29.4	30.0	30.5	31.2	32.4	32.6
14	31.3	26.8	28.9	29.5	29.9	30.6	32.4	32.6
15	31.2	26.9	28.8	29.4	29.9	30.5	32.4	32.6
16	31.8	27.8	28.9	30.1	30.5	31.1	32.4	32.6
17	34.3	30.5	28.1	32.6	33.0	33.6	32.4	32.6
18	31.5	27.9	28.7	29.9	30.2	30.7	32.4	32.6
19	30.6	27.2	28.3	29.0	29.3	29.8	32.4	32.6
20	30.7	27.6	28.1	29.3	29.6	30.0	32.4	32.6
21	32.9	29.9	27.0	31.4	31.7	32.2	32.4	32.6
22	29.6	26.8	27.2	28.3	28.6	29.0	32.4	32.6
23	28.3	25.6	26.5	27.0	27.2	27.6	32.4	32.6
24	27.6	25.0	26.0	26.3	26.6	27.0	32.4	32.6
25	27.3	24.9	25.7	26.1	26.4	26.7	32.4	32.6
26	27.7	25.4	25.6	26.6	26.8	27.1	32.4	32.6
27	29.9	27.7	24.5	28.8	29.0	29.3	32.4	32.6
28	26.7	24.6	24.7	25.6	25.8	26.1	32.4	32.6
29	25.2	23.2	23.9	24.2	24.4	24.7	32.4	32.6
30	24.3	22.4	23.1	23.3	23.5	23.8	32.4	32.6
31	23.6	21.8	22.6	22.7	22.9	23.1	32.4	32.6
32	23.1	21.3	22.1	22.2	22.4	22.7	32.4	32.6
33	22.8	21.0	21.8	21.9	22.0	22.3	32.4	32.6
34	22.5	20.8	21.6	21.6	21.8	22.1	32.4	32.6
35	22.3	20.7	21.4	21.5	21.6	21.9	32.4	32.6
36	22.2	20.6	21.4	21.4	21.6	21.8	32.4	32.6

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

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	40.6	29.4	69.5	55.1	48.2	43.7
	At Deterioration	40.6	29.4	68.1	63.7	56.5	50.2
Maximum Longitudinal Moment (k-ft/ft)		37.5	30.8	39.3	40.5	40.0	38.1
Maximum Live Load Deflection (in)		0.233	0.162	0.204	0.208	0.212	0.221

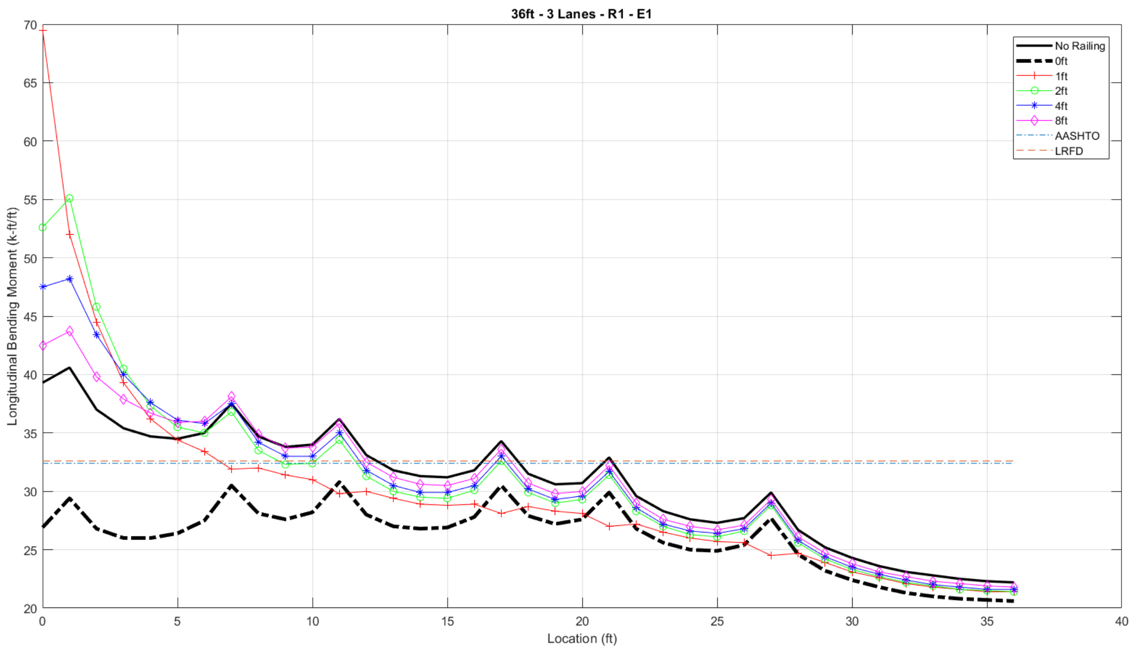

Figure 4.1: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 36ft, One Railing with Edge Loading E1

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➢ Span =36ft - Three Lanes - One Railing - E2

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

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	39.3	38.3	38.7	38.8	38.9	39.1	32.4	32.6
1	40.6	39.6	40.0	40.1	40.2	40.4	32.4	32.6
2	37.0	35.9	36.3	36.4	36.5	36.7	32.4	32.6
3	35.4	34.3	34.7	34.8	34.9	35.1	32.4	32.6
4	34.7	33.5	34.0	34.1	34.2	34.4	32.4	32.6
5	34.5	33.3	33.8	33.8	34.0	34.2	32.4	32.6
6	35.0	33.8	34.3	34.4	34.5	34.7	32.4	32.6
7	37.5	36.2	36.7	36.8	36.9	37.1	32.4	32.6
8	34.7	33.3	33.9	34.0	34.1	34.3	32.4	32.6
9	33.8	32.4	33.0	33.0	33.2	33.4	32.4	32.6
10	34.0	32.5	33.2	33.3	33.4	33.6	32.4	32.6
11	36.2	34.6	35.3	35.4	35.6	35.8	32.4	32.6
12	33.1	31.4	32.2	32.2	32.4	32.7	32.4	32.6
13	31.8	30.1	30.8	30.9	31.1	31.4	32.4	32.6
14	31.3	29.4	30.3	30.4	30.6	30.8	32.4	32.6
15	31.2	29.3	30.1	30.2	30.4	30.7	32.4	32.6
16	31.8	29.8	30.7	30.8	31.0	31.4	32.4	32.6
17	34.3	32.1	33.1	33.3	33.5	33.8	32.4	32.6
18	31.5	29.2	30.2	30.4	30.6	31.0	32.4	32.6
19	30.6	28.1	29.3	29.4	29.7	30.1	32.4	32.6
20	30.7	28.2	29.4	29.6	29.9	30.3	32.4	32.6
21	32.9	30.1	31.5	31.7	32.0	32.4	32.4	32.6
22	29.6	26.7	28.2	28.4	28.7	29.2	32.4	32.6
23	28.3	25.2	26.8	27.0	27.4	27.8	32.4	32.6
24	27.6	24.3	26.1	26.4	26.7	27.2	32.4	32.6
25	27.3	23.8	25.9	26.1	26.5	27.0	32.4	32.6
26	27.7	24.0	26.3	26.6	27.0	27.5	32.4	32.6
27	29.9	26.0	28.6	28.9	29.4	29.9	32.4	32.6
28	26.7	22.5	25.5	25.9	26.4	26.8	32.4	32.6
29	25.2	20.7	24.3	24.7	25.2	25.6	32.4	32.6
30	24.3	19.5	23.8	24.3	24.8	24.9	32.4	32.6
31	23.6	18.6	23.8	24.3	24.7	24.6	32.4	32.6
32	23.1	17.8	24.3	24.9	25.1	24.5	32.4	32.6
33	22.8	17.1	25.6	26.2	25.9	24.4	32.4	32.6
34	22.5	16.5	28.2	28.4	26.8	24.4	32.4	32.6
35	22.3	15.9	32.9	32.0	27.4	24.4	32.4	32.6
36	22.2	15.5	43.3	31.0	27.7	24.3	32.4	32.6

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

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	40.6	39.6	43.3	40.1	40.2	40.4
	At Deterioration	40.6	39.6	41.8	40.8	38.0	32.6
Maximum Longitudinal Moment (k-ft/ft)		37.5	36.2	36.7	36.8	36.9	37.1
Maximum Live Load Deflection (in)		0.233	0.226	0.229	0.300	0.230	0.232

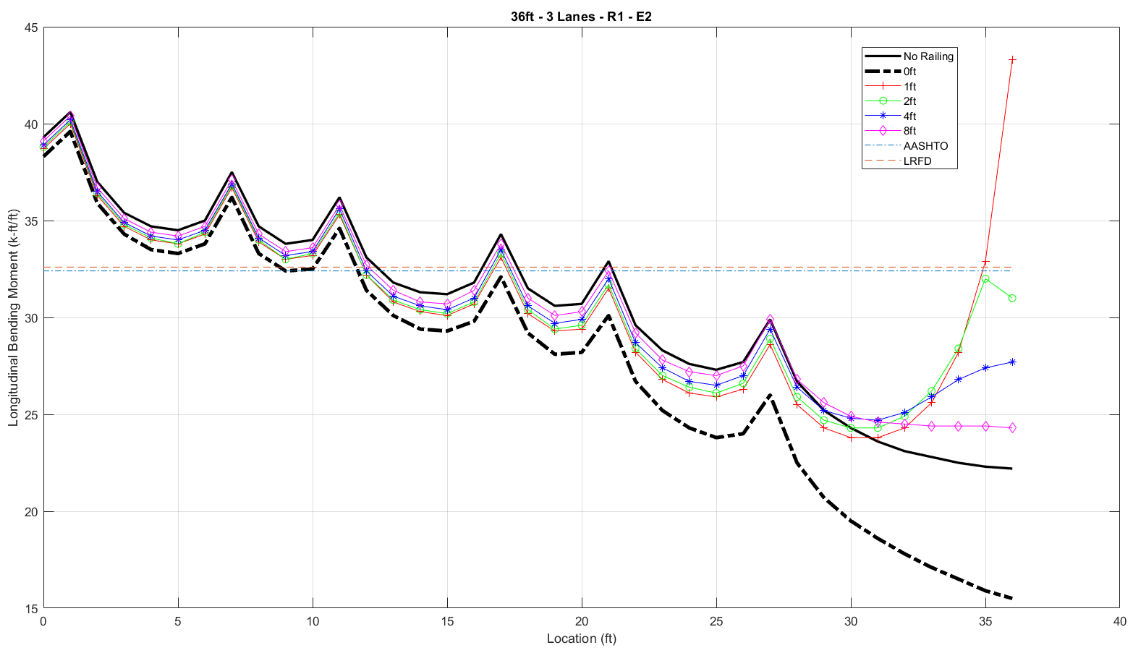

Figure 4.2: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 36ft, One Railing with Edge Loading E2

➢ Span =54ft - Three Lanes - One Railing - E1

Table 4.3a: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, One Railing with Edge Loading E1

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	70.6	46.6	162.4	119.1	100.7	84.6	50.2	57.7
1	71.8	48.9	125.8	116.1	98.1	84.8	50.2	57.7
2	68.0	46.7	98.3	99.3	90.6	79.9	50.2	57.7
3	66.4	46.1	84.4	86.3	84.0	76.8	50.2	57.7
4	65.5	46.2	75.3	77.8	78.2	74.4	50.2	57.7
5	65.2	46.7	69.9	72.3	73.8	72.4	50.2	57.7
6	65.7	48.0	67.0	69.1	71.1	71.2	50.2	57.7
7	68.0	51.0	67.0	68.9	71.0	72.0	50.2	57.7
8	65.1	48.7	62.4	64.1	66.2	67.7	50.2	57.7
9	64.1	48.3	60.2	61.7	63.7	65.6	50.2	57.7
10	64.3	49.0	59.5	60.9	62.8	64.7	50.2	57.7
11	66.4	51.6	60.9	62.2	64.0	66.0	50.2	57.7
12	63.1	48.9	57.3	58.5	60.1	62.2	50.2	57.7
13	61.8	48.0	55.6	56.7	58.3	60.3	50.2	57.7
14	61.2	47.8	54.8	55.8	57.3	59.2	50.2	57.7
15	61.0	48.1	54.5	55.4	56.8	58.7	50.2	57.7
16	61.6	49.1	55.0	55.9	57.2	59.0	50.2	57.7
17	64.0	51.9	57.4	58.2	59.4	61.2	50.2	57.7
18	61.1	49.3	54.5	55.3	56.5	58.2	50.2	57.7
19	60.2	48.7	53.6	54.3	55.4	57.1	50.2	57.7
20	60.3	49.2	53.8	54.5	55.5	57.1	50.2	57.7
21	62.4	51.6	56.0	56.6	57.6	59.1	50.2	57.7
22	59.2	48.6	52.8	53.4	54.4	55.9	50.2	57.7
23	57.8	47.5	51.5	52.1	53.0	54.4	50.2	57.7
24	57.1	47.1	50.9	51.4	52.3	53.7	50.2	57.7
25	56.8	47.0	50.7	51.3	52.1	53.5	50.2	57.7
26	57.2	47.7	51.2	51.7	52.6	53.9	50.2	57.7
27	59.4	50.1	53.5	54.0	54.8	56.1	50.2	57.7
28	56.2	47.1	50.4	50.9	51.7	52.9	50.2	57.7
29	54.8	45.8	49.0	49.5	50.3	51.5	50.2	57.7
30	53.9	45.1	48.2	48.7	49.4	50.6	50.2	57.7
31	53.3	44.6	47.7	48.1	48.9	50.0	50.2	57.7
32	52.8	44.3	47.3	47.7	48.5	49.6	50.2	57.7
33	52.5	44.1	47.0	47.5	48.2	49.3	50.2	57.7
34	52.2	44.0	46.9	47.3	48.0	49.1	50.2	57.7
35	52.1	44.0	46.8	47.2	47.9	49.0	50.2	57.7
36	52.0	44.0	46.8	47.3	47.9	49.0	50.2	57.7

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Table 4.3b: FEA Summary Results for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, One Railing with Edge Loading E1

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	71.8	48.9	162.4	119.1	100.7	84.8
	At Deterioration	71.8	48.9	162.3	147.3	132.5	119.1
Maximum Longitudinal Moment (k-ft/ft)		68.0	51.9	84.0	86.3	84.0	76.8
Maximum Live Load Deflection (in)		0.500	0.338	0.401	0.410	0.422	0.442

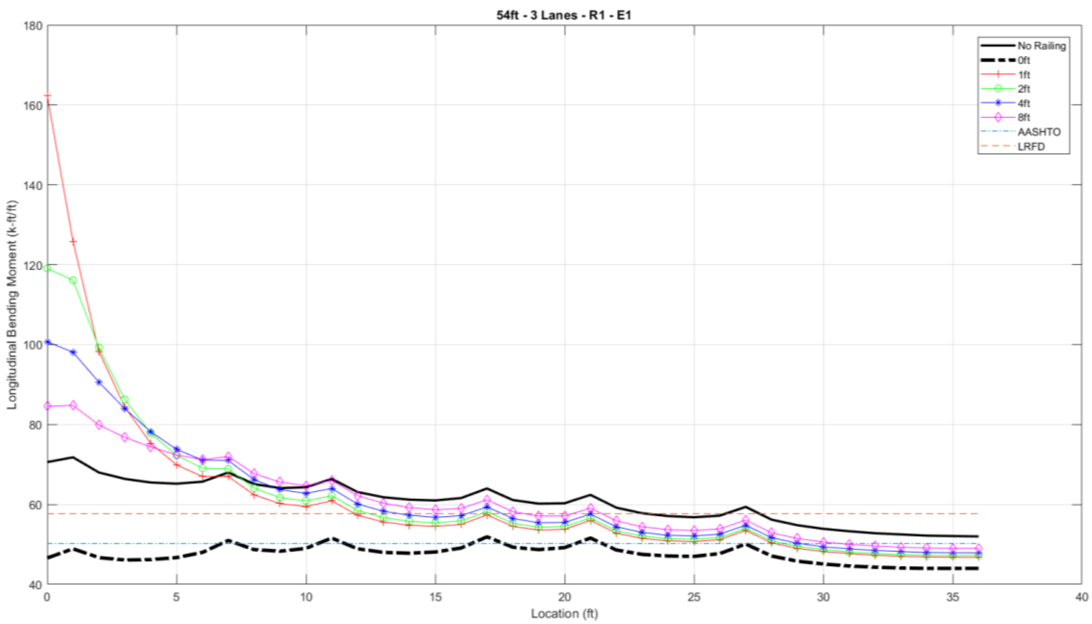

Figure 4.3: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, One Railing with Edge Loading E1

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➢ Span =54ft - Three Lanes - One Railing - E2

Table 4.4a: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, One Railing with Edge Loading E2

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	70.6	63.8	66.1	66.4	67.0	67.9	50.2	57.7
1	71.8	64.9	67.3	67.6	68.2	69.1	50.2	57.7
2	68.0	61.1	63.4	63.8	64.3	65.3	50.2	57.7
3	66.4	59.3	61.7	62.1	62.6	63.6	50.2	57.7
4	65.5	58.4	60.8	61.2	61.8	62.7	50.2	57.7
5	65.2	58.0	60.4	60.8	61.4	62.4	50.2	57.7
6	65.7	58.3	60.8	61.2	61.8	62.8	50.2	57.7
7	68.0	60.5	63.1	63.5	64.1	65.1	50.2	57.7
8	65.1	57.4	60.1	60.5	61.2	62.2	50.2	57.7
9	64.1	56.3	59.1	59.5	60.1	61.2	50.2	57.7
10	64.3	56.3	59.1	59.6	60.3	61.3	50.2	57.7
11	66.4	58.2	61.2	61.6	62.3	63.4	50.2	57.7
12	63.1	54.8	57.9	58.3	59.1	60.2	50.2	57.7
13	61.8	53.3	56.5	56.9	57.7	58.9	50.2	57.7
14	61.2	52.4	55.8	56.3	57.1	58.3	50.2	57.7
15	61.0	52.0	55.6	56.1	56.9	58.2	50.2	57.7
16	61.6	52.4	56.1	56.6	57.5	58.8	50.2	57.7
17	64.0	54.5	58.5	59.1	60.0	61.4	50.2	57.7
18	61.1	51.4	55.6	56.2	57.2	58.6	50.2	57.7
19	60.2	50.2	54.6	55.3	56.3	57.8	50.2	57.7
20	60.3	50.0	54.8	55.5	56.6	58.1	50.2	57.7
21	62.4	51.8	57.0	57.7	58.9	60.5	50.2	57.7
22	59.2	48.3	53.9	54.7	55.9	57.5	50.2	57.7
23	57.8	46.5	52.7	53.6	54.8	56.5	50.2	57.7
24	57.1	45.5	52.3	53.2	54.6	56.3	50.2	57.7
25	56.8	44.9	52.4	53.4	54.9	56.6	50.2	57.7
26	57.2	44.9	53.4	54.5	56.0	57.7	50.2	57.7
27	59.4	46.7	56.3	57.5	59.2	60.8	50.2	57.7
28	56.2	43.1	54.1	55.5	57.2	58.6	50.2	57.7
29	54.8	41.2	54.1	55.6	57.4	58.3	50.2	57.7
30	53.9	39.9	55.2	57.0	58.6	58.7	50.2	57.7
31	53.3	38.8	57.5	59.4	60.7	59.6	50.2	57.7
32	52.8	37.8	61.3	63.3	63.7	60.6	50.2	57.7
33	52.5	37.0	67.6	69.4	67.6	61.6	50.2	57.7
34	52.2	36.3	78.3	78.6	71.6	62.4	50.2	57.7
35	52.1	35.6	96.0	89.6	74.4	63.1	50.2	57.7
36	52.0	35.2	133.0	91.9	77.0	63.6	50.2	57.7

Table 4.4b: FEA Summary Results for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, One Railing with Edge Loading E2

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	71.8	64.9	133.0	91.9	77.0	69.1
	At Deterioration	71.8	64.9	133.0	117.0	105.6	97.9
Maximum Longitudinal Moment (k-ft/ft)		68.0	60.5	63.1	63.5	64.1	65.1
Maximum Live Load Deflection (in)		0.500	0.447	0.464	0.467	0.471	0.477

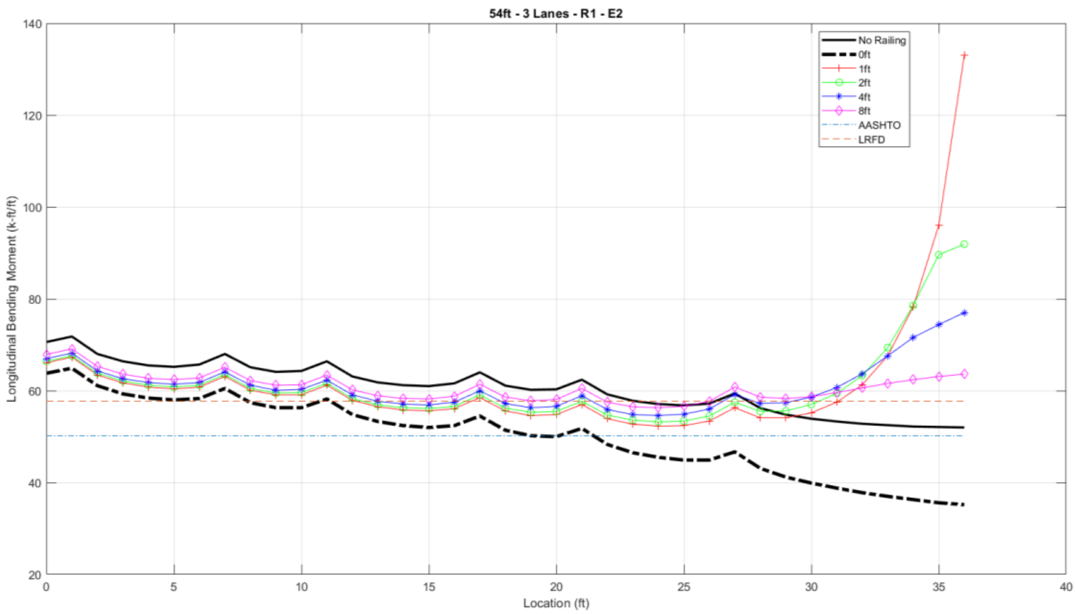

Figure 4.4: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, One Railing with Edge Loading E2

4.2.2 FEA Longitudinal Moments and Deflections for 4 Lanes with One Railing

- 36-4L-R1-E1
- 36-4L-R1-E2
- 54-4L-R1-E1
- 54-4L-R1-E2

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➢ <u>Span =36ft - Four Lanes - One Railing - E1</u>

Table 4.5a: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, One Railing with Edge Loading E1

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	40.7	28.0	74.9	54.7	49.3	44.1	32.4	30.8
1	42.1	30.4	58.7	57.3	50.0	45.3	32.4	30.8
2	38.4	27.9	47.3	47.7	45.2	41.4	32.4	30.8
3	36.9	27.2	41.3	42.2	41.8	39.5	32.4	30.8
4	36.2	27.2	38.0	38.9	39.3	38.3	32.4	30.8
5	36.0	27.7	36.2	37.1	37.8	37.6	32.4	30.8
6	36.6	28.9	35.9	36.6	37.4	37.7	32.4	30.8
7	39.1	31.9	37.7	38.4	39.2	39.7	32.4	30.8
8	36.3	29.6	34.6	35.2	35.9	36.6	32.4	30.8
9	35.5	29.2	33.5	34.0	34.7	35.4	32.4	30.8
10	35.8	29.9	33.6	34.1	34.8	35.5	32.4	30.8
11	38.0	32.5	35.8	36.2	36.9	37.6	32.4	30.8
12	34.9	29.8	32.7	33.1	33.7	34.4	32.4	30.8
13	33.7	28.9	31.6	31.9	32.4	33.1	32.4	30.8
14	33.3	28.7	31.2	31.5	32.0	32.6	32.4	30.8
15	33.2	29.0	31.2	31.5	31.9	32.6	32.4	30.8
16	34.0	30.0	32.0	32.3	32.7	33.3	32.4	30.8
17	36.5	32.8	34.6	34.9	35.3	35.8	32.4	30.8
18	33.8	30.3	32.0	32.2	32.6	33.1	32.4	30.8
19	32.9	29.7	31.2	31.5	31.8	32.3	32.4	30.8
20	33.2	30.2	31.6	31.8	32.1	32.6	32.4	30.8
21	35.4	32.6	33.9	34.1	34.4	34.8	32.4	30.8
22	32.4	29.7	30.9	31.1	31.3	31.7	32.4	30.8
23	31.1	28.6	29.7	29.9	30.1	30.5	32.4	30.8
24	30.6	28.2	29.3	29.4	29.7	30.0	32.4	30.8
25	30.5	28.3	29.3	29.4	29.6	29.9	32.4	30.8
26	31.1	29.0	29.9	30.1	30.3	30.6	32.4	30.8
27	33.5	31.6	32.5	32.6	32.8	33.1	32.4	30.8
28	30.7	28.9	29.7	29.8	30.0	30.2	32.4	30.8
29	29.7	28.0	28.8	28.9	29.0	29.3	32.4	30.8
30	29.9	28.3	29.0	29.1	29.2	29.5	32.4	30.8
31	31.9	30.4	31.1	31.2	31.3	31.6	32.4	30.8
32	28.7	27.3	27.9	28.0	28.1	28.3	32.4	30.8
33	27.2	25.9	26.5	26.6	26.7	26.9	32.4	30.8
34	26.5	25.2	25.8	25.8	26.0	26.2	32.4	30.8
35	26.1	25.0	25.5	25.5	25.7	25.8	32.4	30.8
36	26.5	25.4	25.8	25.9	26.0	26.2	32.4	30.8
37	28.6	27.5	28.0	28.0	28.1	28.3	32.4	30.8
38	25.2	24.2	24.7	24.7	24.8	25.0	32.4	30.8
39	23.7	22.7	23.1	23.2	23.3	23.4	32.4	30.8
40	22.7	21.8	22.1	22.2	22.3	22.4	32.4	30.8
41	21.9	21.0	21.4	21.4	21.5	21.7	32.4	30.8
42	21.3	20.4	20.8	20.9	20.9	21.1	32.4	30.8
43	20.8	20.0	20.3	20.4	20.5	20.6	32.4	30.8
44	20.4	19.6	19.9	20.0	20.1	20.2	32.4	30.8
45	20.1	19.3	19.6	19.7	19.8	19.9	32.4	30.8
46	19.8	19.1	19.4	19.5	19.5	19.6	32.4	30.8
47	19.6	19.0	19.3	19.3	19.4	19.5	32.4	30.8
48	19.5	18.9	19.1	19.2	19.3	19.4	32.4	30.8

Table 4.5b: FEA Summary Results for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, One Railing with Edge Loading E1

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	42.1	30.4	74.9	57.3	50.0	45.3
	At Deterioration	42.1	30.4	72.4	66.5	58.3	51.3
Maximum Longitudinal Moment (k-ft/ft)		39.1	32.8	41.3	42.2	41.8	39.7
Maximum Live Load Deflection (in)		0.243	0.173	0.213	0.216	0.221	0.230

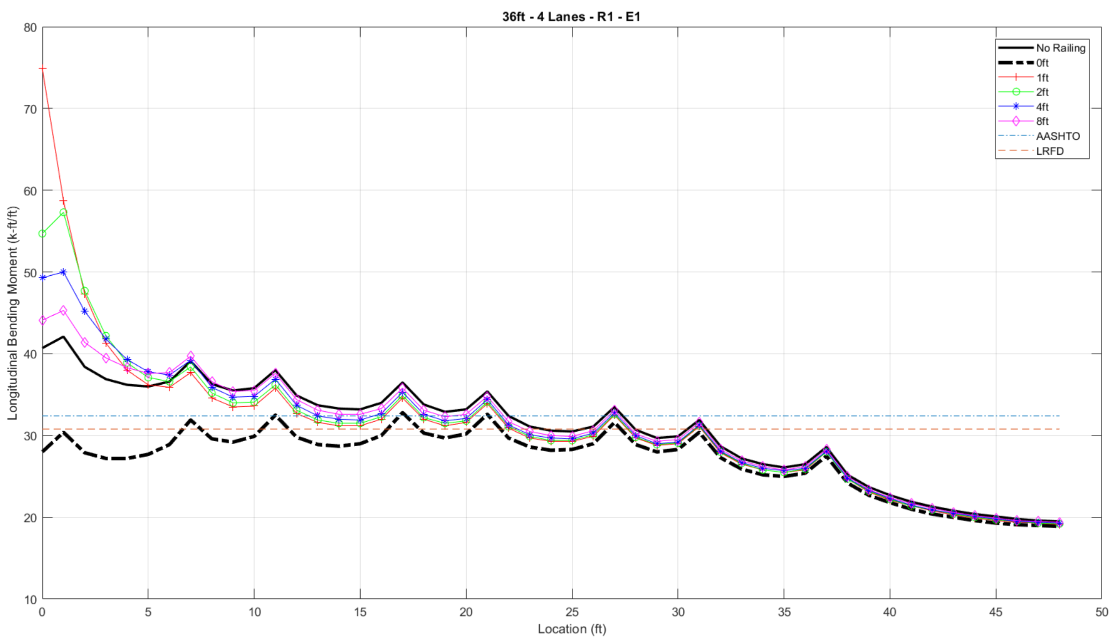

Figure 4.5: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, One Railing with Edge Loading E1

➢ Span =36ft - Four Lanes - One Railing - E2

Table 4.6a: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, One Railing with Edge Loading E2

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	40.7	40.4	40.5	40.6	40.6	40.7	32.4	30.8
1	42.1	41.7	41.8	41.9	41.9	42.0	32.4	30.8
2	38.4	38.0	38.2	38.2	38.2	38.3	32.4	30.8
3	36.9	36.5	36.6	36.6	36.7	36.7	32.4	30.8
4	36.2	35.7	35.9	35.9	36.0	36.0	32.4	30.8
5	36.0	35.5	35.7	35.8	35.8	35.9	32.4	30.8
6	36.6	36.1	36.3	36.4	36.4	36.5	32.4	30.8
7	39.1	38.6	38.8	38.8	38.9	39.0	32.4	30.8
8	36.3	35.8	36.0	36.0	36.1	36.2	32.4	30.8
9	35.5	34.9	35.2	35.2	35.2	35.3	32.4	30.8
10	35.8	35.2	35.4	35.5	35.5	35.6	32.4	30.8
11	38.0	37.4	37.7	37.7	37.7	37.8	32.4	30.8
12	34.9	34.3	34.6	34.6	34.7	34.8	32.4	30.8
13	33.7	33.1	33.4	33.4	33.5	33.6	32.4	30.8
14	33.3	32.6	32.9	32.9	33.0	33.1	32.4	30.8
15	33.2	32.5	32.8	32.9	32.9	33.1	32.4	30.8
16	34.0	33.2	33.5	33.6	33.6	33.8	32.4	30.8
17	36.5	35.7	36.0	36.1	36.2	36.3	32.4	30.8
18	33.8	32.9	33.3	33.3	33.4	33.5	32.4	30.8
19	32.9	32.0	32.4	32.5	32.6	32.7	32.4	30.8
20	33.2	32.3	32.7	32.7	32.8	33.0	32.4	30.8
21	35.4	34.4	34.9	34.9	35.0	35.2	32.4	30.8
22	32.4	31.2	31.7	31.8	31.9	32.1	32.4	30.8
23	31.1	29.9	30.4	30.5	30.6	30.8	32.4	30.8
24	30.6	29.3	29.8	29.9	30.0	30.2	32.4	30.8
25	30.5	29.1	29.7	29.8	29.9	30.1	32.4	30.8
26	31.1	29.6	30.3	30.4	30.5	30.7	32.4	30.8
27	33.5	32.0	32.7	32.8	32.9	33.2	32.4	30.8
28	30.7	29.0	29.8	29.9	30.0	30.3	32.4	30.8
29	29.7	27.9	28.8	28.9	29.1	29.3	32.4	30.8
30	29.9	28.0	28.9	29.0	29.2	29.5	32.4	30.8
31	31.9	29.9	30.9	31.0	31.2	31.5	32.4	30.8
32	28.7	26.5	27.6	27.7	27.9	28.3	32.4	30.8
33	27.2	24.9	26.1	26.2	26.5	26.8	32.4	30.8
34	26.5	24.0	25.3	25.5	25.7	26.1	32.4	30.8
35	26.1	23.5	24.9	25.1	25.4	25.8	32.4	30.8
36	26.5	23.6	25.2	25.4	25.8	26.2	32.4	30.8
37	28.6	25.6	27.3	27.6	27.9	28.3	32.4	30.8
38	25.2	22.0	24.1	24.3	24.7	25.1	32.4	30.8
39	23.7	20.2	22.5	22.8	23.2	23.6	32.4	30.8
40	22.7	19.0	21.7	22.0	22.4	22.8	32.4	30.8
41	21.9	18.0	21.1	21.5	21.9	22.2	32.4	30.8
42	21.3	17.1	20.9	21.3	21.7	21.9	32.4	30.8
43	20.8	16.4	21.0	21.4	21.8	21.7	32.4	30.8
44	20.4	15.7	21.5	22.0	22.2	21.6	32.4	30.8
45	20.1	15.1	22.6	23.1	22.9	21.6	32.4	30.8
46	19.8	14.6	24.9	25.1	23.7	21.5	32.4	30.8
47	19.6	14.0	29.1	28.3	24.2	21.5	32.4	30.8
48	19.5	13.6	38.3	27.4	24.4	21.4	32.4	30.8

Table 4.6b: FEA Summary Results for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, One Railing with Edge Loading E2

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	42.1	41.7	41.8	41.9	41.9	42.0
	At Deterioration	42.1	41.7	37.0	34.6	32.1	28.6
Maximum Longitudinal Moment (k-ft/ft)		39.1	38.6	38.8	38.8	38.9	39.0
Maximum Live Load Deflection (in)		0.243	0.240	0.241	0.241	0.242	0.242

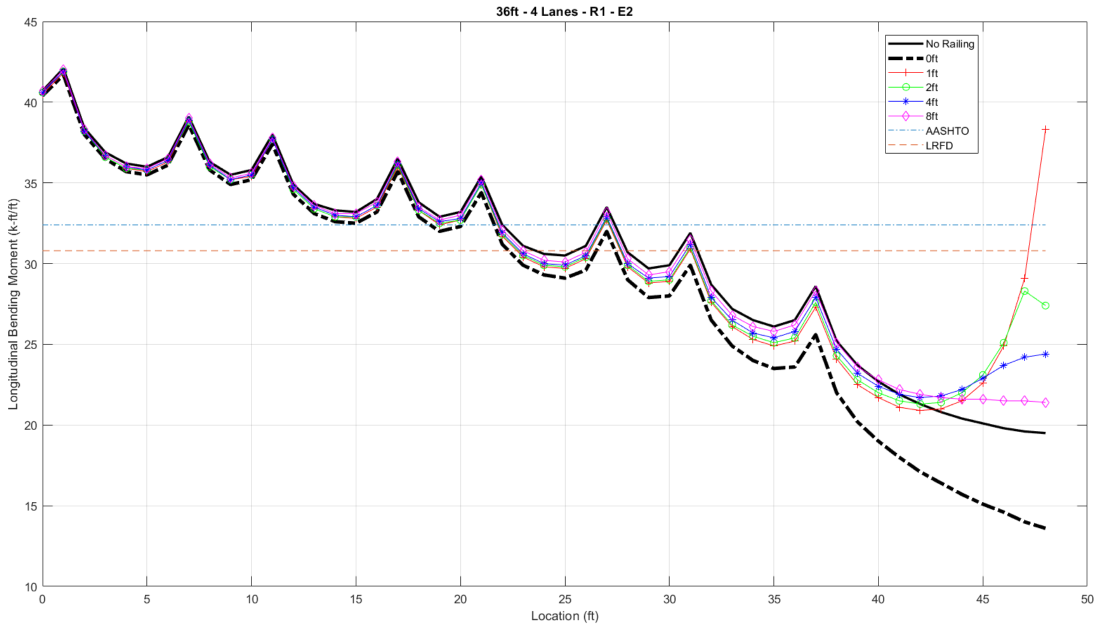

Figure 4.6: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, One Railing with Edge Loading E2

- <u>Span =54ft - Four Lanes - One Railing - E1</u>

Table 4.7a: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, One Railing with Edge Loading E1

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	73.5	63.4	114.7	94.1	87.3	80.2	50.2	57.7
1	74.6	65.2	99.4	96.7	87.7	81.2	50.2	57.7
2	70.8	62.1	85.1	85.6	82.3	77.1	50.2	57.7
3	69.1	60.9	77.7	78.8	78.0	74.7	50.2	57.7
4	68.3	60.5	73.3	74.4	74.7	73.1	50.2	57.7
5	68.0	60.5	70.6	71.6	72.4	71.8	50.2	57.7
6	68.4	61.3	69.5	70.4	71.3	71.4	50.2	57.7
7	70.8	64.0	70.7	71.5	72.5	72.9	50.2	57.7
8	67.8	61.3	67.0	67.7	68.6	69.4	50.2	57.7
9	66.8	60.6	65.5	66.1	67.0	67.8	50.2	57.7
10	67.0	61.0	65.3	65.8	66.6	67.5	50.2	57.7
11	69.1	63.3	67.1	67.5	68.3	69.2	50.2	57.7
12	65.9	60.3	63.7	64.1	64.8	65.6	50.2	57.7
13	64.5	59.2	62.2	62.6	63.2	64.0	50.2	57.7
14	63.9	58.8	61.5	61.8	62.4	63.2	50.2	57.7
15	63.8	58.8	61.3	61.6	62.1	62.9	50.2	57.7
16	64.3	59.6	61.8	62.1	62.6	63.3	50.2	57.7
17	66.8	62.2	64.2	64.5	65.0	65.7	50.2	57.7
18	63.9	59.5	61.4	61.6	62.1	62.7	50.2	57.7
19	62.9	58.7	60.4	60.7	61.1	61.7	50.2	57.7
20	63.1	59.0	60.6	60.9	61.2	61.9	50.2	57.7
21	65.2	61.3	62.8	63.0	63.3	63.9	50.2	57.7
22	62.0	58.2	59.6	59.8	60.2	60.7	50.2	57.7
23	60.6	57.0	58.3	58.5	58.8	59.3	50.2	57.7
24	60.0	56.5	57.7	57.9	58.2	58.7	50.2	57.7
25	59.8	56.4	57.6	57.8	58.0	58.5	50.2	57.7
26	60.4	57.0	58.2	58.3	58.6	59.1	50.2	57.7
27	62.7	59.5	60.6	60.8	61.0	61.5	50.2	57.7
28	59.8	56.7	57.7	57.9	58.1	58.6	50.2	57.7
29	58.8	55.8	56.8	56.9	57.2	57.6	50.2	57.7
30	58.9	56.0	56.9	57.1	57.3	57.7	50.2	57.7
31	60.9	58.1	59.0	59.1	59.3	59.6	50.2	57.7
32	57.6	54.9	55.7	55.9	56.1	56.4	50.2	57.7
33	56.1	53.5	54.3	54.4	54.6	54.9	50.2	57.7
34	55.4	52.8	53.6	53.7	53.9	54.2	50.2	57.7
35	55.0	52.5	53.3	53.4	53.6	53.8	50.2	57.7
36	55.3	52.9	53.6	53.7	53.9	54.2	50.2	57.7
37	57.4	55.0	55.8	55.9	56.0	56.4	50.2	57.7
38	54.1	51.8	52.5	52.6	52.8	53.1	50.2	57.7
39	52.5	50.3	51.0	51.1	51.2	51.5	50.2	57.7
40	51.5	49.3	50.0	50.1	50.3	50.5	50.2	57.7
41	50.8	48.6	49.3	49.4	49.5	49.8	50.2	57.7
42	50.2	48.1	48.7	48.8	49.0	49.2	50.2	57.7
43	49.7	47.7	48.3	48.4	48.5	48.8	50.2	57.7
44	49.4	47.3	47.9	48.0	48.2	48.4	50.2	57.7
45	49.1	47.1	47.7	47.8	47.9	48.2	50.2	57.7
46	48.9	46.9	47.5	47.6	47.7	48.0	50.2	57.7
47	48.8	46.8	47.4	47.5	47.6	47.9	50.2	57.7
48	48.7	46.8	47.4	47.4	47.6	47.8	50.2	57.7

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Table 4.7b: FEA Summary Results for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, One Railing with Edge Loading E1

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	74.6	65.2	114.7	96.7	87.7	81.2
	At Deterioration	74.6	65.2	114.7	109.4	102.3	95.8
Maximum Longitudinal Moment (k-ft/ft)		70.8	64.0	77.7	78.8	78.0	74.7
Maximum Live Load Deflection (in)		0.522	0.455	0.486	0.489	0.493	0.500

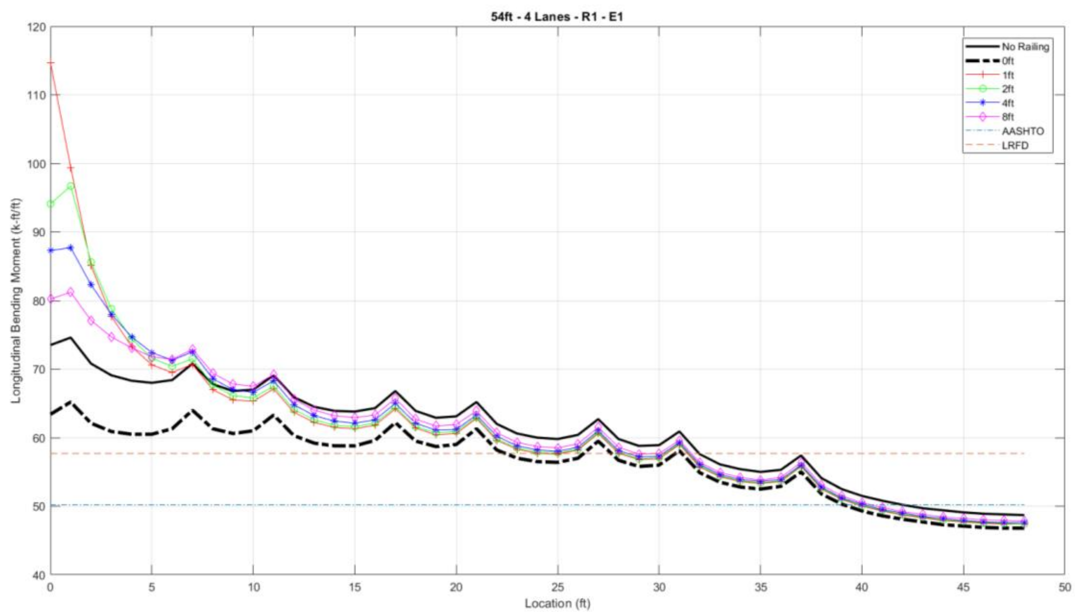

Figure 4.7: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, One Railing with Edge Loading E1

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➢ Span =54ft - Four Lanes - One Railing - E2

Table 4.8a: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, One Railing with Edge Loading E2

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	73.5	72.1	72.5	72.5	72.6	72.8	50.2	57.7
1	74.6	73.2	73.6	73.7	73.8	74.0	50.2	57.7
2	70.8	69.4	69.8	69.9	70.0	70.2	50.2	57.7
3	69.1	67.7	68.1	68.2	68.3	68.5	50.2	57.7
4	68.3	66.8	67.3	67.3	67.4	67.6	50.2	57.7
5	68.0	66.5	66.9	67.0	67.1	67.3	50.2	57.7
6	68.4	66.9	67.3	67.4	67.5	67.7	50.2	57.7
7	70.8	69.2	69.7	69.7	69.8	70.1	50.2	57.7
8	67.8	66.2	66.7	66.8	66.9	67.1	50.2	57.7
9	66.8	65.2	65.7	65.8	65.9	66.1	50.2	57.7
10	67.0	65.3	65.8	65.9	66.0	66.2	50.2	57.7
11	69.1	67.4	67.9	67.9	68.1	68.3	50.2	57.7
12	65.9	64.1	64.6	64.7	64.8	65.1	50.2	57.7
13	64.5	62.7	63.3	63.3	63.5	63.7	50.2	57.7
14	63.9	62.0	62.6	62.7	62.8	63.1	50.2	57.7
15	63.8	61.8	62.4	62.5	62.7	62.9	50.2	57.7
16	64.3	62.4	63.0	63.1	63.2	63.5	50.2	57.7
17	66.8	64.7	65.4	65.4	65.6	65.9	50.2	57.7
18	63.9	61.8	62.5	62.5	62.7	63.0	50.2	57.7
19	62.9	60.8	61.5	61.6	61.7	62.0	50.2	57.7
20	63.1	60.9	61.6	61.7	61.9	62.2	50.2	57.7
21	65.2	62.9	63.7	63.8	64.0	64.3	50.2	57.7
22	62.0	59.6	60.4	60.5	60.7	61.1	50.2	57.7
23	60.6	58.2	59.0	59.1	59.3	59.7	50.2	57.7
24	60.0	57.5	58.3	58.5	58.7	59.1	50.2	57.7
25	59.8	57.2	58.1	58.3	58.5	58.9	50.2	57.7
26	60.4	57.6	58.6	58.8	59.0	59.4	50.2	57.7
27	62.7	59.9	61.0	61.1	61.4	61.8	50.2	57.7
28	59.8	56.9	58.0	58.2	58.4	58.9	50.2	57.7
29	58.8	55.7	57.0	57.1	57.4	57.9	50.2	57.7
30	58.9	55.7	57.1	57.2	57.6	58.0	50.2	57.7
31	60.9	57.5	59.1	59.3	59.6	60.1	50.2	57.7
32	57.6	54.2	55.8	56.0	56.3	56.9	50.2	57.7
33	56.1	52.6	54.3	54.5	54.9	55.5	50.2	57.7
34	55.4	51.7	53.6	53.8	54.2	54.8	50.2	57.7
35	55.0	51.2	53.3	53.6	54.0	54.6	50.2	57.7
36	55.3	51.4	53.7	54.0	54.5	55.1	50.2	57.7
37	57.4	53.3	55.9	56.3	56.8	57.5	50.2	57.7
38	54.1	49.8	52.8	53.2	53.8	54.4	50.2	57.7
39	52.5	48.1	51.6	52.0	52.6	53.2	50.2	57.7
40	51.5	46.9	51.0	51.5	52.1	52.6	50.2	57.7
41	50.8	46.0	50.8	51.3	52.0	52.4	50.2	57.7
42	50.2	45.2	51.0	51.6	52.3	52.4	50.2	57.7
43	49.7	44.6	51.7	52.4	53.0	52.6	50.2	57.7
44	49.4	44.0	53.0	53.8	54.0	52.9	50.2	57.7
45	49.1	43.5	55.4	56.1	55.6	53.2	50.2	57.7
46	48.9	43.1	59.2	59.7	57.3	53.5	50.2	57.7
47	48.8	42.8	67.1	64.9	58.3	53.6	50.2	57.7
48	48.7	42.4	78.5	63.7	58.8	53.6	50.2	57.7

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Table 4.8b: FEA Summary Results for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, One Railing with Edge Loading E2

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	74.6	73.2	78.5	73.7	73.8	74.0
	At Deterioration	74.6	73.2	78.5	76.1	72.3	67.2
Maximum Longitudinal Moment (k-ft/ft)		70.8	69.2	69.7	69.7	69.8	70.1
Maximum Live Load Deflection (in)		0.522	0.511	0.514	0.515	0.515	0.517

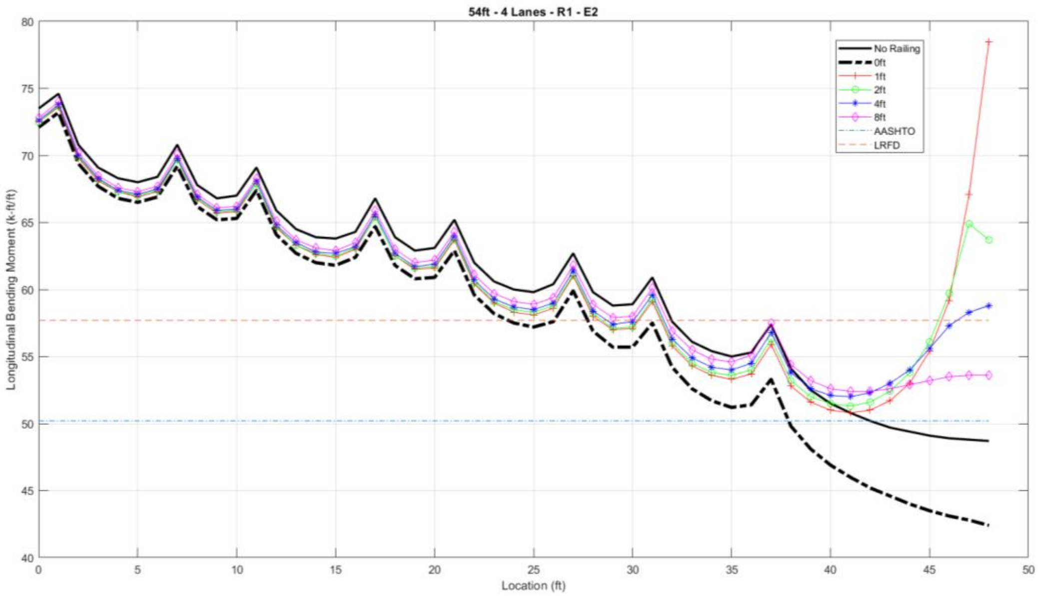

Figure 4.8: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, One Railing with Edge Loading E2

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4.2.3 FEA Longitudinal Moments and Deflections for 3 Lanes with Two Railings

- 36-3L-R2-E1
- 54-3L-R2-E1

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➢ Span =36ft - Three Lanes - Two Railings - E1

Table 4.9a: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 36ft, Two Railings with Edge Loading E1

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	39.3	26.2	67.8	51.3	46.3	41.4	32.4	32.6
1	40.6	28.6	50.6	53.8	46.9	42.6	32.4	32.6
2	37.0	25.9	43.2	44.5	42.2	38.7	32.4	32.6
3	35.4	25.1	38.1	39.2	38.8	36.7	32.4	32.6
4	34.7	25.1	35.0	36.1	36.4	35.5	32.4	32.6
5	34.5	25.4	33.2	34.3	34.9	34.7	32.4	32.6
6	35.0	26.5	32.2	33.8	34.5	34.8	32.4	32.6
7	37.5	29.4	30.7	35.5	36.3	36.8	32.4	32.6
8	34.7	26.9	30.7	32.2	32.9	33.5	32.4	32.6
9	33.8	26.4	30.0	31.0	31.6	32.3	32.4	32.6
10	34.0	27.0	29.6	31.0	31.6	32.3	32.4	32.6
11	36.2	29.4	28.3	33.0	33.5	34.2	32.4	32.6
12	33.1	26.5	28.5	29.7	30.3	30.9	32.4	32.6
13	31.8	25.5	27.7	28.3	28.8	29.5	32.4	32.6
14	31.3	25.1	27.2	27.7	28.2	28.8	32.4	32.6
15	31.2	25.2	27.0	27.6	28.0	28.6	32.4	32.6
16	31.8	26.0	27.0	28.1	28.5	29.1	32.4	32.6
17	34.3	28.6	26.1	30.6	30.9	31.4	32.4	32.6
18	31.5	25.8	26.5	27.7	28.0	28.5	32.4	32.6
19	30.6	25.0	26.0	26.7	27.0	27.5	32.4	32.6
20	30.7	25.2	25.6	26.8	27.1	27.5	32.4	32.6
21	32.9	27.3	24.4	28.8	29.1	29.5	32.4	32.6
22	29.6	24.1	24.4	25.5	25.7	26.1	32.4	32.6
23	28.3	22.7	23.5	24.0	24.2	24.6	32.4	32.6
24	27.6	22.0	22.8	23.2	23.4	23.7	32.4	32.6
25	27.3	21.6	22.3	22.7	23.0	23.3	32.4	32.6
26	27.7	22.0	22.0	23.0	23.2	23.5	32.4	32.6
27	29.9	24.0	20.7	25.0	25.2	25.5	32.4	32.6
28	26.7	20.6	20.6	21.6	21.7	22.0	32.4	32.6
29	25.2	19.0	19.5	19.9	20.0	20.3	32.4	32.6
30	24.3	17.9	18.5	18.7	18.9	19.1	32.4	32.6
31	23.6	17.0	17.7	17.8	18.0	18.2	32.4	32.6
32	23.1	16.3	17.0	17.0	17.2	17.4	32.4	32.6
33	22.8	15.7	16.3	16.4	16.5	16.7	32.4	32.6
34	22.5	15.1	15.7	15.8	15.9	16.1	32.4	32.6
35	22.3	14.6	15.2	15.2	15.4	15.6	32.4	32.6
36	22.2	14.3	14.8	14.9	15.0	15.2	32.4	32.6

Table 4.9b: FEA Summary Results for Three-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 36ft, Two Railings with Edge Loading E1

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	40.6	28.6	67.8	53.8	46.9	42.6
	At Deterioration	40.6	28.6	67.1	62.1	55.3	47.8
Maximum Longitudinal Moment (k-ft/ft)		37.5	29.4	43.2	44.5	42.2	38.7
Maximum Live Load Deflection (in)		0.233	0.157	0.198	0.202	0.206	0.214

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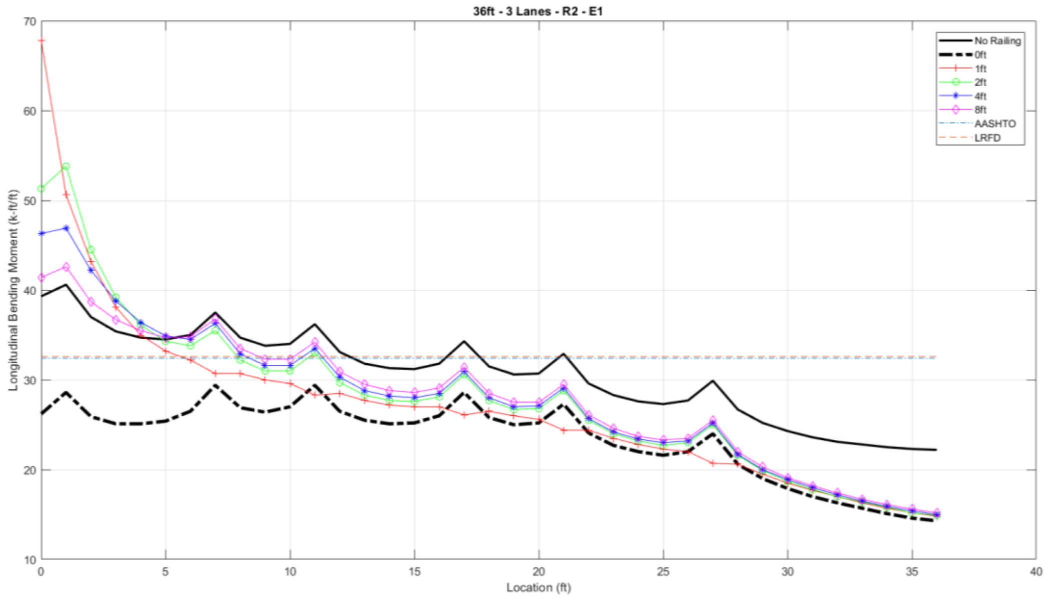

Figure 4.9: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 36ft, Two Railings with Edge Loading E1

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➢ Span =54ft - Three Lanes - Two Railings - E1

Table 4.10a: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, Two Railings with Edge Loading E1

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	70.6	41.9	147.3	109.2	92.0	77.0	50.2	57.7
1	71.8	44.1	111.3	106.2	89.5	77.2	50.2	57.7
2	68.0	41.7	89.5	90.3	82.2	72.2	50.2	57.7
3	66.4	41.0	76.2	78.1	75.9	69.1	50.2	57.7
4	65.5	41.0	68.1	70.1	70.4	66.8	50.2	57.7
5	65.2	41.4	62.8	64.9	66.2	64.8	50.2	57.7
6	65.7	42.5	59.4	61.9	63.7	63.6	50.2	57.7
7	68.0	45.4	56.1	61.8	63.6	64.4	50.2	57.7
8	65.1	42.9	54.7	57.0	58.8	60.1	50.2	57.7
9	64.1	42.4	53.0	54.6	56.4	57.9	50.2	57.7
10	64.3	42.9	51.7	53.7	55.3	57.0	50.2	57.7
11	66.4	45.3	49.8	54.9	56.5	58.2	50.2	57.7
12	63.1	42.4	49.3	51.0	52.5	54.2	50.2	57.7
13	61.8	41.3	48.1	49.1	50.5	52.2	50.2	57.7
14	61.2	40.9	47.2	48.0	49.3	51.0	50.2	57.7
15	61.0	41.0	46.6	47.5	48.7	50.3	50.2	57.7
16	61.6	41.7	46.3	47.7	48.8	50.4	50.2	57.7
17	64.0	44.2	45.1	49.8	50.9	52.4	50.2	57.7
18	61.1	41.5	45.2	46.6	47.6	49.1	50.2	57.7
19	60.2	40.6	44.6	45.4	46.4	47.8	50.2	57.7
20	60.3	40.8	44.0	45.3	46.2	47.5	50.2	57.7
21	62.4	42.8	42.5	47.1	48.0	49.3	50.2	57.7
22	59.2	39.6	42.4	43.6	44.5	45.7	50.2	57.7
23	57.8	38.1	41.4	42.0	42.8	43.9	50.2	57.7
24	57.1	37.4	40.5	41.0	41.8	42.9	50.2	57.7
25	56.8	37.0	39.9	40.5	41.2	42.3	50.2	57.7
26	57.2	37.3	39.5	40.6	41.3	42.3	50.2	57.7
27	59.4	39.3	38.1	42.5	43.1	44.2	50.2	57.7
28	56.2	35.9	37.9	39.0	39.6	40.6	50.2	57.7
29	54.8	34.2	36.7	37.2	37.8	38.7	50.2	57.7
30	53.9	33.1	35.7	35.9	36.5	37.4	50.2	57.7
31	53.3	32.2	34.7	34.9	35.5	36.4	50.2	57.7
32	52.8	31.4	33.9	34.1	34.6	35.5	50.2	57.7
33	52.5	30.7	33.2	33.3	33.9	34.7	50.2	57.7
34	52.2	30.2	32.5	32.7	33.2	34.0	50.2	57.7
35	52.1	29.6	31.9	32.0	32.5	33.3	50.2	57.7
36	52.0	29.3	31.5	31.8	32.3	32.9	50.2	57.7

Table 4.10b: FEA Summary Results for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, Two Railings with Edge Loading E1

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	71.8	44.1	147.3	109.2	92.0	77.2
	At Deterioration	71.8	44.1	147.3	135.2	120.9	108.1
Maximum Longitudinal Moment (k-ft/ft)		68.0	45.4	76.2	78.1	75.9	69.1
Maximum Live Load Deflection (in)		0.500	0.297	0.359	0.366	0.376	0.394

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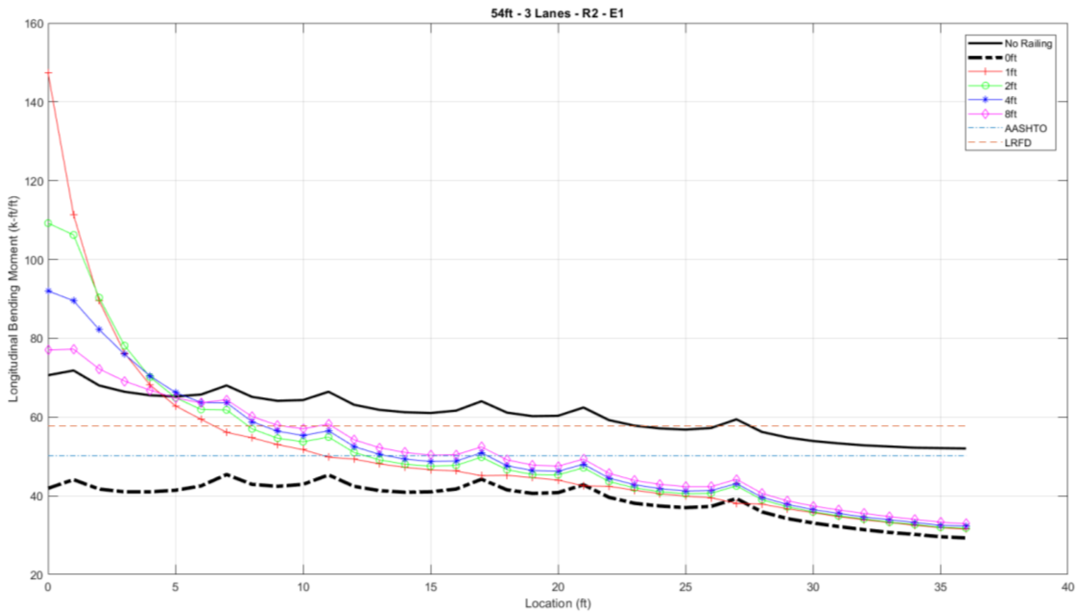

Figure 4.10: Longitudinal Moment Distribution at Critical Section for Three-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 36ft, Two Railings with Edge Loading E1

4.2.4 FEA Longitudinal Moments and Deflections for 4 Lanes with Two Railings

- 36-4L-R2-E1
- 54-4L-R2-E1

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➢ Span =36ft - Four Lanes - Two Railings - E1

Table 4.11a: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, Two Railings with Edge Loading E1

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	40.7	27.7	71.8	54.2	48.9	43.7	32.4	30.8
1	42.1	30.1	53.8	56.8	49.6	44.9	32.4	30.8
2	38.4	27.6	46.0	47.3	44.8	41.0	32.4	30.8
3	36.9	26.9	40.6	41.8	41.3	39.1	32.4	30.8
4	36.2	26.9	37.4	38.5	38.9	37.8	32.4	30.8
5	36.0	27.3	35.6	36.7	37.4	37.1	32.4	30.8
6	36.6	28.5	34.6	36.2	37.0	37.2	32.4	30.8
7	39.1	31.5	33.1	37.9	38.7	39.2	32.4	30.8
8	36.3	29.1	33.1	34.7	35.4	36.1	32.4	30.8
9	35.5	28.7	32.5	33.5	34.2	34.9	32.4	30.8
10	35.8	29.4	32.2	33.6	34.2	35.0	32.4	30.8
11	38.0	32.0	31.0	35.7	36.3	37.0	32.4	30.8
12	34.9	29.2	31.3	32.5	33.1	33.8	32.4	30.8
13	33.7	28.3	30.7	31.3	31.8	32.5	32.4	30.8
14	33.3	28.1	30.3	30.8	31.3	31.9	32.4	30.8
15	33.2	28.3	30.2	30.8	31.2	31.8	32.4	30.8
16	34.0	29.3	30.4	31.5	31.9	32.5	32.4	30.8
17	36.5	32.0	29.6	34.1	34.5	35.0	32.4	30.8
18	33.8	29.5	30.2	31.4	31.7	32.2	32.4	30.8
19	32.9	28.8	29.9	30.5	30.9	31.3	32.4	30.8
20	33.2	29.3	29.7	30.9	31.2	31.6	32.4	30.8
21	35.4	31.6	28.7	33.1	33.4	33.8	32.4	30.8
22	32.4	28.6	28.9	30.0	30.2	30.6	32.4	30.8
23	31.1	27.5	28.3	28.7	29.0	29.3	32.4	30.8
24	30.6	27.0	27.8	28.2	28.4	28.7	32.4	30.8
25	30.5	27.0	27.6	28.0	28.3	28.6	32.4	30.8
26	31.1	27.6	27.6	28.6	28.8	29.1	32.4	30.8
27	33.5	30.1	26.7	31.1	31.2	31.5	32.4	30.8
28	30.7	27.3	27.1	28.1	28.3	28.6	32.4	30.8
29	29.7	26.3	26.6	27.1	27.3	27.5	32.4	30.8
30	29.9	26.5	26.1	27.2	27.4	27.6	32.4	30.8
31	31.9	28.5	24.8	29.2	29.3	29.5	32.4	30.8
32	28.7	25.2	24.8	25.8	26.0	26.2	32.4	30.8
33	27.2	23.7	23.9	24.3	24.4	24.6	32.4	30.8
34	26.5	22.8	23.1	23.4	23.5	23.7	32.4	30.8
35	26.1	22.4	22.6	22.9	23.0	23.2	32.4	30.8
36	26.5	22.6	22.2	23.1	23.2	23.4	32.4	30.8
37	28.6	24.6	20.8	25.1	25.2	25.3	32.4	30.8
38	25.2	21.1	20.7	21.6	21.7	21.8	32.4	30.8
39	23.7	19.4	19.5	19.8	19.9	20.0	32.4	30.8
40	22.7	18.2	18.5	18.6	18.7	18.8	32.4	30.8
41	21.9	17.2	17.6	17.6	17.7	17.8	32.4	30.8
42	21.3	16.4	16.7	16.8	16.9	17.0	32.4	30.8
43	20.8	15.7	16.0	16.0	16.1	16.2	32.4	30.8
44	20.4	15.1	15.4	15.4	15.4	15.5	32.4	30.8
45	20.1	14.5	14.8	14.8	14.9	14.9	32.4	30.8
46	19.8	14.0	14.3	14.3	14.3	14.4	32.4	30.8
47	19.6	13.5	13.8	13.8	13.8	13.9	32.4	30.8
48	19.5	13.1	13.3	13.4	13.4	13.5	32.4	30.8

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Table 4.11b: FEA Summary Results for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, Two Railings with Edge Loading E1

FEA Results		No Railing	Railing Deterioration Length (ft)				
			0	1	2	4	8
Edge Beam Moment	At Center	42.1	30.1	71.8	56.8	49.6	44.9
	At Deterioration	42.1	30.1	46.7	65.9	61.0	52.9
Maximum Longitudinal Moment (k-ft/ft)		39.1	32.0	45.9	47.3	44.8	41.0
Maximum Live Load Deflection (in)		0.243	0.168	0.211	0.214	0.219	0.228

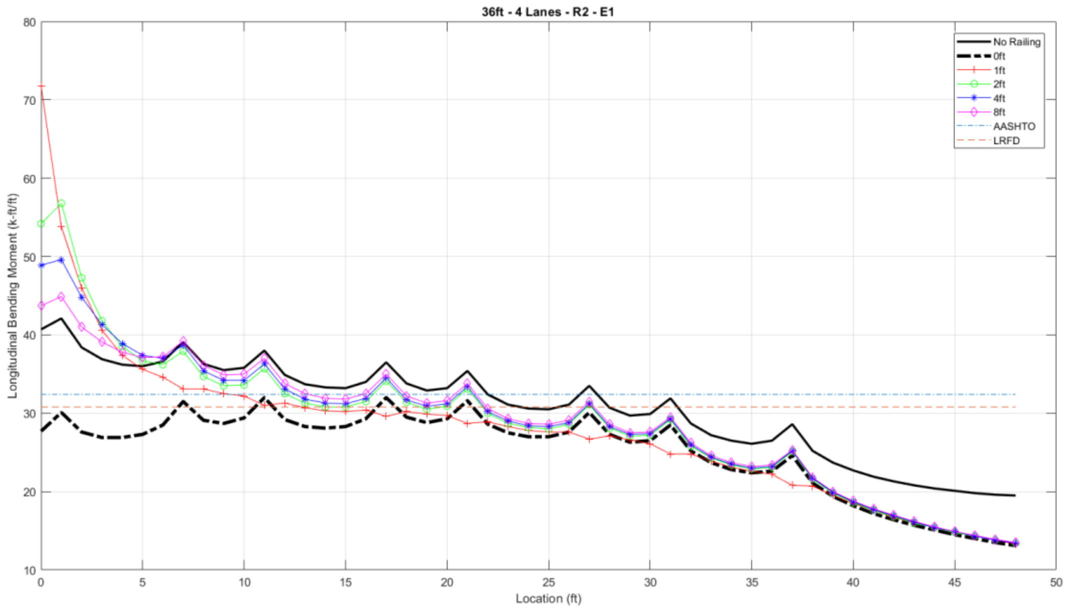

Figure 4.11: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 36ft, Deck Width = 48ft, Two Railings with Edge Loading E1

➢ Span =54ft - Four Lanes - Two Railings - E1

Table 4.12a: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, Two Railings with Edge Loading E1

Location (ft)	FEA Longitudinal Moment at Critical Section (kip-ft/ft)						AASHTO Specs Moment (k-ft/ft)	AASHTO LRFD Moment (k-ft/ft)
	No Railing	Railing Deterioration Length (ft)						
		0	1	2	4	8		
0	73.5	62.2	113.1	92.5	85.7	78.7	50.2	57.7
1	74.6	63.9	97.1	95.1	86.1	79.8	50.2	57.7
2	70.8	60.8	83.7	84.1	80.8	75.6	50.2	57.7
3	69.1	59.6	76.2	77.2	76.5	73.2	50.2	57.7
4	68.3	59.1	71.8	72.8	73.1	71.6	50.2	57.7
5	68.0	59.1	69.1	70.1	70.8	70.3	50.2	57.7
6	68.4	59.9	68.0	68.9	69.7	69.9	50.2	57.7
7	70.8	62.5	69.2	70.0	70.9	71.4	50.2	57.7
8	67.8	59.8	65.5	66.2	67.0	67.8	50.2	57.7
9	66.8	59.1	63.9	64.5	65.3	66.2	50.2	57.7
10	67.0	59.4	63.6	64.2	64.9	65.8	50.2	57.7
11	69.1	61.7	65.4	65.9	66.6	67.5	50.2	57.7
12	65.9	58.7	61.9	62.4	63.0	63.9	50.2	57.7
13	64.5	57.5	60.4	60.8	61.4	62.2	50.2	57.7
14	63.9	57.0	59.7	60.0	60.6	61.4	50.2	57.7
15	63.8	57.0	59.4	59.7	60.2	61.0	50.2	57.7
16	64.3	57.7	59.9	60.2	60.7	61.4	50.2	57.7
17	66.8	60.2	62.3	62.5	63.0	63.7	50.2	57.7
18	63.9	57.5	59.3	59.6	60.0	60.7	50.2	57.7
19	62.9	56.6	58.3	58.6	59.0	59.6	50.2	57.7
20	63.1	56.9	58.5	58.7	59.1	59.7	50.2	57.7
21	65.2	59.0	60.5	60.7	61.1	61.7	50.2	57.7
22	62.0	55.9	57.3	57.5	57.8	58.4	50.2	57.7
23	60.6	54.6	55.9	56.1	56.4	56.9	50.2	57.7
24	60.0	54.0	55.2	55.4	55.7	56.2	50.2	57.7
25	59.8	53.8	55.0	55.2	55.5	55.9	50.2	57.7
26	60.4	54.4	55.5	55.7	55.9	56.4	50.2	57.7
27	62.7	56.8	57.8	58.0	58.2	58.7	50.2	57.7
28	59.8	53.9	54.9	55.0	55.2	55.7	50.2	57.7
29	58.8	52.8	53.8	53.9	54.2	54.6	50.2	57.7
30	58.9	52.9	53.8	54.0	54.2	54.6	50.2	57.7
31	60.9	54.9	55.8	55.9	56.1	56.5	50.2	57.7
32	57.6	51.6	52.4	52.5	52.7	53.1	50.2	57.7
33	56.1	50.0	50.8	51.0	51.2	51.5	50.2	57.7
34	55.4	49.2	50.0	50.1	50.3	50.6	50.2	57.7
35	55.0	48.8	49.5	49.6	49.8	50.1	50.2	57.7
36	55.3	49.0	49.8	49.9	50.0	50.3	50.2	57.7
37	57.4	51.0	51.7	51.8	52.0	52.3	50.2	57.7
38	54.1	47.6	48.3	48.4	48.6	48.9	50.2	57.7
39	52.5	46.0	46.6	46.7	46.9	47.1	50.2	57.7
40	51.5	44.9	45.5	45.6	45.7	46.0	50.2	57.7
41	50.8	44.0	44.6	44.7	44.8	45.1	50.2	57.7
42	50.2	43.3	43.8	43.9	44.1	44.3	50.2	57.7
43	49.7	42.6	43.2	43.3	43.4	43.7	50.2	57.7
44	49.4	42.1	42.7	42.8	42.9	43.1	50.2	57.7
45	49.1	41.7	42.2	42.3	42.4	42.7	50.2	57.7
46	48.9	41.3	41.8	41.9	42.1	42.3	50.2	57.7
47	48.8	41.0	41.5	41.6	41.7	41.9	50.2	57.7
48	48.7	40.8	41.3	41.4	41.5	41.7	50.2	57.7

Table 4.12b: FEA Summary Results for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, Two Railings with Edge Loading E1

FEA Results		No Railing	Railing Deterioration Length (ft)					
			0	1	2	4	8
Edge Beam Moment	At Center	74.6	63.9	113.1	95.1	86.1	79.8
	At Deterioration	74.6	63.9	113.1	107.6	100.7	94.1
Maximum Longitudinal Moment (k-ft/ft)		70.8	62.5	76.2	77.2	76.5	73.2
Maximum Live Load Deflection (in)		0.522	0.445	0.476	0.479	0.482	0.489

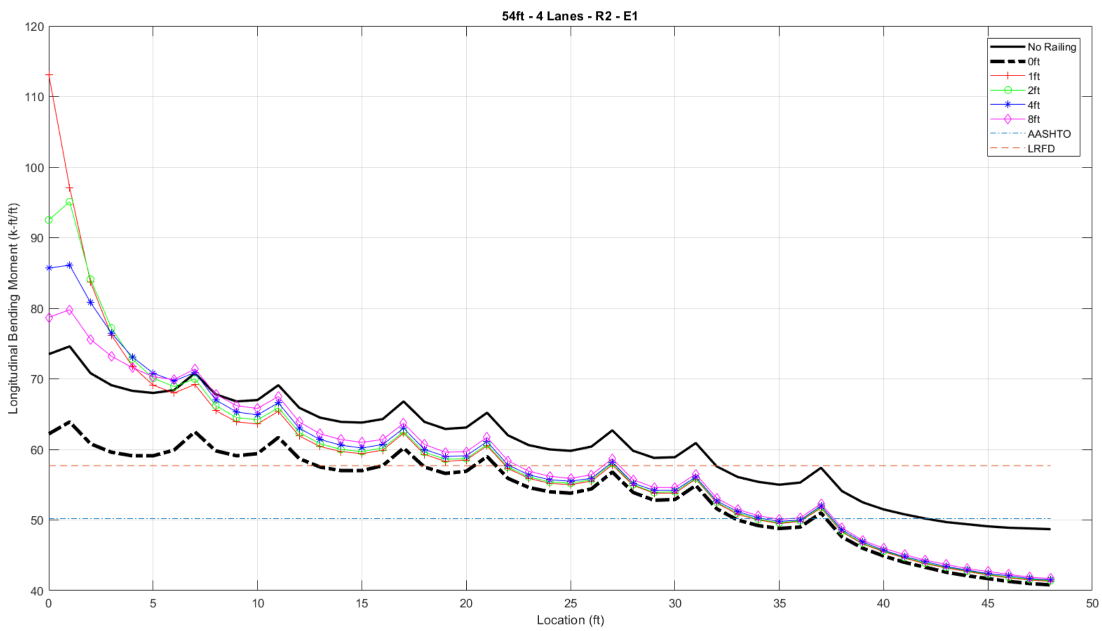

Figure 4.12: Longitudinal Moment Distribution at Critical Section for Four-Lane Single Span Bridge – Deck Span = 54ft, Deck Width = 48ft, Two Railings with Edge Loading E1

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4.3 Summary and Result Comparison with AASHTO and Reference Cases

The following section discusses and compares the FEA results with the AASHTO Standard Specs of 2002 as well as the AASHTO LRFD procedures, and with the reference cases.

4.3.1 Comparison of FEA Results with AASHTO

The following section will be divided into three subsections, categorized by the presence of one or two railings (R1 or R2) and edge loading condition (E1 or E2). Each of which will contain five tables, two of which compare the maximum longitudinal bending moments with the AASHTO Standard Specs and AASHTO LRFD, another two of which compare the maximum edge moments with the AASTHO Standard Specs and AASHTO LRFD, and the last table compares the maximum live load deflection with the AASHTO and LRFD procedures, which offer the same upper limit for live load deflection.

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4.3.1.1 FEA (R1-E1) Results with AASHTO

From table 4.13, AASHTO Specs generally tend to underestimate the longitudinal moments, and this underestimation tends to increase with increasing number of lanes and span lengths. Railing deterioration also worsens the underestimation to reach a maximum value of 42% underestimation for the 54-ft three-lane 2-ft deteriorated case.

From table 4.14, AASHTO Specs seem to overestimate edge moments in the presence of one railing due to the railing carrying some of the moment. However, 1ft of railing deterioration quickly worsens the case due to high moment concentrations which lead to critically significant underestimations reaching 65% for the 54-ft 4-lane case. Higher deterioration length further cause underestimations from AASHTO, but the underestimation value reduces due to the reduction in moment concentrations.

From table 4.15, AASHTO LRFD tends to underestimate all longitudinal moments by 6% to 33%, with two exceptional cases where overestimation, albeit slight, occurs in the event of no railing deterioration for three-lane bridges.

From table 4.16, AASHTO LRFD tends to overestimate only the edge beam moments in the event of a non-deteriorated railing. Otherwise, underestimations present themselves for all other bridge cases from a modest 4% to a dangerously high percentage of 57%.

From table 4.17, it can be seen that AASHTO Standard Specs and AASHTO LRFD (similar criterion) both overestimate the maximum live load deflections for all bridge cases. Moreover, the presence of one non-deteriorated railing greatly reduces the maximum deflection. However, an increase in railing deterioration length increases the live load deflection values to reach values nearing the case with which no railings were present.

Table 4.13: Comparison of FEA Maximum Longitudinal Moments for (R1-E1) 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 Length (ft)										
				0		1		2		4		8		
3	36	37.5	-14%	30.8	5%	39.3	-18%	40.5	-20%	40	-19%	38.1	-15%	32.4
	54	68	-26%	51.9	-3%	84	-40%	86.3	-42%	84	-40%	76.8	-35%	50.2
4	36	39.1	-17%	32.8	-1%	41.3	-22%	42.2	-23%	41.8	-22%	39.7	-18%	32.4
	54	70.8	-29%	64	-22%	77.7	-35%	78.8	-36%	78	-36%	74.7	-33%	50.2

Table 4.14: Comparison of FEA Maximum Edge Moments for (R1-E1) with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	-5%	29.4	31%	69.5	-45%	63.7	-40%	56.5	-32%	50.2	-24%	38.4
	54	71.8	-20%	48.9	18%	162.4	-65%	147.3	-61%	132.5	-57%	119.1	-52%	57.6
4	36	42.1	-9%	30.4	26%	74.9	-49%	66.5	-42%	58.3	-34%	51.3	-25%	38.4
	54	74.6	-23%	65.2	-12%	114.7	-50%	109.4	-47%	102.3	-44%	95.8	-40%	57.6

Table 4.15: Comparison of FEA Maximum Longitudinal Moments for (R1-E1) with AASHTO 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 Length (ft)										
				0		1		2		4		8		
3	36	37.5	-13%	30.8	6%	39.3	-17%	40.5	-20%	40	-19%	38.1	-14%	32.6
	54	68	-15%	51.9	11%	84	-31%	86.3	-33%	84	-31%	76.8	-25%	57.7
4	36	39.1	-21%	32.8	-6%	41.3	-25%	42.2	-27%	41.8	-26%	39.7	-22%	30.8
	54	70.8	-19%	64	-10%	77.7	-26%	78.8	-27%	78	-26%	74.7	-23%	57.7

Table 4.16: Comparison of FEA Maximum Edge Moments for (R1-E1) with AASHTO LRFD Moment

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	-6%	29.4	30%	69.5	-45%	63.7	-40%	56.5	-32%	50.2	-24%	38.2
	54	71.8	-4%	48.9	42%	162.4	-57%	147.3	-53%	132.5	-48%	119.1	-42%	69.2
4	36	42.1	-12%	30.4	21%	74.9	-51%	66.5	-45%	58.3	-37%	51.3	-28%	36.9
	54	74.6	-7%	65.2	6%	114.7	-40%	109.4	-37%	102.3	-32%	95.8	-28%	69.2

Table 4.17: Comparison of FEA Maximum Live Load Deflections with AASHTO Deflection Criterion

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												AASHTO Deflection (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	132%	0.162	233%	0.204	165%	0.208	160%	0.212	155%	0.221	144%	0.54
	54	0.5	62%	0.338	140%	0.401	102%	0.41	98%	0.422	92%	0.442	83%	0.81
4	36	0.243	122%	0.173	212%	0.213	154%	0.216	150%	0.221	144%	0.23	135%	0.54
	54	0.522	55%	0.455	78%	0.486	67%	0.489	66%	0.493	64%	0.5	62%	0.81

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4.3.1.2 FEA (R1-E2) Results with AASHTO

From table 4.18, AASHTO Specs underestimate longitudinal moments for all bridge cases reaching a maximum underestimation value of 29% for the longest span and four lanes.

From table 4.19, AASHTO Specs severely underestimate the edge moments from FEA by as high as 57%. This underestimation reduces with increasing railing deterioration length.

From table 4.20, AASHTO LRFD only matches one case of 54-ft 3-lane bridge. Otherwise, underestimations prevail from 5% to 21%.

From table 4.21, AASHTO LRFD almost slightly agrees with FEA results in the absence of railings and in the presence of only one non-deteriorated railing. Otherwise, underestimations severely prevail in the events of 1-ft and 2-ft railing deteriorations, and start to decrease with increasing railing deterioration lengths.

From table 4.22, AASHTO overestimates all maximum live load deflections for all cases. Again, this overestimation converges towards the no-railing case with increasing railing deterioration length.

Table 4.18: Comparison of FEA Maximum Longitudinal Moments for (R1-E2) 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 Length (ft)										
				0		1		2		4		8		
3	36	37.5	-14%	36.2	-10%	36.7	-12%	36.8	-12%	36.9	-12%	37.1	-13%	32.4
	54	68	-26%	60.5	-17%	63.1	-20%	63.5	-21%	64.1	-22%	65.1	-23%	50.2
4	36	39.1	-17%	38.6	-16%	38.8	-16%	38.8	-16%	38.9	-17%	39	-17%	32.4
	54	70.8	-29%	69.2	-27%	69.7	-28%	69.7	-28%	69.8	-28%	70.1	-28%	50.2

Table 4.19: Comparison of FEA Maximum Edge Moments for (R1-E2) with AASHTO Specs Moment

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												AASHTO Specs Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	-5%	39.6	-3%	43.3	-11%	40.8	-6%	40.2	-4%	40.4	-5%	38.4
	54	71.8	-20%	64.9	-11%	133	-57%	117	-51%	105.6	-45%	97.9	-41%	57.6
4	36	42.1	-9%	41.7	-8%	41.8	-8%	41.9	-8%	41.9	-8%	42	-9%	38.4
	54	74.6	-23%	73.2	-21%	78.5	-27%	76.1	-24%	73.8	-22%	74	-22%	57.6

Table 4.20: Comparison of FEA Maximum Longitudinal Moments for (R1-E2) with AASHTO 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 Length (ft)										
				0		1		2		4		8		
3	36	37.5	-13%	36.2	-10%	36.7	-11%	36.8	-11%	36.9	-12%	37.1	-12%	32.6
	54	68	-15%	60.5	-5%	63.1	-9%	63.5	-9%	64.1	-10%	65.1	-11%	57.7
4	36	39.1	-21%	38.6	-20%	38.8	-21%	38.8	-21%	38.9	-21%	39	-21%	30.8
	54	70.8	-19%	69.2	-17%	69.7	-17%	69.7	-17%	69.8	-17%	70.1	-18%	57.7

Table 4.21: Comparison of FEA Maximum Edge Moments for (R1-E2) with AASHTO LRFD Moment

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												AASHTO LRFD Moment (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	-6%	39.6	-4%	43.3	-12%	40.8	-6%	40.2	-5%	40.4	-5%	38.2
	54	71.8	-4%	64.9	7%	133	-48%	117	-41%	105.6	-34%	97.9	-29%	69.2
4	36	42.1	-12%	41.7	-12%	41.8	-12%	41.9	-12%	41.9	-12%	42	-12%	36.9
	54	74.6	-7%	73.2	-5%	78.5	-12%	76.1	-9%	73.8	-6%	74	-6%	69.2

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Table 4.22: Comparison of FEA Maximum Live Load Deflections with AASHTO Deflection Criterion

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												AASHTO Deflection (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	132%	0.226	139%	0.229	136%	0.3	80%	0.23	135%	0.232	133%	0.54
	54	0.5	62%	0.447	81%	0.464	75%	0.467	73%	0.471	72%	0.477	70%	0.81
4	36	0.243	122%	0.24	125%	0.241	124%	0.241	124%	0.242	123%	0.242	123%	0.54
	54	0.522	55%	0.511	59%	0.514	58%	0.515	57%	0.515	57%	0.517	57%	0.81

4.3.1.3 FEA (R2-E1) Results with AASHTO

From table 4.23, it can be seen that the presence of two railings reduces the longitudinal moments and improves the estimation of AASHTO for FEA results except for the 54-foot 4-lane case. Again, even 1ft of railing deterioration quickly worsens the case whereby AASHTO underestimates the FEA moments. However, as viewed previously, an increase in deterioration length quickly causes the convergence of results towards the original no-railing case, and causes a decrease in the severity of underestimation that AASHTO has for FEA results.

From table 4.24, AASHTO Specs are better at estimating edge moments than longitudinal moments, namely for the case of two railings without deterioration. Again, however, an increase in deterioration renders AASHTO Specs underestimating the moments but this underestimation is reduced with increasing deterioration lengths.

From table 4.25, AASHTO LRFD only overestimates or agrees with FEA results for the events of having two non-deteriorated railings. In other cases, underestimations, ranging from 4% to 33%, prevail, and decrease with increasing deterioration lengths.

From table 4.26, AASHTO LRFD either agrees with the FEA edge beam moments for the events of the absence of railings or the presence, thereof, of two non-deteriorated railings. Underestimations prevail for all other cases, and decrease with increasing deterioration lengths, and range from 4% to 53%.

From table 4.27, AASHTO overestimates live load deflections for all bridge cases, even as railing deterioration length increases.

Table 4.23: Comparison of FEA Maximum Longitudinal Moments for (R2-E1) with AASHTO Specs Moment

Number	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO
of		No Railing		Railing Deterioration Length (ft)										Specs Moment
Lanes				0		1		2		4		8		(Kip-ft/ft)
3	36	37.5	-14%	29.4	10%	43.2	-25%	44.5	-27%	42.2	-23%	38.7	-16%	32.4
	54	68	-26%	45.4	11%	76.2	-34%	78.1	-36%	75.9	-34%	69.1	-27%	50.2
4	36	39.1	-17%	32	1%	45.9	-29%	47.3	-32%	44.8	-28%	41	-21%	32.4
	54	70.8	-29%	62.5	-20%	76.2	-34%	77.2	-35%	76.5	-34%	73.2	-31%	50.2

Table 4.24: Comparison of FEA Maximum Edge Moments for (R2-E1) with AASHTO Specs Moment

Number	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												AASHTO
of		No Railing		Railing Deterioration Length (ft)										Specs Moment
Lanes				0		1		2		4		8		(Kip-ft/ft)
3	36	40.6	-5%	28.6	34%	67.8	-43%	62.1	-38%	55.3	-31%	47.8	-20%	38.4
	54	71.8	-20%	44.1	31%	147.3	-61%	135.2	-57%	120.9	-52%	108.1	-47%	57.6
4	36	42.1	-9%	30.1	28%	71.8	-47%	65.9	-42%	61	-37%	52.9	-27%	38.4
	54	74.6	-23%	63.9	-10%	113.1	-49%	107.6	-46%	100.7	-43%	94.1	-39%	57.6

Table 4.25: Comparison of FEA Maximum Longitudinal Moments for (R2-E1) with AASHTO LRFD Moment

Number	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												AASHTO
of		No Railing		Railing Deterioration Length (ft)										LRFD Moment
Lanes				0		1		2		4		8		(Kip-ft/ft)
3	36	37.5	-13%	29.4	11%	43.2	-25%	44.5	-27%	42.2	-23%	38.7	-16%	32.6
	54	68	-15%	45.4	27%	76.2	-24%	78.1	-26%	75.9	-24%	69.1	-16%	57.7
4	36	39.1	-21%	32	-4%	45.9	-33%	47.3	-35%	44.8	-31%	41	-25%	30.8
	54	70.8	-19%	62.5	-8%	76.2	-24%	77.2	-25%	76.5	-25%	73.2	-21%	57.7

Table 4.26: Comparison of FEA Maximum Edge Moments for (R2-E1) with AASHTO LRFD Moment

Number	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												AASHTO
of		No Railing		Railing Deterioration Length (ft)										LRFD Moment
Lanes				0		1		2		4		8		(Kip-ft/ft)
3	36	40.6	-6%	28.6	34%	67.8	-44%	62.1	-38%	55.3	-31%	47.8	-20%	38.2
	54	71.8	-4%	44.1	57%	147.3	-53%	135.2	-49%	120.9	-43%	108.1	-36%	69.2
4	36	42.1	-12%	30.1	23%	71.8	-49%	65.9	-44%	61	-40%	52.9	-30%	36.9
	54	74.6	-7%	63.9	8%	113.1	-39%	107.6	-36%	100.7	-31%	94.1	-26%	69.2

Table 4.27: Comparison of FEA Maximum Live Load Deflections with AASHTO Deflection Criterion

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												AASHTO Deflection (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	132%	0.157	244%	0.198	173%	0.202	167%	0.206	162%	0.214	152%	0.54
	54	0.5	62%	0.297	173%	0.359	126%	0.366	121%	0.376	115%	0.394	106%	0.81
4	36	0.243	122%	0.168	221%	0.211	156%	0.214	152%	0.219	147%	0.228	137%	0.54
	54	0.522	55%	0.445	82%	0.476	70%	0.479	69%	0.482	68%	0.489	66%	0.81

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4.3.2 Comparison of FEA Results with the No-Railing (NR) Reference Case

In this subsection, the same values which were obtained from the FEA cases are going to be compared with the no-railing (NR) case as the reference this time. Here, there are three tables for each loading condition; one which compares longitudinal bending moments to the NR case, another which compares edge beam moments to the NR case, and the final one which compares live load deflections with NR case. The results here are tabulated in the form of ratios, with the ratio being that of the case at-hand to the NR reference case.

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4.3.2.1 FEA (R1-E1) Results with NR-Case

From table 4.28, the presence of just one railing for E1 loading reduces the longitudinal bending moments in the slab by a range of 10-24%. Sudden deterioration of the railing renders the NR-case better as the longitudinal moments greatly increase by 5-24%. Greater deterioration lengths offer more convergence, as seen before, towards the NR-case, which renders the non-deteriorated portion of the railing the same as its absence thereof.

From table 4.29, the presence of only one railing with E1 loading reduces edge beam moments by 13-32%. However, deterioration of the railing quickly spikes up the moment values, rendering the NR-case much better.

From table 4.30, the presence of one railing, whether deteriorated or not, minimizes the maximum live load deflection in the slab. However, larger deterioration lengths yield results which converge towards the NR-case.

Table 4.28: Comparison of FEA Maximum Longitudinal Moments for (R1-E1) with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MNR (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	37.5	1.00	30.8	0.82	39.3	1.05	40.5	1.08	40.0	1.07	38.1	1.02	37.5
	54	68.0	1.00	51.9	0.76	84.0	1.24	86.3	1.27	84.0	1.24	76.8	1.13	68
4	36	39.1	1.00	32.8	0.84	41.3	1.06	42.2	1.08	41.8	1.07	39.7	1.02	39.1
	54	70.8	1.00	64.0	0.90	77.7	1.10	78.8	1.11	78.0	1.10	74.7	1.06	70.8

Table 4.29: Comparison of FEA Maximum Edge Moments for (R1-E1) with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												Edge Beam Moment MNR (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	1.00	29.4	0.72	69.5	1.71	63.7	1.57	56.5	1.39	50.2	1.24	40.6
	54	71.8	1.00	48.9	0.68	162.4	2.26	147.3	2.05	132.5	1.85	119.1	1.66	71.8
4	36	42.1	1.00	30.4	0.72	74.9	1.78	66.5	1.58	58.3	1.38	51.3	1.22	42.1
	54	74.6	1.00	65.2	0.87	114.7	1.54	109.4	1.47	102.3	1.37	95.8	1.28	74.6

Table 4.30: Comparison of FEA Maximum Live Load Deflections with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												Maximum Deflection ΔNR (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	1.00	0.162	0.70	0.204	0.88	0.208	0.89	0.212	0.91	0.221	0.95	0.233
	54	0.500	1.00	0.338	0.68	0.401	0.80	0.410	0.82	0.422	0.84	0.442	0.88	0.5
4	36	0.243	1.00	0.173	0.71	0.213	0.88	0.216	0.89	0.221	0.91	0.230	0.95	0.243
	54	0.522	1.00	0.455	0.87	0.486	0.93	0.489	0.94	0.493	0.94	0.500	0.96	0.522

4.3.2.2 FEA (R1-E2) Results with NR-Case

From table 4.31, the presence of one railing with edge loading E2 yields no considerable advantage over the NR-case, with reductions in longitudinal moments being a modest 1-7% with one exception in 3-lane 54-ft span length case, whereby a reduction of 11% was witnessed for no railing deterioration.

From table 4.32, also no significant reductions are made in edge moments. In fact, it is noticed that for the 54-ft 3-lane case, railing deterioration increases the beam moment by 36-85% more than the base NR-case, rendering the bridge much more economical to design if the NR-case were considered.

From table 4.33, it is also evident that insignificant reductions in live load deflections are obtained in the presence of one railing with E2 loading, with the highest reduction being 11% for 54-ft 3-lane case.

Table 4.31: Comparison of FEA Maximum Longitudinal Moments for (R1-E2) with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MNR (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	37.5	1.00	36.2	0.97	36.7	0.98	36.8	0.98	36.9	0.98	37.1	0.99	37.5
	54	68.0	1.00	60.5	0.89	63.1	0.93	63.5	0.93	64.1	0.94	65.1	0.96	68
4	36	39.1	1.00	38.6	0.99	38.8	0.99	38.8	0.99	38.9	0.99	39	1.00	39.1
	54	70.8	1.00	69.2	0.98	69.7	0.98	69.7	0.98	69.8	0.99	70.1	0.99	70.8

Table 4.32: Comparison of FEA Maximum Edge Moments for (R1-E2) with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												Edge Beam Moment MNR (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	1.00	39.6	0.98	43.3	1.07	40.8	1.00	40.2	0.99	40.4	1.00	40.6
	54	71.8	1.00	64.9	0.90	133	1.85	117	1.63	105.6	1.47	97.9	1.36	71.8
4	36	42.1	1.00	41.7	0.99	41.8	0.99	41.9	1.00	41.9	1.00	42.0	1.00	42.1
	54	74.6	1.00	73.2	0.98	78.5	1.05	76.1	1.02	73.8	0.99	74.0	0.99	74.6

Table 4.33: Comparison of FEA Maximum Live Load Deflections with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												Maximum Deflection ΔNR (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	1.00	0.226	0.97	0.229	0.98	0.300	1.29	0.230	0.99	0.232	1.00	0.233
	54	0.500	1.00	0.447	0.89	0.464	0.93	0.467	0.93	0.471	0.94	0.477	0.95	0.5
4	36	0.243	1.00	0.240	0.99	0.241	0.99	0.241	0.99	0.242	1.00	0.242	1.00	0.243
	54	0.522	1.00	0.511	0.98	0.514	0.98	0.515	0.99	0.515	0.99	0.517	0.99	0.522

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4.3.2.3 FEA (R2-E1) Results with NR-Case

From table 4.34, the advantage of two railings over no railings is evident; reductions in longitudinal bending moments for the non-deteriorated cases range between 12-33%. However, minor deteriorations quickly negate the advantage of the railing by allowing the longitudinal moments to increase by 8-17% more than the NR-case. Again, an increase in the deterioration length, however, quickly reduces the maximum longitudinal moments, and the values then converge towards the NR-case.

From table 4.35, edge moments are greatly reduced by 14-39% in the presence of two railings. However, notice the sudden increase in edge moments when the railing is deteriorated; the most pronounced case is easily decipherable; the 54-ft 3-lane case with 1ft deterioration yields two-times the maximum edge moment, as compared with the NR-case. Contrary to previous expectations and observations, increases in railing deterioration lengths do not result in convergence towards the NR-case, with edge moments still 18-51% higher than the base NR-case, even for 8ft of railing deterioration.

From table 4.36, regardless of deteriorations, the presence of two railings quickly reduces the maximum live load deflection as compared with the NR-case.

Table 4.34: Comparison of FEA Maximum Longitudinal Moments for (R2-E1) with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MNR (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	37.50	1.00	29.40	0.78	43.20	1.15	44.50	1.19	42.20	1.13	38.70	1.03	37.5
	54	68.00	1.00	45.40	0.67	76.20	1.12	78.10	1.15	75.90	1.12	69.10	1.02	68
4	36	39.10	1.00	32.00	0.82	45.90	1.17	47.30	1.21	44.80	1.15	41.00	1.05	39.1
	54	70.80	1.00	62.50	0.88	76.20	1.08	77.20	1.09	76.50	1.08	73.20	1.03	70.8

Table 4.35: Comparison of FEA Maximum Edge Moments for (R2-E1) with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												Edge Beam Moment MNR (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.60	1.00	28.60	0.70	67.80	1.67	62.10	1.53	55.30	1.36	47.80	1.18	40.6
	54	71.80	1.00	44.10	0.61	147.30	2.05	135.20	1.88	120.90	1.68	108.10	1.51	71.8
4	36	42.10	1.00	30.10	0.71	71.80	1.71	65.90	1.57	61.00	1.45	52.90	1.26	42.1
	54	74.60	1.00	63.90	0.86	113.10	1.52	107.60	1.44	100.70	1.35	94.10	1.26	74.6

Table 4.36: Comparison of FEA Maximum Live Load Deflections with NR-Case

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												Maximum Deflection ΔNR (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	1.00	0.157	0.67	0.198	0.85	0.202	0.87	0.206	0.88	0.214	0.92	0.233
	54	0.500	1.00	0.297	0.59	0.359	0.72	0.366	0.73	0.376	0.75	0.394	0.79	0.5
4	36	0.243	1.00	0.168	0.69	0.211	0.87	0.214	0.88	0.219	0.90	0.228	0.94	0.243
	54	0.522	1.00	0.445	0.85	0.476	0.91	0.479	0.92	0.482	0.92	0.489	0.94	0.522

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4.3.3 Comparison of FEA with the Two-Railing Non-Deteriorated (R2-0) Case

In this subsection, the same FEA results are compared with the non-deteriorated two-railing case. The basis of this comparison is whether it is better to include a second railing if one railing were present, or if it is better not include any railings at all if no railings were originally present. Again, results are tabulated and present ratios as in the previous subsection.

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4.3.3.1 FEA (R1-E1) Results with R2-0 Case

From table 4.37, it can be seen that the presence of two non-deteriorated railings is better in all the R1-E1 bridge cases in the event of maximum longitudinal moments. Although the presence of one non-deteriorated railing under E1 closely mimics the presence of two railings, the problem lies in the event of the deterioration of this railing, whereby the longitudinal moments become 20-90% higher than those which would have been obtained in the case of two railings.

From table 4.38, edge beam moments are significantly larger in the presence of one railing as compared to two railings, reaching a maximum value of 3.7, implying a 3.7-times increase in the maximum edge moment for one railing as compared with two railings.

From table 4.39, the presence of one non-deteriorated railing mimics the presence of two railings in terms of deflections. However, increases in deterioration lengths yield higher deflections which range from 9-68% higher than the base case with two non-deteriorated railings.

Table 4.37: Comparison of FEA Maximum Longitudinal Moments for (R1-E1) with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	37.5	1.28	30.8	1.05	39.3	1.34	40.5	1.38	40.0	1.36	38.1	1.30	29.4
	54	68.0	1.50	51.9	1.14	84.0	1.85	86.3	1.90	84.0	1.85	76.8	1.69	45.4
4	36	39.1	1.22	32.8	1.03	41.3	1.29	42.2	1.32	41.8	1.31	39.7	1.24	32
	54	70.8	1.13	64.0	1.02	77.7	1.24	78.8	1.26	78.0	1.25	74.7	1.20	62.5

Table 4.38: Comparison of FEA Maximum Edge Moments for (R1-E1) with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												Edge Beam Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	1.42	29.4	1.03	69.5	2.43	63.7	2.23	56.5	1.98	50.2	1.76	28.6
	54	71.8	1.63	48.9	1.11	162.4	3.68	147.3	3.34	132.5	3.00	119.1	2.70	44.1
4	36	42.1	1.40	30.4	1.01	74.9	2.49	66.5	2.21	58.3	1.94	51.3	1.70	30.1
	54	74.6	1.17	65.2	1.02	114.7	1.79	109.4	1.71	102.3	1.60	95.8	1.50	63.9

Table 4.39: Comparison of FEA Maximum Live Load Deflections with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												Maximum Deflection ΔR0 (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	1.48	0.162	1.03	0.204	1.30	0.208	1.32	0.212	1.35	0.221	1.41	0.157
	54	0.500	1.68	0.338	1.14	0.401	1.35	0.410	1.38	0.422	1.42	0.442	1.49	0.297
4	36	0.243	1.45	0.173	1.03	0.213	1.27	0.216	1.29	0.221	1.32	0.230	1.37	0.168
	54	0.522	1.17	0.455	1.02	0.486	1.09	0.489	1.10	0.493	1.11	0.500	1.12	0.445

4.3.3.2 FEA (R1-E2) Results with R2-0 Case

From table 4.40, the presence of one railing with E2 loading allows for a minimum increase in maximum longitudinal moments by 11% and a maximum increase by 43%, thus proving the importance of including two railings in reducing the longitudinal bending moments in all R1-E2 bridge cases.

From table 4.41, having one railing with E2 loading greatly increases edge beam moments by a minimum of 15% for 54-ft 4-lane bridges and a maximum of 202% (thus 3.02-times the R2-0 value) for the 54-ft 3-lane 1-ft deteriorated bridge case.

From table 4.42, maximum live load deflections under R1-E2 cases are at least 15% more than the R2-0 case, and in some cases, the deflections reach values which are 91% larger than those which would have been obtained in the presence of two non-deteriorated railings.

Table 4.40: Comparison of FEA Maximum Longitudinal Moments for (R1-E2) with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	37.5	1.28	36.2	1.23	36.7	1.25	36.8	1.25	36.9	1.26	37.1	1.26	29.4
	54	68.0	1.50	60.5	1.33	63.1	1.39	63.5	1.40	64.1	1.41	65.1	1.43	45.4
4	36	39.1	1.22	38.6	1.21	38.8	1.21	38.8	1.21	38.9	1.22	39	1.22	32
	54	70.8	1.13	69.2	1.11	69.7	1.12	69.7	1.12	69.8	1.12	70.1	1.12	62.5

Table 4.41: Comparison of FEA Maximum Edge Moments for (R1-E2) with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												Edge Beam Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.6	1.42	39.6	1.38	43.3	1.51	40.8	1.43	40.2	1.41	40.4	1.41	28.6
	54	71.8	1.63	64.9	1.47	133	3.02	117	2.65	105.6	2.39	97.9	2.22	44.1
4	36	42.1	1.40	41.7	1.39	41.8	1.39	41.9	1.39	41.9	1.39	42.0	1.40	30.1
	54	74.6	1.17	73.2	1.15	78.5	1.23	76.1	1.19	73.8	1.15	74.0	1.16	63.9

Table 4.42: Comparison of FEA Maximum Live Load Deflections with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												Maximum Deflection ΔR0 (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	1.48	0.226	1.44	0.229	1.46	0.300	1.91	0.230	1.46	0.232	1.48	0.157
	54	0.500	1.68	0.447	1.51	0.464	1.56	0.467	1.57	0.471	1.59	0.477	1.61	0.297
4	36	0.243	1.45	0.240	1.43	0.241	1.43	0.241	1.43	0.242	1.44	0.242	1.44	0.168
	54	0.522	1.17	0.511	1.15	0.514	1.16	0.515	1.16	0.515	1.16	0.517	1.16	0.445

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4.3.3.3 FEA (R2-E1) Results with R2-0 Case

From table 4.43, having two non-deteriorated railings is an advantage over the no-railing case as they reduce the longitudinal bending moments. However, with increasing railing deterioration length, and even at a relatively high deterioration length of 8ft, although the two-railing case is still better, it is shown that the no-railing case is far better than all other cases when compared with the non-deteriorated two-railing case.

From table 4.44, having just one deteriorated railing out of the two results in exorbitantly high edge beam moments ranging from a minimum of being 17% higher to a maximum of being 234% higher than those obtained from a non-deteriorated two-railing bridge case.

From table 4.45, it can be seen that the presence of two non-deteriorated railings is clearly better in reducing the maximum live load deflections. However, it is also noticeable that the effect of deterioration and the effect of reduction in maximum load deflections is minimal for large 54-ft spans having 4 lanes, whereby the maximum live load deflections are nearly all the same. For smaller spans, one deteriorated railing causes an increase of around 25-35% in the live load deflections, as compared with the base non-deteriorated two-railing case.

Table 4.43: Comparison of FEA Maximum Longitudinal Moments for (R2-E1) with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Longitudinal Moment (Kip-ft/ft)												Maximum Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	37.50	1.28	29.40	1.00	43.20	1.47	44.50	1.51	42.20	1.44	38.70	1.32	29.4
	54	68.00	1.50	45.40	1.00	76.20	1.68	78.10	1.72	75.90	1.67	69.10	1.52	45.4
4	36	39.10	1.22	32.00	1.00	45.90	1.43	47.30	1.48	44.80	1.40	41.00	1.28	32
	54	70.80	1.13	62.50	1.00	76.20	1.22	77.20	1.24	76.50	1.22	73.20	1.17	62.5

Table 4.44: Comparison of FEA Maximum Edge Moments for (R2-E1) with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Edge Moment (Kip-ft/ft)												Edge Beam Moment MR0 (Kip-ft/ft)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	40.60	1.42	28.60	1.00	67.80	2.37	62.10	2.17	55.30	1.93	47.80	1.67	28.6
	54	71.80	1.63	44.10	1.00	147.30	3.34	135.20	3.07	120.90	2.74	108.10	2.45	44.1
4	36	42.10	1.40	30.10	1.00	71.80	2.39	65.90	2.19	61.00	2.03	52.90	1.76	30.1
	54	74.60	1.17	63.90	1.00	113.10	1.77	107.60	1.68	100.70	1.58	94.10	1.47	63.9

Table 4.45: Comparison of FEA Maximum Live Load Deflections with R2-0 Case

Number of Lanes	Span Length (ft)	FEA Maximum Slab Deflection (in)												Maximum Deflection ΔR0 (in)
		No Railing		Railing Deterioration Length (ft)										
				0		1		2		4		8		
3	36	0.233	1.48	0.157	1.00	0.198	1.26	0.202	1.29	0.206	1.31	0.214	1.36	0.157
	54	0.500	1.68	0.297	1.00	0.359	1.21	0.366	1.23	0.376	1.27	0.394	1.33	0.297
4	36	0.243	1.45	0.168	1.00	0.211	1.26	0.214	1.27	0.219	1.30	0.228	1.36	0.168
	54	0.522	1.17	0.445	1.00	0.476	1.07	0.479	1.08	0.482	1.08	0.489	1.10	0.445

4.4 Summary of Results

Table 4.46 below presents a comprehensive summary of the ratios of the moments for all loading conditions as compared to the reference cases and AASHTO Standard Specs and AASHTO LRFD procedures.

Table 4.46: Summary of Ratio Ranges for FEA Results to Reference Cases and Codes

FEA Loading Condition	Moment Ratios for One and Two Railings							
	FEA / FEA-NR		FEA / FEA R2-0		FEA / AASHTO Specs		FEA / AASHTO LRFD	
	Slab	Edge Beam	Slab	Edge Beam	Slab	Edge Beam	Slab	Edge Beam
R1-E1	0.76 to 1.27	0.68 to 2.26	1.02 to 1.90	1.01 to 3.68	0.95 to 1.72	1.06 to 2.82	0.90 to 1.50	0.71 to 2.35
R1-E2	0.89 to 1.00	0.90 to 1.85	1.11 to 1.43	1.15 to 3.02	1.12 to 1.39	1.03 to 2.30	1.05 to 1.27	0.94 to 1.92
R2-E1	0.67 to 1.21	0.61 to 2.05	1.13 to 1.72	1.17 to 3.34	0.99 to 1.56	1.06 to 2.56	0.79 to 1.54	0.64 to 2.13

4.4.1 FEA Results with NR Reference Case

- For the R1-E1 case, the presence of one railing contributes somewhat to the reduction in slab moments as compared with the NR case. However, as mentioned before, the presence of small deterioration lengths of 1ft or 2ft quickly increase the longitudinal slab moments. The edge beam moments, on the other hand, vary significantly, especially in the presence of railing deterioration, whereby, even with the presence of one railing, edge moments reach more than double the values of the base NR case.
- For the R1-E2 case, insignificant reductions are seen in longitudinal moments. However, edge moments experience significant increases in the cases of deterioration of railings, with values reaching almost twice the base case value.

- For the R2-E1 case, it is evident that the presence of two railings far outweighs its disadvantages, as the moments, both slab and edge, reach high values in the absence of both railings.

4.4.2 FEA Results with R2-0 Reference Case

- For the R1-E1 case, with small deterioration lengths, and this is especially evident in longer spans, the slab moments and edge moments far exceed the base case with two non-deteriorated railings, thus proving the importance of having another railing to counteract some of the damage dealt to the bridge.
- For R1-E2 case, the slab moment increase is less severe than its predecessor, but for large spans and small deterioration lengths, significant increases in slab moments reaching almost 40% are still seen. For edge moments, increases are exorbitantly high, reaching three-times the base case value, which further reinforces the notion of having two railings.
- For the R2-E1 case, minor deteriorations cause significant increases in both slab and edge moments, with the latter case being more severe due to heavy stress concentrations at the deterioration location.

4.4.3 FEA Results with AASHTO Standard Specifications (2002)

- It should be noted that the FEA moments can be reduced as per AASHTO procedures to consider the improbability of having three and four lanes loaded simultaneously

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- For the R1-E1 case, AASHTO either exactly estimate or greatly underestimate the slab moments. For the edge moments, the case is more severe, as AASHTO completely misses on estimating any of the cases' edge moments, whereby, in some cases with small deterioration lengths, the FEA-obtained maximum edge moments exceed AASHTO Specs by more than 180%.
- For the R1-E2 case, AASHTO underestimation of slab moments is less pronounced than the previous case, but still significant in some cases. However, edge beam moment estimation is erroneous, especially with small deterioration lengths, with FEA-obtained results exceeding AASHTO Specs by around 130%.
- For the R2-E1 case, there is also severe underestimation from AASHTO Specs for both longitudinal moments and edge beam moments.

4.4.4 FEA Results with AASHTO LRFD (2012)

- It should be noted that the FEA moments can be reduced as per AASHTO procedures to consider the improbability of having three and four lanes loaded simultaneously
- For the R1-E1 case, AASHTO LRFD approximately estimates FEA slab moments, especially in the presence of the non-deteriorated railing, and tends to underestimate FEA results in the presence of deteriorations by almost 50%. For edge beam moments, AASHTO LRFD overestimates the cases for which one non-deteriorated railing is present by a maximum of 29%, and for small deterioration lengths especially, tends to severely underestimate FEA moments by a maximum of 150%.
- For the R1-E2 case, AASHTO LRFD underestimates them by a maximum of 27%. For the edge moments, AASHTO LRFD slightly overestimates them in the presence of one

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non-deteriorated railing by a maximum of 6%, while it severely underestimates them at other cases of small deterioration lengths by values reaching a maximum of 92%.

- For the R2-E1 case, AASHTO LRFD either overestimates slab moments for two non-deteriorated railings by a maximum of 21%, or underestimates remaining cases by values reaching 54%. Edge beam moments are greatly overestimated by a maximum of 36% for non-deteriorated railings, but are also greatly underestimated by a maximum factor exceeding 100%, especially in the event of 1-ft deteriorated railing.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

In this text, effects of the full deterioration of railings on the wheel load distribution of trucks, the slab moments, the edge moments, and live load deflections in one-span reinforced concrete slab bridges were investigated. Bridges were assumed to be simply-supported at the piers, and were subjected to AASHTO HS-20 design truck loading, with the truck loading varied between E1 and E2 loading conditions. The finite element method was utilized in the analysis, with the commercial software package SAP2000 being the chosen tool for analysis. The bridge deck and railing were both modelled as shell elements, and the trucks were positioned in such a manner to cause the maximum bending moments in the slab.

Comparisons were performed with AASHTO Standard Specifications (2002) and AASHTO LRFD procedures (2012), as well as two main reference cases, the first of which being the no-railing (NR) base case, and the other being the two-railing non-deteriorated (R2-0) base case.

The railing was taken as 8in x 30in and was deteriorated by lengths varying from 0ft to 8ft and the slab moments as well as the edge beam moments at the critical slab section were extracted, tabulated, and compared with the provisions and reference cases. The same was done for the maximum live load deflections, too.

The slab was divided into square shell elements of size 1ft x 1ft, while the railing was divided into rectangular shell elements of 1ft x 1.25ft, with the shell elements retaining the properties of the original structure.

The results were divided based on the loading conditions to which the bridge was exposed to, namely one railing or two railing (R1 or R2), as well as edge loading conditions E1 or E2, with E1 being such that the truck is positioned at 1ft away from the railing, and E2 being such that the railing is on the other side of the truck, with the truck being 1ft away from the edge which is free from any railing.

5.2 Conclusions and Recommendations

Several repetitive patterns were observed during the research, and could be summarized in the form of bullet points below:

- The FEA slab moments were less affected by railing deterioration as compared with edge beam moments, still, however, these moments, for small deteriorations, will be closer to the NR case.
- This increase in slab moments may be absorbed by the safety factor on a temporary basis. However, it is advisable, beyond breakage, that the traffic be limited either by preventing heavy-weight vehicles, such as trucks, to cross the bridge, or by preventing the traffic passage on the lane next to the breakage until the repair of the railing has been completed.
- AASHTO Standard Specs greatly underestimate both slab moments as well as edge beam moments.

- AASHTO LRFD either agrees with or underestimates FEA slab moments for all bridge cases.
- AASHTO LRFD greatly underestimates most of the edge beam moments for all bridge cases, especially when either no railings are present, or when deteriorated railings are present.
- The presence of two railings is greatly advantageous, as it offers reduced slab moments as well as edge beam moments.
- It should be noted that small deteriorations render the edge beam moments greatly higher than in the absence of railings, so edge beams need to be properly reinforced.

The major recommendation at the end of this research is to properly reinforce the edge beam moments in the event that any railing deterioration occurs. Furthermore, it is obligatory to quickly remedy any deterioration that might occur, either small or large, since, as mentioned previously, small deterioration lengths cause high moment concentrations for which the edge beam moments might be prepared, while the larger deterioration lengths help in dissipating the moment concentrations, but the results end up converging towards the no-railing case, meaning that, for large deteriorations, the bridge acts as if the railing is not present at all.

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