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Development of a Strategy to Address Load Posted Bridges through Reduction in Uncertainty in Load Ratings—Volume 3: Refined Load Rating Recommendations and Examples

Technical Report 0-6955-R1-Vol3

Cooperative Research Program

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16. Abstract Maintaining the functionality of the transportation infrastructure depends on the successful management of aging bridge assets. Departments of Transportation rely on the load rating process to evaluate the sufficiency of the bridge structures in their state, and post load restrictions if the capacity of a bridge does not meet the maximum load effect based on the current legal loads. Removing load postings is always of interest because there can be commerce, traffic, and emergency egress issues due to rerouting of vehicles. Posted structures come in various shapes and sizes, built in different eras and environments, and exhibit vastly different structural behaviors. As such, there is no clear-cut single solution for addressing the possibility of removing postings. In this study, strategies have been developed to reduce uncertainty in load rating procedure in a safe and appropriate manner to potentially increase or remove the load postings of typical steel and concrete bridges. The Volume 1 Report documents findings including a review of the state-of-the-practice and state-of-the-art for load rating existing bridges, a review and synthesis of the bridge characteristics of load posted bridges in Texas, and the basic load rating analysis conducted for selected representative bridges to identify the controlling limit states. The Volume 2 Report provides details of further study for typical bridges including refined analysis for more accurate prediction of load distribution, load testing, model updating and calibration, and refined load rating analysis. This Volume 3 Report provides specific recommendations for refined load rating analysis for the four selected bridges types, and provides detailed refined load rating examples for each bridge type. The effect of each refinement on the revised load ratings is evaluated, and the implications for potentially increasing the posted loads or removal of load posting are discussed.		
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DEVELOPMENT OF A STRATEGY TO ADDRESS LOAD POSTED BRIDGES THROUGH REDUCTION IN UNCERTAINTY IN LOAD RATINGS —VOLUME 3: REFINED LOAD RATING RECOMMENDATIONS AND EXAMPLES

by

Mary Beth D. Hueste, Ph.D., P.E.
Research Engineer
Texas A&M Transportation Institute

Stefan Hurlebaus, Ph.D.
Research Scientist
Texas A&M Transportation Institute

John B. Mander, Ph.D.
Research Engineer
Texas A&M Transportation Institute

Stephanie Paal, Ph.D.
Assistant Research Scientist
Texas A&M Transportation Institute

Tevfik Terzioglu, Ph.D.
Postdoctoral Research Associate
Texas A&M Transportation Institute

Matthew Stieglitz
Graduate Assistant Researcher
Texas A&M Transportation Institute

Nuzhat Kabir
Graduate Assistant Researcher
Texas A&M Transportation Institute

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College Station, Texas 77843-3135

DISCLAIMER

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1 INTRODUCTION

1.1 BACKGROUND AND SIGNIFICANCE

There is an ever-increasing demand on highways for improved mobility and connectivity for delivering more goods and services, which increases the importance of reliable, well-maintained transportation infrastructure. Maintaining the functionality and health of the transportation infrastructure depends on the successful management of aging bridge assets. Transportation agencies use the load rating process to evaluate the condition and adequacy of the existing bridge infrastructure. Bridges that do not have sufficient capacity to carry the current legal loads are posted for more restrictive load limits based on the procedure provided in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). According to the National Bridge Inventory (NBI 2016) database, the state of Texas has 2111 bridges that are posted at load levels below the legal limit. About half of the load posted bridges are not structurally deficient but require load posting using the basic load rating assumptions that are primarily based on the AASHTO MBE. The basic load rating methods can include conservative assumptions for calculating the capacity and load effects on the bridge. Because load posted bridges can cause major restrictions on freight movement, economic vitality, traffic congestion, and emergency egress, removing postings of load posted bridges in a safe and appropriate manner is always of interest for both TxDOT and State of Texas. However, load posted bridges vary greatly in terms of geometry, size, construction style, age, and environmental conditions; and their in situ structural behavior can also differ significantly as compared to simplified models. As such, there is no clear-cut single solution for addressing the possibility of removing postings.

1.2 OBJECTIVES AND SCOPE

The overall objective of this project is to determine appropriate strategies for bridge load rating through a reduction in uncertainty that can potentially increase or remove the load postings of typical bridges in Texas. Some of the uncertainty and inherent conservatism in the current basic load rating procedures can potentially be minimized by using more accurate material properties, refined modeling, and load testing to understand the in situ structural behavior. The proposed approach to addressing posted bridges begins with developing a strategy to reduce uncertainty in a safe and appropriate manner, based on the specific details of a bridge and refinements in the load rating process. The AASHTO MBE allows for refined load rating, but does not address the challenge of identifying the appropriate structures. Therefore, this research project quantifies and characterizes the population of load posted bridges in Texas and reviews areas of opportunity, including more accurate material properties and information from bridge inspections, refined modeling to assess possible reductions in live load distribution, and load testing for verification of structural response. The load rating calculations using refined information and techniques presented in this research are expected to provide better accuracy in load rating and can potentially eliminate load postings or increase the allowable loads on load posted bridges.

1.3 RESEARCH PLAN

The outcome of this research study supports TxDOT's implementation of refined load rating approaches to potentially remove or increase the posted load limits in the Texas bridge inventory. The following tasks were conducted to accomplish the research objectives.

- Task 1. Project Management and Research Coordination
- Task 2. Review State-of-the-Art, State-of-the-Practice, and Load-Posted Bridge Inventory
- Task 3. Conduct Basic Load Ratings and Identify Areas of Opportunity
- Task 4. Refined Analysis for more Accurate Prediction of Live Load Distribution
- Task 5. Load Testing, Model Updating and Calibration, and Refined Load Ratings
- Task 6. Develop Refined Load Rating Guidelines and Examples

The Volume 1 Report (Hueste et al. 2019a) documents the findings of Tasks 2 and 3, which includes a summary of the state-of-the-practice and state-of-the-art for load rating of existing bridges, a review and synthesis of the characteristics of load posted bridges in Texas, and the basic load rating analysis for selected representative bridges to identify the controlling limit states.

The Volume 2 Report (Hueste et al. 2019b) documents the findings of Tasks 4 and 5 including refined analysis for more accurate LLDF prediction, load testing, model updating and calibration, and refined load rating analysis. Refined analysis includes three-dimensional linear finite element modeling, which can provide more accurate estimation of load distribution and live load distribution factors (LLDFs). The load testing of the selected bridges, along with model updating and calibration based on the field measurements, are used to determine refined load ratings to compare with the basic load ratings. The results are reviewed with respect to the potential implications and opportunities for load rating these bridges and similar bridge structures.

This Volume 3 Report documents the details of the developed refined load rating guidelines for the four selected bridges types, and provides detailed refined load rating examples for each bridge type. The effect of each refinement on the revised load ratings has been evaluated, and implications for potentially increasing the posted loads or removal of load posting have been discussed.

1.4 REPORT OUTLINE

This Volume 3 Report consists of nine main chapters that document the findings of Task 6.

- Chapter 1 presents the background and significance, research objectives and scope of the project, research plan including specific tasks, and outlines the Volume 3 research report.
- Chapter 2 provides the refined load rating recommendations and commentary for steel multi-girder bridges.
- Chapter 3 presents the refined load rating recommendations and commentary for simple span concrete multi-girder bridges.
- Chapter 4 summarizes the refined load rating recommendations and commentary for simple span integral curb concrete slab bridges.
- Chapter 5 provides several load rating examples for a typical simple-span steel multi-girder bridge, which was modeled and load tested as part of this project (Bridge SM-5). The examples show the basic and refined load rating procedures, and discusses the improvements for each refined load rating recommendation. The initial basic load rating is performed under

the assumption that the bridge is acting without composite girder behavior and that the girder ends are both simply-supported. This initial load rating is used for comparison when conducting refined load ratings assuming partial composite action, partial end restraint, or a combination of the two conditions.

- Chapter 6 presents several load rating examples for a typical continuous steel multi-girder bridge, which was modeled and load tested in this project (Bridge SC-12). The examples show the basic and refined load rating procedures and discusses the improvements for each refined load rating recommendation. The initial basic load rating calculations are performed using the assumption that the bridge is acting with no composite girder behavior, and that the girders are continuous over piers and that the girder ends at the abutments are simply-supported. The initial basic load ratings are used for comparison when conducting refined load ratings assuming partial composite action.
- Chapter 7 provides several load rating examples for a typical simple span concrete multi-girder bridge, which was modeled and load tested in this project (Bridge Bridge CM-5). The examples show the basic load rating that assumes simply supported boundary conditions and the material properties specified in the design drawings. The basic load rating is compared to refined load ratings assuming partial end restraint, updated material strengths, and a combination of these two modifications.
- Chapter 8 presents several load rating examples for a typical simple span integral curb concrete slab bridge (Bridge CS-9). The examples include the basic load rating assuming simply supported boundary conditions and the material properties as specified in the design drawings. The basic load rating is compared to refined load ratings assuming partial end restraint, updated material strengths, or a combination of these two parameters.
- Chapter 9 provides a summary of the findings in this Volume 3 Report.

2 RECOMMENDED PROCEDURES FOR REFINED LOAD RATING OF STEEL MULTI-GIRDER BRIDGES

Multiple recommendations to refine and improve load ratings for steel multi-girder bridges are proposed based on the results of this research project. These recommendations are supported by the project tasks, including review of the literature, identification of areas of refinement based on the basic load rating analysis, examination of bridge behavior through finite element method (FEM) modeling and analysis, observations made during load testing of two selected steel multi-girder bridges, and calibration and analysis of the associated FEM models.

The first section describes recommended procedures to be conducted during bi-annual inspections of the bridges. Subsequent sections are presented in the order expected to be most efficient, by emphasizing approaches having potential to increase load ratings most significantly and also those that are most easily implemented. By using certain verifications, potential adjustments can be made in the number of lanes on a bridge, partial composite or full composite action can be applied to estimate the flexural capacity of the girders, some end restraint can be used to reduce the maximum positive moment demand, and refined analysis methods can be performed to inform the engineer when updating load ratings.

With respect to implementation, it is important to note that the current load rating procedures for steel bridges in Texas are based on the AASHTO Manual for Bridge Evaluation (AASHTO MBE 2018) and the AASHTO Standard Specifications (AASHTO 2002). In Article 6B1.1, the AASHTO MBE states, "there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented." Article 1.1.1 of the AASHTO Standard Specifications also states, "alternate rational analyses, based on theories or tests and accepted by the authority having jurisdiction, will be considered as compliance with these Specifications." Furthermore, load tests are allowed through Article 6B1.1 of the AASHTO MBE, which states, "Diagnostic load tests may be helpful in establishing the safe load capacity…" As such, the following recommendations provide specific guidance for refined load rating approaches applicable to typical steel multi-girder load posted bridges in Texas.

RECOMMENDATION | **COMMENTARY**

2.1 INSPECTION

The following should be performed during routine inspection of the bridge. Observations made will be relevant to the methods used to determine refined load ratings.

2.1.1 Geometry and Traffic

Examine and note the bridge geometry with respect to the roadway width, lane widths, and number of lanes.

C2.1.1 Geometry and Traffic

Refer to the NBI records for ADT and ADTT information.

2.1.2 Girder Flange Embedment

Examine if the top flanges of the girders are embedded in the concrete deck and estimate the depth of embedment. Confirm the depth of embedment relative to that shown in the structural drawings. If the flanges are embedded, examine the condition of the underside of the deck near the girder flanges.

C2.1.2 Girder Flange Embedment

Cracking of the deck near the top flanges of a bridge with embedded flanges could indicate that slippage is occurring between the deck and girders. If no cracks are present, this suggests that composite action between the girder and deck is occurring.

2.1.3 End Conditions

Examine the conditions at the ends of the bridge for signs of potential end restraint. Look for rust or deterioration causing locking between the girders and the bearing. If the top surface of the concrete deck is exposed, look for the presence of transverse tension cracks in the deck near the abutments.

C2.1.3 End Conditions

Cracking of the top surface of the deck near bridge ends could indicate the presence of end restraint leading to some negative moment at the girder ends. If significant, this can reduce the positive moment demand at midspan.

RECOMMENDATION **COMMENTARY**

2.1.4 Material Properties

Use the most accurate material property information available for capacity calculations during the load rating process. Material properties can be determined using suitable NDE techniques based on standard test procedures and through standard laboratory testing of extracted samples to obtain more accurate material data. Information regarding the reinforcing steel grade may also be determined from mill test certificates, if available, so the corresponding yield strength of steel for design may be used for load rating.

Improved concrete strength would be relevant when considering the presence of full or partial composite action between the girders and deck.

C2.1.4 Material Properties

The default material properties provided in the AASHTO MBE are based on bridge age and may not reflect the actual material strengths.

2.2 NUMBER OF LANES

Consider the bridge geometry and traffic conditions as observed during the inspection (Section 2.1). Bridges with a roadway width under 24 ft, experiencing a low ADTT, and with low likelihood of two design trucks passing each other on the bridge at the same time could be analyzed as a one-lane bridge, using one-lane LLDFs, if TxDOT deems appropriate.

A bridge meeting these criteria can be re-striped as a one-lane bridge where this does not impede functionality or safety.

C2.2 NUMBER OF LANES

TxDOT is already applying this approach to some two-lane bridges based on inspection records.

RECOMMENDATION

2.3 COMPOSITE ACTION

Three levels of analysis are proposed to consider partial composite action in the load rating process. A Level I analysis is performed prior to conducting a load test. A Level I analysis is used to indicate the potential benefit of confirming composite action to achieve an acceptable rating factor. A Level II analysis involves the use of a load test to confirm the composite behavior of a structure that is assumed to not be significantly affected by end restraint. A Level III analysis involves the use of a load test to confirm the composite behavior of a structure that is assumed to have some amount of end restraint causing a negative moment at the girder ends.

2.3.1 Level I Analysis

Level I analysis evaluates the potential benefit of composite action. An analysis is performed in the office supported by the inspection information, without conducting a load test, and therefore involves more uncertainty than a Level II or Level III analysis.

COMMENTARY

C2.3 COMPOSITE ACTION

A Level 1 analysis is used to determine the potential benefit of composite action in increasing the flexural rating factor for a bridge girder. When partial composite action is sufficient to remove the load posting, the potential for composite action is deemed promising. At this stage, further analysis and evaluation are warranted to reduce uncertainty. Level II and III analysis provide additional guidance for verification through field testing.

C2.3.1 Level I Analysis

The potential for composite action is greater for bridges with the girder top flanges embedded into the deck.

Level I analysis could be performed by first analyzing the bridge using fully composite and fully non-composite section assumptions. This provides upper and lower bound flexural rating factors for the bridge. If the use of composite action is promising, a Level II analysis should be performed to provide more certainty for updating the rating factor based on the presence of composite action.

RECOMMENDATION

2.3.1.1 Top Flanges Embedded

For bridges with top flanges embedded into the deck,

a) Check if the underside of the deck at the girder to deck interface is in good condition per the inspection.
b) If there is cracking observed at the underside of the deck, estimate an appropriate amount of partial composite action to use.

2.3.1.2 Nominal Moment Capacity

If partial composite action is to be considered, estimate a ratio of the maximum shear force that can be transferred across the interface over the maximum shear force that can be transferred across the interface when the section is fully composite. If full composite action is to be used, this ratio is 1.0.

Conduct a composite section analysis of the girder, and multiply the interface shear force for full composite action by the assumed ratio for composite or partially composite action.

Determine the plastic neutral axis location using the reduced concrete or steel force for composite behavior. Only consider the concrete area in compression.
Determine the partially composite moment capacity or fully composite moment capacity

COMMENTARY

C2.3.1.1 Top Flanges Embedded

Embedded top flanges, along with the absence of cracking at the flange to deck interface, suggests the potential for significant composite action. It is recommended to conduct a load test to confirm the assumption of significant composite action for steel girder bridges without studs.

C2.3.1.2 Nominal Moment Capacity

It is important to note that this analysis can provide an estimated upper bound for the moment capacity of a girder. A lower bound is found by assuming non-composite action. It may be difficult to reliably predict the level of composite action without a load test. If the use of composite action is promising, a Level II analysis is recommended to provide more certainty for updating the rating factor.

RECOMMENDATION | **COMMENTARY**

by summing the moments of the components about the neutral axis to obtain the plastic moment capacity. Continue with a composite section analysis as prescribed in the *AASHTO Standard Specifications* to obtain a nominal moment capacity.

2.3.2 Level II Analysis

A Level II analysis is performed in conjunction with conducting a load test, to reduce the uncertainty relative to an initial Level I analysis. The Level II analysis pertains to bridges with the steel girder top flanges either embedded into the deck or not embedded.

Importantly, this analysis method is intended for bridges in which end conditions do not appear to have a significant effect on girder end restraint. During the inspection, it should be confirmed that the bridge does not have any transverse tension cracking in the deck top face near the abutments, the girders do not show signs of deterioration causing locking with the bearings, and there is an open, unfilled gap between the girder ends and the back wall of the abutment.

C2.3.2 Level II Analysis

Neglecting the presence of end restraint can lead to overestimating the level of composition action because restraint at the girder ends can also contribute to reductions in the midspan deflection.

2.3.2.1 Level of Composite Action

Determine the theoretical composite and non-composite moment of inertia of a girder.

C2.3.2.1 Level of Composite Action

The suggested recommendations intentionally do not specify whether to analyze an interior girder or exterior girder

RECOMMENDATION

Determine the theoretical composite and non-composite deflections of the girder using an elastic analysis for a known truck used in load testing.

Conduct a load test with the known truck to determine an individual girder deflection of the same girder previously analyzed.

Prorate the measured test deflection by calculating its difference from the theoretical composite and non-composite deflections.

Calculate the acting moment of inertia of the girder I_{equiv} by interpolating between the composite and non-composite moments of inertia using the same prorated differences determined from the deflections.

The theoretical composite, theoretical non-composite, and acting partially composite moments of inertias are now known.

The acting partially composite moment of inertia estimate from the load test can be set equal to I_{equiv}. The following expression from the 14th edition AISC Steel Construction Manual (Eq. C-I3-4), can be used to determine the level of composite action.

$$I_{equiv} = I_{nc} + \sqrt{\frac{\sum Q_n}{C_f}}(I_c - I_{nc}) \quad (2.1)$$

COMMENTARY

when determining the level of composite action. When selecting an exterior or interior girder, it should be kept in mind that, depending on the method used, it may be easier to measure a deflection in the field for an exterior girder; however, the AASHTO LLDFs tend to be more accurate for interior girders. This could affect the calculated theoretical composite and non-composite deflection values.

A three-dimensional finite element method (FEM) model and analysis, when carried out correctly, can also provide a more accurate estimate of live load distribution and the corresponding expected deflection values in the field.

RECOMMENDATION | **COMMENTARY**

where:

I_{equiv} = Partial composite moment of inertia (in^4)

I_{nc} = Theoretical non-composite moment of inertia (in^4)

I_c = Theoretical composite moment of inertia (in^4)

ΣQ_n = Interface shear resistance (kip)

C_f = Interface shear resistance for fully composite action (kip)

Using Equation (2.1), determine the ratio $\frac{\sum Q_n}{C_f}$. Note that the equation is only valid for $\frac{\sum Q_n}{C_f} \geq 0.0625$. This is the ratio of the acting interface shear resistance to the interface shear resistance necessary for full composite action.

2.3.2.2 Nominal Moment Capacity

Multiply the interface shear force for full composite action by the $\frac{\sum Q_n}{C_f}$ ratio for composite section analysis.

Determine the plastic neutral axis location using the reduced interface shear force.

Determine the moment capacity of the partially composite or fully composite section by summing the moments of the internal force components about the neutral axis to obtain the plastic moment capacity. Continue with a composite section analysis

C2.3.2.2 Nominal Moment Capacity

It is important to note that this analysis provides an upper bound for the moment capacity of a girder. A lower bound is provided by assuming non-composite action.

RECOMMENDATION | **COMMENTARY**

as prescribed in the AASHTO Standard Specifications to obtain the corresponding nominal moment capacity. Finally, update the load rating calculations.

2.3.3 Level III Analysis

This level of analysis is performed in conjunction with conducting a load test, and therefore involves less uncertainty than a Level I analysis. It also provides additional considerations beyond a Level II analysis.

Level III analysis pertains to bridges with the steel girder top flanges either embedded into the deck or not embedded. This method of analysis is more appropriate when end restraint at the girder ends, due to unintended restraint at the bearings, may influence the moments along the girder span. During the inspection, signs of girder end restraint include transverse tension cracking in the deck to face near the abutments, signs of girder deterioration causing locking with the bearings, and the lack of an open, unfilled gap between the girder ends and the back wall of the abutment.

C2.3.3 Level III Analysis

Neglecting the presence of end restraint can lead to overestimating the level of composition action as restraint at the girder ends can also contribute to reductions in the midspan deflection.

2.3.3.1 Level of Composite Action

Determine the theoretical composite and non-composite moment of inertia of a girder.

C2.3.3.1 Level of Composite Action

The suggested recommendations intentionally do not specify whether to analyze an interior girder or exterior girder

RECOMMENDATION	COMMENTARY

Determine the theoretical composite and non-composite deflections of that girder using an elastic analysis.

Conduct a load test with a known truck to determine an individual girder deflection of the same girder previously analyzed. Also determine the strain at the end of the girder.

Prorate the measured test deflection by calculating its difference from the theoretical composite and non-composite deflections.

Calculate the acting moment of inertia of the girder I_{equiv} by interpolating between the composite and non-composite moments of inertia using the same prorated differences of the deflections.

The theoretical composite, theoretical non-composite, and acting partially composite moments of inertia are now known.

Using Equation (2.1), determine the $\frac{\sum Q_n}{C_f}$ ratio. Note that this equation is only valid for $\frac{\sum Q_n}{C_f} \geq 0.0625$. This is the ratio of the acting interface shear resistance over the interface shear resistance necessary for full composite action.

when determining the level of composite action. When selecting an exterior or interior girder, it should be kept in mind that, depending on the method used, it may be easier to measure a deflection in the field for an exterior girder; however, the AASHTO LLDFs tend to be more accurate for interior girders. This could affect the calculated theoretical composite and non-composite deflections.

A three-dimensional finite element method (FEM) model and analysis, when carried out correctly, can also provide a more accurate estimate of live load distribution and the corresponding expected deflection values in the field.

RECOMMENDATION

COMMENTARY

2.3.3.2 Girder Neutral Axis Location

Multiply the interface shear force for full composite action (smaller of $A_s f_y$ or $0.85 f_c' A_c$) by $\frac{\sum Q_n}{C_f}$ ratio for the composite section analysis.

Determine the plastic neutral axis location using the reduced interface shear force.

2.3.3.3 Nominal Moment Capacity Considering End Restraint

Add the magnitude of the deflection due to end restraint to the magnitude of the positive downward deflection measured during load testing. This will give a larger deflection value than measured during testing.

Prorate this deflection value between the theoretical composite and non-composite deflections.

Approximate the true moment of inertia of the girder using the same prorated amount between the composite and non-composite moments of inertia.

The theoretical composite, theoretical non-composite, and partial composite (true) moments of inertia are now known.

C2.3.3.3 Nominal Moment Capacity Considering End Restraint

It is important to note that the presence of end restraint could affect the measured deflections during load testing. Reduced deflections due to end restraint could suggest that the measured results exhibit higher levels of composite action than are actually occurring. Using higher levels of interface shear transfer can lead to an unconservative estimate of the moment capacity.

If signs of end restraint are observed during the inspection, and deflection results from the load test imply that end restraint could be occurring, it must be accounted for. This can be done by adding the deflection due to end restraint to the theoretical composite and theoretical non-composite deflections. This deflection value will use the relevant moment of inertia value, either the theoretical composite moment of inertia or the theoretical non-composite moment of

RECOMMENDATION **COMMENTARY**

Using Equation (2.1), determine the $\frac{\sum Q_n}{C_f}$ ratio. This is the ratio of the acting interface shear resistance to the interface shear resistance necessary for full composite action.

Multiply the interface shear force for full composite action by the $\frac{\sum Q_n}{C_f}$ ratio for composite section analysis.

Determine the plastic neutral axis location using the reduced interface shear force.

Determine the partially composite or fully composite moment capacity by summing the moments of the internal force components about the neutral axis to obtain the plastic moment capacity. Continue with a composite section analysis as prescribed in the AASHTO Standard Specifications to obtain the corresponding nominal moment capacity. Finally, update the load rating calculations.

inertia. The reduced deflection due to end restraint for a simply supported bridge can be determined using Eqn. (C2.2).

$$\Delta = \frac{|M|L^2}{8EI} \tag{C2.2}$$

where:

M = The average restraining moment at the two girder ends, obtained from field testing. (See Section 5.3 for an example calculation.)

L = Span length

E = Elastic modulus of steel

I = The relevant moment of inertia for either a composite or non-composite section

2.4 END RESTRAINT

Most multi-girder steel bridges include spans that are simply supported. Two levels of analysis may be used to consider the effect of unintended end restraint in the load rating process. A Level I analysis is performed without conducting a load test; however, the bridge behavior is therefore not confirmed. A Level II analysis involves

C2.4 END RESTRAINT

The suggested recommendations intentionally do not specify whether to analyze an interior girder or exterior girder when determining the level of composite action. When selecting an exterior or interior girder, it should be kept in mind that, depending on the method used, it may be easier to measure a deflection in

RECOMMENDATION

the use of a load test to confirm the bridge behavior.

2.4.1 Level I Analysis

Level I analysis is performed in the office, without conducting a load test, and therefore involves more uncertainty than a Level II analysis.

The critical bridge girder for load rating is modeled as a one-dimensional beam, and the maximum restraining moment is determined by considering the boundary conditions as fully restrained for all six degrees of freedom.

Once the value of the restraining moment at both ends of the bridge is known, determine a reduced midspan moment to use in load rating through Equation (2.3).

$$M_{midspan} = M_{truck-simple} - \frac{(M_{end1} + M_{end2})}{2} \qquad (2.3)$$

COMMENTARY

the field for an exterior girder; however, the AASHTO LLDFs tend to be more accurate for interior girders. This could affect the calculated theoretical composite and non-composite deflections.

A three-dimensional finite element method (FEM) model and analysis, when carried out correctly, can also provide a more accurate estimate of live load distribution and the corresponding expected deflection values in the field.

C2.4.1 Level I Analysis

The analysis shown includes determination of the RF considering fully fixed boundary conditions, along with the RF considering simply supported boundary conditions. These boundary conditions provide the upper and lower bound rating factors for the bridge based on end restraint. Depending on the upper and lower bound RFs (relative to 1.0), the bridge condition, and the judgment of the engineer, a Level I analysis can inform the load posting decision, and determine the need to continue to a Level II analysis to reduce uncertainty.

If the use of end restraint is promising, a Level II analysis is recommended to determine the level of end restraint present at the bridge girder ends using a load test.

RECOMMENDATION | **COMMENTARY**

where:

$M_{midspan}$ = Live load moment at midspan considering restraining moments at the ends of the girders

$M_{truck-simple}$ = Live load moment at midspan considering a simply supported boundary condition

M_{end} = Restraining moment at either end of the girder

Determine a new upper bound rating factor considering the fully restrained boundary conditions.

Determine a lower bound rating factor considering simply supported boundary conditions. This rating factor is the same as the currently determined rating factor.

Estimate the degree of restraint and corresponding rating factor to assign to the bridge based on the upper and lower bound rating factors. The commentary provides additional guidance to determine next steps, such as a Level II analysis.

2.4.2 Level II Analysis

Through a load test, verify that end restraint is occurring at the ends of a girder under loading using some method to infer a moment at the girder end, and through the visual inspection.

C2.4.2 Level II Analysis

As it is difficult to determine the amount of partial end restraint a bridge is exhibiting analytically, without conducting field testing, only one level of analysis to determine the end restraint in a structure is

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Determine the value of the restraining moment observed during the load test. Prorate this restraining moment value to the design truck and determine a reduced midspan moment to use in load rating through Equation (2.3).

Decide the amount of the difference between the theoretical moment and the calculated midspan moment to use based on field observations.

recommended. This procedure requires conducting a load test and measuring the strain in the bottom flange of a girder. The measured strain and location of the theoretical neutral axis can be used to determine the restraining moment at the end of a girder.

2.5 LIVE LOAD DISTRIBUTION FACTORS

The use of the AASHTO Standard Specification LLDFs is recommended for load rating calculations of multi-girder steel bridges.

Two levels of analysis are suggested when a bridge has a low load rating factor after considering the earlier recommendations. The possibility of reduced LLDFs can be determined. A Level I Analysis can be performed in the office, however requires the use of an FEM model. A Level II Analysis requires conducting a load test on the bridge.

2.5.1 Level I Analysis

Develop an FEM model of the bridge to determine a more accurate understanding of the live load distribution to the girders.

C2.5.1 Level I Analysis

Detailed guidance for developing refined FEM model of steel multi-girder bridges is provided in Chapter 2 of the Volume 2 report (Hueste et al. 2019b).

RECOMMENDATION | **COMMENTARY**

2.5.2 Level II Analysis

Conduct a load test on the bridge to determine the measured live load distribution to the girders.

C2.5.2 Level II Analysis

Detailed guidance for conducting nondestructive load tests and calculating live load distribution from measured results are provided in Chapter 6 of the Volume 2 report (Hueste et al. 2019b).

2.6 CONTINUITY CONSIDERATIONS

This suggested analysis pertains to continuous bridges and involves using fewer simplifying assumptions in the load rating process.

For dead load moment demand on continuous bridges, use continuous beam coefficients to determine moments when spans are approximately equal. Use a thorough multi-span structural analysis method to determine moments if spans are not equal.

For live load moment demand on continuous bridges, use a thorough multi-span structural analysis method to determine moments.

C2.6 CONTINUITY CONSIDERATIONS

TxDOT is currently using 0.75*L* or 0.8*L* to find positive moment region midspan moments for continuous bridges. The resulting moment values are used to load rate the bridge girders.

The suggested approach does not require the use of three-dimensional FEM models, but would provide a more accurate estimate of the demand moments that is likely less conservative than current practice.

3 RECOMMENDED PROCEDURES FOR REFINED LOAD RATING OF CONCRETE MULTI-GIRDER BRIDGES

Recommendations to improve the load rating of simple span concrete multi-girder bridges have been developed based on the results of this research project. These recommendations are supported by the project tasks, including a comprehensive literature review, identification of areas of improvement during basic load rating of similar bridges, examination of bridge behavior through finite element method (FEM) modeling and analysis, findings from load testing a representative concrete multi-girder bridge, and results from refinement and calibration of the associated FEM model.

The first section describes recommended procedures to be conducted during the bi-annual inspections of the bridges. Subsequent sections are presented in the order expected to be most efficient, by emphasizing approaches having potential to increase load ratings most significantly and also those that are most easily implemented. By using certain verifications, potential adjustments can be made in the number of lanes on a bridge, some end restraint can be used to reduce the maximum positive moment demand, and refined analysis methods can be performed to inform the engineer when updating load ratings.

With respect to implementation, it is important to note that the current load rating procedures for concrete bridges in Texas are based on the AASHTO Manual for Bridge Evaluation (AASHTO MBE 2018) and the AASHTO Standard Specifications (AASHTO 2002). In Article 6B1.1, the AASHTO MBE states, “there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented.” Article 1.1.1 of the AASHTO Standard Specifications also states, “alternate rational analyses, based on theories or tests and accepted by the authority having jurisdiction, will be considered as compliance with these Specifications.” Furthermore, load tests are allowed through Article 6B1.1 of the AASHTO MBE, which states, “Diagnostic load tests may be helpful in establishing the safe load capacity…” As such, the following recommendations provide specific guidance for refined load rating approaches applicable to typical concrete multi-girder load posted bridges in Texas.

RECOMMENDATION | **COMMENTARY**

3.1 INSPECTION

The following should be performed during routine inspection of the bridge. Observations made will be relevant to the methods used to determine refined load ratings.

3.1.1 Geometry and Traffic

Measure and record bridge geometry such as span length, roadway width, lane widths and number of lanes.

C3.1.1 Geometry and Traffic

Refer to the NBI records for ADT and ADTT information.

3.1.2 End Conditions

Examine the conditions at the ends of the bridge for signs of potential end restraint. Look for deterioration causing locking between the girders and the bearing. If the top surface of the concrete deck is exposed, look for the presence of transverse tension cracks in the deck near the abutments.

C3.1.2 End Conditions

Cracking of the top surface of the deck near bridge ends could indicate the presence of end restraint leading to some negative moment at the girder ends. If significant, this can reduce the positive moment demand at midspan.

3.1.3 Material Properties

Use the most accurate material property information available for capacity calculations during the load rating process. Material properties can be determined using suitable NDE techniques based on standard test procedures and through standard laboratory testing of extracted samples to obtain more accurate material data. Information regarding the reinforcing steel grade may also be determined from mill test

C3.1.3 Material Properties

The default material properties provided in the AASHTO MBE are based on bridge age and may not reflect the actual material strengths.

In-situ material strengths can be evaluated on site with the help of suitable nondestructive evaluate (NDE) equipment. If possible, standard laboratory testing should also be used by obtaining concrete

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certificates, if available, so the corresponding yield strength of steel for design may be used for load rating.	core samples and steel coupons from the bridge. The material strength could then be determined from these samples at the laboratory. An increase in material strength values would increase the capacity of the member. This would help increase the RF of the bridge.
3.2 NUMBER OF LANES	**C3.2 NUMBER OF LANES**
Consider the bridge geometry and traffic conditions as observed during the inspection (Section 3.1). Bridges with a roadway width under 24 ft, experiencing a low ADTT, and with low likelihood of two design trucks passing each other on the bridge at the same time could be analyzed as a one-lane bridge, using one-lane LLDFs, if TxDOT deems appropriate.	TxDOT is already applying this approach to some two-lane bridges based on inspection records.
A bridge meeting these criteria can be re-striped as a one-lane bridge where this does not impede functionality or safety.	
3.3 MATERIAL PROPERTIES	
The in-situ material strength may be higher than the AASHTO MBE recommended values or those prescribed in the as-built drawings. A higher material strength (concrete compressive strength and yield strength of rebar) would result in greater capacity of the component. Because capacity comes into play in the determination of the rating	

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factors (RF) of a bridge, this increased value would help increase the RFs of the bridge.

3.4 END RESTRAINT

Most multi-girder concrete bridges include spans that are simply supported. Two levels of analysis may be used to consider the effect of unintended end restraint in the load rating process. A Level I analysis is performed prior to conducting a load test. A Level I analysis can be used to indicate the potential benefit of confirming end restraint to achieve an acceptable rating factor. A Level II analysis involves the use of a load test to confirm the bridge behavior.

3.4.1 Level I Analysis

Level I analysis evaluates the potential benefit of end restraint. An analysis is performed in the office supported by the inspection information, without conducting a load test, and therefore involves more uncertainty than a Level II analysis.

The critical bridge girder for load rating is modeled as a one-dimensional beam, and the maximum restraining moment is determined by considering the boundary conditions as fully restrained for all six degrees of freedom.

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C3.4 END RESTRAINT

A Level 1 analysis is used to determine the potential benefit of end restraint in increasing the flexural rating factor for a bridge girder. When partial end restraint is sufficient to remove the load posting, the potential for end restraint is deemed promising. At this stage, further analysis and evaluation are warranted to reduce uncertainty. Level II analysis provides additional guidance for verification through field testing.

C3.4.1 Level I Analysis

The analysis shown includes determination of the RF considering fully fixed boundary conditions, along with the RF considering simply supported boundary conditions. These boundary conditions provide the upper and lower bound rating factors for the bridge based on end restraint. Depending on the upper and lower bound RFs (relative to 1.0), the bridge condition, and the judgment of the engineer, a Level I analysis can inform the load posting decision, and determined the need to continue to a Level II analysis to reduce uncertainty.

RECOMMENDATION	COMMENTARY

Once the value of the restraining moment at both ends of the bridge is known, determine a reduced midspan moment to use in load rating through Equation (2.3).

$$M_{midspan} = M_{truck-simple} - \frac{(M_{end1} + M_{end2})}{2} \quad (3.1)$$

where:

$M_{midspan}$ = Live load moment at midspan considering restraining moments at the ends of the girders

$M_{truck-simple}$ = Live load moment at midspan considering a simply supported boundary condition

M_{end} = Restraining moment at either end of the girder

Determine a new upper bound rating factor considering the fully restrained boundary conditions.

Determine a lower bound rating factor considering simply supported boundary conditions. This rating factor is the same as the currently determined rating factor.

When partial end restraint is sufficient to remove the load posting, the potential for end restraint is deemed promising. At this stage, further analysis and evaluation are warranted to reduce uncertainty. The commentary provides additional guidance to

If including the presence of end restraint is promising, a Level II analysis is recommended to determine the level of end restraint present at the bridge girder ends using a load test.

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determine next steps, such as a Level II analysis.

3.4.2 Level II Analysis

A Level II analysis is performed in conjunction with conducting a load test, to reduce the uncertainty relative to an initial Level I analysis.

Through a load test, verify that end restraint is occurring at the ends of a girder under loading using a suitable method of measurement to infer a moment at the girder end, and through the visual inspection.

Determine the value of the restraining moment observed during the load test. Prorate this restraining moment value to the design truck and determine a reduced midspan moment to use in load rating through Equation (2.3).

Decide the amount of the difference between the theoretical moment and the calculated midspan moment to use based on field observations.

C3.4.2 Level II Analysis

Because it is difficult to determine the amount of partial end restraint a bridge is exhibiting analytically without conducting field testing, only one level of analysis to determine the end restraint in a structure is recommended. This procedure requires conducting a load test and measuring the strain at the top and bottom of a girder, if possible. Alternatively, the measured bottom strain and location of the theoretical neutral axis can be used to determine the restraining moment at the end of a girder.

3.5 LIVE LOAD DISTRIBUTION FACTORS

The use of the AASHTO Standard Specification LLDFs is recommended for load

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rating calculations of multi-girder concrete bridges.

Two levels of analysis are suggested when a bridge has a low load rating factor after considering the earlier recommendations. The possibility of reduced LLDFs can be determined. A Level I Analysis can be performed in the office, however requires the use of an FEM model. A Level II Analysis requires conducting a load test on the bridge.

3.5.1 Level I Analysis

Develop an FEM model of the bridge to determine a more accurate understanding of the live load distribution to the girders.

C3.5.1 Level I Analysis

Detailed guidance for developing refined FEM model of concrete multi-girder bridges are provided in Chapter 4 of the Volume 2 report (Hueste et al. 2019b).

3.5.2 Level II Analysis

Conduct a load test on the bridge to determine the measured live load distribution to the girders.

C3.5.2 Level II Analysis

Detailed guidance for conducting nondestructive load test and calculating live load distribution from measured results are provided in Chapter 8 of the Volume 2 report (Hueste et al. 2019b).

4 RECOMMENDED PROCEDURES FOR REFINED LOAD RATING OF CONCRETE SLAB BRIDGES WITH INTEGRAL CURBS

Recommendations to improve the load rating of simple span concrete slab bridges with integral curbs have been developed based on the results of this research project. These recommendations are supported by the project tasks, including a comprehensive literature review, identification of areas of improvement during basic load rating of similar bridges, examination of bridge behavior through finite element method (FEM) modeling and analysis, findings from load testing a representative concrete slab bridge, and results from refinement and calibration of the associated FEM model.

The first section describes recommended procedures to be conducted during the bi-annual inspections of the bridges. Subsequent sections are presented in the order expected to be most efficient, by emphasizing approaches having potential to increase load ratings most significantly and also those that are most easily implemented. By using certain verifications, potential adjustments can be made in the number of lanes on a bridge, some end restraint can be used to reduce the maximum positive moment demand, and refined analysis methods can be performed to inform the engineer when updating load ratings.

With respect to implementation, it is important to note that the current load rating procedures for concrete bridges in Texas are based on the AASHTO Manual for Bridge Evaluation (AASHTO MBE 2018) and the AASHTO Standard Specifications (AASHTO 2002). In Article 6B1.1, the AASHTO MBE states, “there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented.” Article 1.1.1 of the AASHTO Standard Specifications also states, “alternate rational analyses, based on theories or tests and accepted by the authority having jurisdiction, will be considered as compliance with these Specifications.” Furthermore, load tests are allowed through Article 6B1.1 of the AASHTO MBE, which states, “Diagnostic load tests may be helpful in establishing the safe load capacity…” As such, the following recommendations provide specific guidance for refined load rating approaches applicable to typical concrete slab load posted bridges in Texas.

RECOMMENDATION | **COMMENTARY**

4.1 INSPECTION

The following should be performed during routine inspection of the bridge. Observations made will be relevant to the methods used to improve load ratings.

4.1.1 Geometry and Traffic

Measure and record bridge geometry such as span length, roadway width, lane widths and number of lanes.

C4.1.1 Geometry and Traffic

Refer to the NBI records for ADT and ADTT information.

4.1.2 End Conditions

Examine the conditions at the ends of the bridge for signs of potential end restraint. Look for deterioration causing locking between the slab and the bearing. If the top surface of the concrete deck is exposed, look for the presence of transverse tension cracks in the deck near the abutments.

C4.1.2 End Conditions

Cracking of the top surface of the deck near bridge ends could indicate the presence of end restraint leading to some negative moment at the ends of the bridge. If significant, this can reduce the positive moment demand at midspan.

4.1.3 Material Properties

Use the most accurate material property information available for capacity calculations during the load rating process. Material properties can be determined using suitable NDE techniques based on standard test procedures and through standard laboratory testing of extracted samples to obtain more accurate material data. Information regarding the reinforcing steel grade may also be determined from mill test certificates, if available, so the

C4.1.3 Material Properties

The default material properties provided in the AASHTO MBE are based on bridge age and may not reflect the actual material strengths.

In-situ material strengths may be evaluated on site with the help of suitable nondestructive evaluate (NDE) equipment. If possible, standard laboratory testing should also be used by obtaining concrete core samples and steel coupons from the

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corresponding yield strength of steel for design may be used for load rating.

4.2 NUMBER OF LANES

Consider the bridge geometry and traffic conditions as observed during the inspection (Section 4.1). Bridges with a roadway width under 24 ft, experiencing a low ADTT, and with low likelihood of two design trucks passing each other on the bridge at the same time could be analyzed as a one-lane bridge, using one-lane LLDFs, if TxDOT deems appropriate.

A bridge meeting these criteria can be re-striped as a one-lane bridge where this does not impede functionality or safety.

4.3 MATERIAL PROPERTIES

The in-situ material strength may be higher than the AASHTO MBE recommended values or those prescribed in the as-built drawings. A higher material strength (concrete compressive strength and yield strength of rebar) would result in greater capacity of the component. Because capacity comes into play in the determination of the rating

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bridge. The material strength could then be determined from these samples at the laboratory. An increase in material strength values would increase the capacity of the member. This would help increase the RF of the bridge.

C4.2 NUMBER OF LANES

TxDOT is already applying this approach to some two-lane bridges based on inspection records.

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factors (RF) of a bridge, this increased value would help increase the RFs of the bridge.

4.4 END RESTRAINT

Most concrete slab bridges include spans that are simply supported. Two levels of analysis may be used to consider the effect of unintended end restraint in the load rating process. A Level I analysis is performed prior to conducting a load test. A Level I analysis can be used to indicate the potential benefit of confirming end restraint to achieve an acceptable rating factor. A Level II analysis involves the use of a load test to confirm the bridge behavior.

4.4.1 Level I Analysis

Level I analysis evaluates the potential benefit of end restraint. An analysis is performed in the office supported by the inspection information, without conducting a load test, and therefore involves more uncertainty than a Level II analysis.

The critical bridge section for load rating is modeled as a one-dimensional beam, and the maximum restraining moment is determined by considering the boundary conditions as fully restrained for all six degrees of freedom.

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C4.4 END RESTRAINT

As the assumed bridge behavior is not confirmed via load test, a Level I analysis is inherently uncertain than a Level II analysis. Depending on how close the bridge is to passing and observations in the field by the engineer a Level I analysis can be used to assess whether a Level II analysis is of interest or is required.

C4.4.1 Level I Analysis

The analysis shown includes determination of the RF considering fully fixed boundary conditions, along with the RF considering simply supported boundary conditions. These boundary conditions provide the upper and lower bound rating factors for the bridge based on end restraint. Depending on the upper and lower bound RFs (relative to 1.0), the bridge condition, and the judgment of the engineer, a Level I analysis can inform the load posting decision, and determined the need to continue to a Level II analysis to reduce uncertainty.

RECOMMENDATION	COMMENTARY

Once the value of the restraining moment at both ends of the bridge is known, determine a reduced midspan moment to use in load rating through Equation (4.1).

$$M_{midspan} = M_{truck-simple} - \frac{(M_{end1} + M_{end2})}{2} \quad (4.1)$$

where:

$M_{midspan}$ = Live load moment at midspan considering restraining moments at the ends of the bridge

$M_{truck-simple}$ = Live load moment at midspan considering a simply supported boundary condition

M_{end} = Restraining moment at either end of the bridge

Determine a new upper bound rating factor considering the fully restrained boundary conditions.

Determine a lower bound rating factor considering simply supported boundary conditions. This rating factor is the same as the currently determined rating factor.

When partial end restraint is sufficient to remove the load posting, the potential for end restraint is deemed promising. At this stage, further analysis and evaluation are warranted to reduce uncertainty. The

If including the presence of end restraint is promising, a Level II analysis is recommended to determine the level of end restraint present at the bridge girder ends using a load test.

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commentary provides additional guidance to determine next steps, such as a Level II analysis.

4.4.2 Level II Analysis

A Level II analysis is performed in conjunction with conducting a load test to reduce the uncertainty relative to an initial Level I analysis.

Through a load test, verify that end restraint is occurring at the ends of the bridge under loading using a suitable method of measurement to infer a moment at the ends of the bridge, and through the visual inspection.

Determine the value of the restraining moment observed during the load test. Prorate this restraining moment value to the design truck and determine a reduced midspan moment to use in load rating through Equation (4.1).
Decide the amount of the difference between the theoretical moment and the calculated midspan moment to use based on field observations.

C4.4.2 Level II Analysis

As it is difficult to determine the amount of partial end restraint a bridge is exhibiting analytically without conducting field testing, only one level of analysis to determine the end restraint in a structure is recommended. This procedure requires conducting a load test and measuring the strain at the top and bottom of bridge ends, if possible. Alternatively, the measured strain and location of the theoretical neutral axis can be used to determine the restraining moment at the end of the bridge.

4.5 LIVE LOAD DISTRIBUTION FACTORS

The use of the Illinois Bulletin 346 (IB346) approach is recommended for determining the distribution of live load to the L-curb

C4.5 LIVE LOAD DISTRIBUTION FACTORS

TxDOT currently uses IB346 to load rate concrete slab bridges with integral curbs. In this approach, L-curb sections are defined

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sections defined by IB346. However, the distribution of moment to the mid-slab region should be found using the equivalent width for concrete slab bridges given in the AASHTO LRFD Specifications (AASHTO 2017) in cases where the AASHTO LRFD Specifications provides a higher moment estimate in comparison to IB346 method.

Alternatively, for the one-lane loading case, the equivalent width recommendations for slab bridges with integral edge beams by Amer et al. (1999) may be considered when the recommended equivalent width provides a higher moment estimate in the mid-slab region as compared to the IB346 method.

Two levels of analysis are suggested when a bridge has a low load rating factor after considering the above recommendations. The possibility of reduced moment demands can be determined. A Level I Analysis can be performed in the office, however requires

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as the curb plus a width of the slab that is four times the slab thickness. The slab portion between these L-curbs share the remainder of the moment. However, the mid-slab moments determined using IB346 approach were found to be unconservative based on load test results of a typical concrete slab bridge with integral curbs.

Amer et al. (1999) provides an empirical equation to calculate the equivalent width of concrete slab bridges with integral curbs as:

$$E = 6.89 + 0.23L \leq \frac{W}{N_L} \qquad \text{(C4.2)}$$

$$C_{edge} = 1.0 + 0.5\left(\frac{d_1}{3.28} - 0.15\right) \geq 1.0 \qquad \text{(C4.3)}$$

where:

E = Equivalent width for a truck load, ft

L = Span length, ft

W = Bridge width, ft

N_L = Number of design lanes

d_1 = Edge beam depth above slab thickness, ft

RECOMMENDATION	COMMENTARY
the use of an FEM model. A Level II Analysis requires conducting a load test for the bridge.	

4.5.1 Level I Analysis

Develop an FEM model of the bridge to determine a more accurate understanding of the live load distribution to the slab and curbs.

C4.5.1 Level I Analysis

Detailed guidance for developing refined FEM model of concrete slab bridges with integral curbs are provided in Chapter 5 of the Volume 2 report (Hueste et al. 2019b).

4.5.2 Level II Analysis

Conduct a load test on the bridge to determine the measured live load distribution to the slab and curbs.

C4.5.2 Level II Analysis

Detailed guidance for conducting nondestructive load test and calculating live load distribution from measured results are provided in Chapter 9 of the Volume 2 report (Hueste et al. 2019b).

5 LOAD RATING EXAMPLE: SIMPLY SUPPORTED STEEL MULTI-GIRDER BRIDGE USING LFR METHOD

5.1 BASIC LOAD RATING ANALYSIS ASSUMING NON-COMPOSITE GIRDERS

This section shows an abbreviated example of the initial basic load rating performed for Bridge SM-5, a steel multi-girder bridge, considering interior girder flexure. This basic load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). The initial load rating is performed with the assumption that the bridge is acting without composite action at the girder-to-deck interface and that the girder ends are both simply supported. Therefore, no end restraint is considered. This initial load rating is used for comparison when conducting load ratings assuming partial composite action, partial end restraint, or a combination of the two.

5.1.1 Bridge Characteristics

Bridge SM-5 is a two-lane bridge with a span length of 40'-2", a deck width of 24'-0", and a roadway width of 23'-6". The girders are braced at third points. The TxDOT HS-20 RFs are 0.47 for Inventory and 0.79 for Operating. Figure 5.1 shows a transverse section of Bridge SM-5. The main bridge characteristics needed for load rating are summarized below.

Steel Girder Section: S15x42.9

Yield Stress: $F_y = 33 \text{ ksi}$

Concrete Compressive Strength: $f'_c = 2.5 \text{ ksi}$

Concrete Density: $\gamma_c = 150 \text{ pcf}$

Deck Thickness: $t_d = 6 \text{ in.}$

Assumed Railing Linear Weight: $w_{rail} = 20 \ \frac{\text{lb}}{\text{ft}}$

Span Length: $L = 40 \text{ ft} - 2 \text{ in.}$

Stringer Spacing: $S = 1.917 \text{ ft}$

Deck Overhang: $S_{overhang} = 6 \text{ in.}$

Asphalt Density: $\gamma_{ws} = 140 \text{ pcf}$

Asphalt Thickness: $t_{ws} = 1 \text{ in.}$

Number of Girders: $N_G = 13$

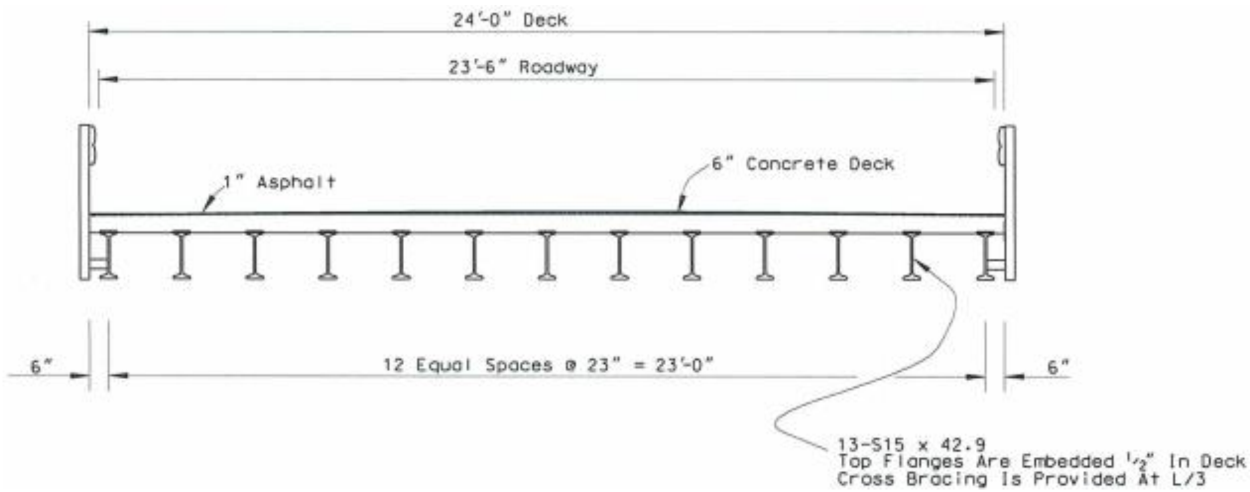

Figure 5.1. Transverse Section of Bridge SM-5 (TxDOT 2018a)

5.1.2 Sectional Properties of Steel Girder

Steel Girder Section: S15x42.9

Area: $A = 12.6\ \mathrm{in}^2$

Major Axis Moment of Inertia: $I_x = 446\ \mathrm{in}^4$

Elastic Section Modulus: $S_x = 59.4\ \mathrm{in}^3$

Plastic Section Modulus: $Z_x = 69.2\ \mathrm{in}^3$

Total Depth: $d = 15\ \mathrm{in.}$

Web Height: $h_w = 13.756\ \mathrm{in.}$

Web Thickness: $t_w = 0.411\ \mathrm{in.}$

Flange Width: $b_f = 5.5\ \mathrm{in.}$

Flange Thickness: $t_f = 0.622\ \mathrm{in.}$

Web Area: $A_w = 5.654\ \mathrm{in}^2$

Flange Area: $A_f = 3.421\ \mathrm{in}^2$

Weak axis radius of gyration: $r_y = 1.06\ \mathrm{in.}$

Web Depth in Compression: $D_c = 6.878\ \mathrm{in.}$

5.1.3 Moment Capacity

To determine the moment capacity for an individual girder, the procedure laid out in Section 10.48 of the AASHTO Standard Specifications (AASHTO 2002) is followed. First, a compact section check is performed. This involves checking the compression flange and web for compactness, and the braced length against a limit. If the girder does not pass one of these checks, a braced noncompact section check is performed. This involves the same checks as before, with different limit values. Finally, if the girder does not pass the lateral bracing check, it is analyzed as a partially braced member.

5.1.3.1 Compact Section Check

Check if section is compact – AASHTO Standard Specifications Article 10.48.1.1.

a) Check compression flange

$$\frac{b_f}{t_f} \leq \frac{4110}{\sqrt{F_y}} \tag{5.1}$$

$$\frac{5.5\text{ in.}}{0.622\text{ in.}} \leq \frac{4110}{\sqrt{33{,}000}}$$

$$8.84 \leq 22.6$$

Compression flange is compact (OK).

b) Check web thickness

$$\frac{h_w}{t_w} \leq \frac{19230}{\sqrt{F_y}} \tag{5.2}$$

$$\frac{13.756\text{ in.}}{0.411\text{ in.}} \leq \frac{19{,}230}{\sqrt{33{,}000}}$$

$$33.47 \leq 105.9$$

Web thickness is compact (OK).

c) Check spacing of lateral bracing for compression flange (moments are obtained from applied moment analysis later)

$$\frac{L_b}{r_y} \leq \frac{\left(3.6 - 2.2\left(\frac{M_1}{M_u}\right)\right)(10^6)}{F_y} \tag{5.3}$$

$$\frac{160.67\text{ in.}}{1.06\text{ in.}} \leq \frac{\left(3.6 - 2.2\left(\frac{91.1}{190.3}\right)\right)(10^6)}{33{,}000}$$

$$151.6 \geq 77.2 \quad \text{(No Good)}$$

Where M_1 is the smaller moment at the end of the unbraced length of the member and M_u is equal to the plastic moment capacity.

Spacing of lateral bracing for compression flange is NOT OK.

5.1.3.2 Braced Non-Compact Section Check

Check if braced non-compact section – AASHTO Standard Specifications Article 10.48.2.

a) Check compression flange

$$\frac{b_f}{t_f} \leq 24 \tag{5.4}$$

$$\frac{5.5\text{ in.}}{0.622\text{ in.}} \leq 24$$

$$8.84 \leq 24$$

Compression flange is OK.

b) Check web thickness. Web thickness is OK per compact section check.

c) Check spacing of lateral bracing for compression flange

$$L_b \leq \frac{20{,}000{,}000A_f}{F_y d} \tag{5.5}$$

$$160.67\text{ in.} \leq \frac{(20{,}000{,}000)(3.421)}{(33{,}000)(15)}$$

$$160.67\text{ in.} \leq 138.2\text{ in.}$$

Spacing of lateral bracing for compression flange is NOT OK.

5.1.3.3 Partially Braced Section Analysis

Analyze as partially braced section – AASHTO Standard Specifications Article 10.48.4.

The factor λ used in partially braced member moment capacity calculations can be taken as 15,400.

$$\lambda = 15{,}400$$

The bending coefficient C_b can conservatively be taken as 1.0.

$$C_b = 1.0$$

The moment of inertia of the compression flange about the vertical axis I_{yc} is calculated as:

$$\begin{aligned} I_{yc} &= \frac{1}{12} t_f b_f^3 \qquad (5.6) \\ &= \left(\frac{1}{12}\right)(0.622\text{ in.})(5.5\text{ in.})^3 = 8.62\text{ in}^4 \end{aligned}$$

The radius of gyration of the compression flange about the vertical axis r_y' can be calculated as:

$$\begin{aligned} r'_y &= \sqrt{\frac{I_{yc}}{A_f}} \qquad (5.7) \\ &= \sqrt{\frac{8.62\text{ in}^4}{3.421\text{ in}^2}} = 1.59\text{ in.} \end{aligned}$$

The torsional property J may be computed as:

$$\begin{aligned} J &= \frac{2\left(b_f t_f^3\right) + \left(h_w t_w^3\right)}{3} \qquad (5.8) \\ &= \frac{2[(5.5)(0.622^3)] + [(13.756)(0.411^3)]}{3} = 1.2\text{ in}^4 \end{aligned}$$

Check,

$$\frac{D_c}{t_w} \leq \frac{\lambda}{\sqrt{F_y}} \tag{5.9}$$

$$\frac{6.878\text{ in.}}{0.411\text{ in.}} \leq \frac{15400}{\sqrt{33{,}000}}$$

$$16.7 \leq 84.8$$

Check is OK. Therefore, flexural resistance M_r may be calculated as:

$$M_r = (91)(10^6)C_b\left(\frac{I_{yc}}{L_b}\right)\sqrt{\frac{0.722J}{I_{yc}} + 9.87\left(\frac{d}{L_b}\right)^2} \leq F_y S_x \tag{5.10}$$

$$= (91)(10^6)(1.0)\left(\frac{8.62}{160.67}\right)\sqrt{\frac{(0.722)(1.2)}{8.62} + 9.87\left(\frac{15}{160.67}\right)^2} \leq (33\text{ ksi})(59.4\text{ in}^3)$$

$$= 2108.6\text{ kip–in.} \leq 1960\text{ kip–in.} = 1960\text{ kip–in.}$$

$$M_r = 163.3\text{ kip–ft}$$

Calculate the bending capacity reduction factor R_b as:

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_f}\right)\left(\frac{D_c}{t_w} - \frac{\lambda}{\sqrt{\frac{M_r}{S_x}}}\right) \leq 1.0 \tag{5.11}$$

$$= 1 - 0.002\left[\frac{(6.878)(0.411)}{3.421}\right]\left(\frac{6.878}{0.411} - \frac{15400}{\sqrt{\frac{2108600}{59.4}}}\right) \leq 1.0$$

$$= 1.11 \leq 1.0$$

$$R_b = 1.0$$

Therefore, the final moment capacity M_n is equal to:

$$M_n = R_b M_r \tag{5.12}$$

$$= (1.0)(163.3\text{ kip–ft}) = 163.3\text{ kip–ft}$$

5.1.4 Structural Analysis for Moment Demand

5.1.4.1 Applied Live Load Moment

For LFR, use the LLDF equations provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. As Bridge SM-5 is a two-lane bridge, for an interior girder:

$$\begin{aligned} DF &= \frac{\mathrm{S}}{5.5\ \mathrm{ft}} \\ &= \frac{1.917\ \mathrm{ft}}{5.5\ \mathrm{ft}} = 0.348 \end{aligned} \tag{5.13}$$

The Impact Factor I is applied to the live load effect to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to:

$$\begin{aligned} I &= \frac{50}{L+125} \leq 0.3 \\ &= \frac{50}{40.167+125} \leq 0.3 \\ &= 0.3 \leq 0.3 \\ I &= 0.3 \end{aligned} \tag{5.14}$$

From interpolation in the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1, the applied live load moment M_{HS20} without the application of the impact factor and the LLDF is equal to:

$$M_{HS20} = 226.4\ \frac{\mathrm{kip{-}ft}}{\mathrm{wheel\ line}}$$

Therefore, the distributed applied live load moment M_{LL} with dynamic effects, on an individual interior girder can be calculated as:

$$\begin{aligned} M_{LL} &= M_{HS20} DF(1+I) \\ &= (226.4)(0.348)(1.3) = 102.4\ \mathrm{kip{-}ft} \end{aligned} \tag{5.15}$$

5.1.4.2 Dead Load Moment of Structural Components

The girder distributed weight w_G for an S15x42.9 section is equal to:

$$w_G = 0.043\ \frac{\text{kip}}{\text{ft}}$$

The distributed load from the deck w_d on an individual girder can be calculated as:

$$\begin{aligned} w_d &= \gamma_c t_d S \\ &= (150\text{ pcf})\left(\frac{6\text{ in.}}{12\,\frac{\text{in.}}{\text{ft}}}\right)(1.917\text{ ft}) = 144\ \frac{\text{lb}}{\text{ft}} = 0.144\,\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{5.16}$$

Therefore, the total distributed load due to dead load of structural components w_{DC} is equal to:

$$\begin{aligned} w_{DC} &= w_G + w_d \\ &= \left(0.043\ \frac{\text{kip}}{\text{ft}}\right) + \left(0.144\,\frac{\text{kip}}{\text{ft}}\right) = 0.187\,\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{5.17}$$

And the applied moment due to dead load of structural components M_{DC} can be calculated as:

$$\begin{aligned} M_{DC} &= \frac{w_{DC}L^2}{8} \\ &= \frac{\left(0.187\,\frac{\text{kip}}{\text{ft}}\right)(40.167\text{ ft})^2}{8} = 37.7\text{ kip–ft} \end{aligned} \tag{5.18}$$

5.1.4.3 Superimposed Dead Load Moment

The superimposed dead load on an individual girder due to the railing w_{rail} can be calculated as:

$$\begin{aligned} w_{rail} &= w_{rail}\,\frac{2}{N_G} \\ &= \left(20\ \frac{\text{lb}}{\text{ft}}\right)\left(\frac{2}{13}\right) = 3\ \frac{\text{lb}}{\text{ft}} = 0.003\,\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{5.19}$$

The superimposed dead load on an individual girder due to the wearing surface w_{ws} can be calculated as:

$$\begin{aligned} w_{ws} &= \gamma_{ws} t_{ws} S \\ &= (140 \text{ pcf})\left(\frac{1 \text{ in.}}{12 \frac{\text{in.}}{\text{ft}}}\right)(1.917 \text{ ft}) = 22 \frac{\text{lb}}{\text{ft}} = 0.022 \frac{\text{kip}}{\text{ft}} \end{aligned} \tag{5.20}$$

Therefore, the total distributed load due to superimposed dead load w_{SDL} is equal to:

$$\begin{aligned} w_{SDL} &= w_{rail} + w_{ws} \\ &= \left(0.003 \frac{\text{kip}}{\text{ft}}\right) + \left(0.022 \frac{\text{kip}}{\text{ft}}\right) = 0.025 \frac{\text{kip}}{\text{ft}} \end{aligned} \tag{5.21}$$

And the applied moment due to superimposed dead load M_{SDL} can be calculated as:

$$\begin{aligned} M_{SDL} &= \frac{w_{SDL} L^2}{8} \\ &= \frac{\left(0.025 \frac{\text{kip}}{\text{ft}}\right)(40.167 \text{ ft})^2}{8} = 5.0 \text{ kip–ft} \end{aligned} \tag{5.22}$$

5.1.4.4 Total Dead Load Moment

Therefore, the total applied moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$\begin{aligned} M_{DL} &= M_{DC} + M_{SDL} \\ &= (37.7 \text{ kip–ft}) + (5.0 \text{ kip–ft}) = 42.7 \text{ kip–ft} \end{aligned} \tag{5.23}$$

5.1.5 LFR Load Rating for Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2 (L + I)} \tag{5.24}$$

5.1.5.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$RF_I = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.25}$$

$$= \frac{163.3 - [(1.3)(42.7)]}{(2.17)(102.4)} = 0.49$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.26}$$

$$= \frac{163.3 - [(1.3)(42.7)]}{(1.3)(102.4)} = 0.81$$

Does not pass.

5.1.5.2 Service Check

For non-composite sections, per AASHTO Standard Specifications Article 10.57.1, the service capacity C_{serv} is equal to:

$$\begin{aligned} C_{serv} &= 0.8F_y \\ &= (0.8)(33\text{ ksi}) = 26.4\text{ ksi} \end{aligned} \tag{5.27}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$\begin{aligned} RF_I &= \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \\ &= \frac{26.4 - (1.0)\left[\frac{(42.7)(12)}{59.4}\right]}{(1.67)\left[\frac{(102.4)(12)}{59.4}\right]} = 0.51 \end{aligned} \tag{5.28}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$\begin{aligned} RF_O &= \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \\ &= \frac{26.4 - (1.0)\left[\frac{(42.7)(12)}{59.4}\right]}{(1.0)\left[\frac{(102.4)(12)}{59.4}\right]} = 0.86 \end{aligned} \tag{5.29}$$

Does not pass.

5.1.6 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge SM-5 from the basic load rating analysis are equal to:

$$RF_I = 0.49$$

$$RF_O = 0.81$$

Note that the above values are very close to those reported by TxDOT (0.47 for Inventory and 0.79 for Operating). For both inventory and operating ratings, the RFs are less than 1.0, and therefore do not pass these load rating according the AASHTO MBE.

5.2 LOAD RATING ANALYSIS CONSIDERING REDUCED NUMBER OF LANES

This section shows an abbreviated version of a load rating analysis performed for Bridge SM-5, a steel multi-girder bridge, considering interior girder flexure. In addition, a reduction in the number of lanes used in analysis is considered due to geometric observations and traffic conditions (Section 2.2). This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). It is also performed under the assumption that the bridge is acting non-compositely and that the girder ends are both roller-supported. Therefore, no end restraint is considered. This load rating with a reduced number of lanes is compared to the basic load rating analysis.

5.2.1 Bridge Characteristics

Bridge SM-5 is a two-lane bridge with a span length of 40'-2", a deck width of 24'-0", and a roadway width of 23'-6". Due to this geometry and the low ADTT of 18, it is very unlikely that two design vehicles will pass on the bridge at the same time. Therefore, the bridge is analyzed as a one-lane bridge to determine the potential change to the rating factors. The girders are braced at third points. The TxDOT HS-20 RFs for this bridge are 0.47 for Inventory and 0.79 for Operating. Figure 5.2 shows a transverse section of Bridge SM-5.

Steel Girder Section: S15x42.9

Yield Stress: $F_y = 33\ \mathrm{ksi}$

Concrete Compressive Strength: $f'_c = 2.5\ \mathrm{ksi}$

Concrete Density: $\gamma_c = 150\ \mathrm{pcf}$

Deck Thickness: $t_d = 6\ \mathrm{in.}$

Assumed Railing Linear Weight: $w_{rail} = 20\ \frac{\mathrm{lb}}{\mathrm{ft}}$

Span Length: $L = 40.167\ \mathrm{ft}$

Stringer Spacing: $S = 1.917\ \mathrm{ft}$

Deck Overhang: $S_{overhang} = 0.5\ \mathrm{ft}$

Asphalt Density: $\gamma_{ws} = 140\ \mathrm{pcf}$

Asphalt Thickness: $t_{ws} = 1\ \mathrm{in.}$

Number of Girders: $N_G = 13$

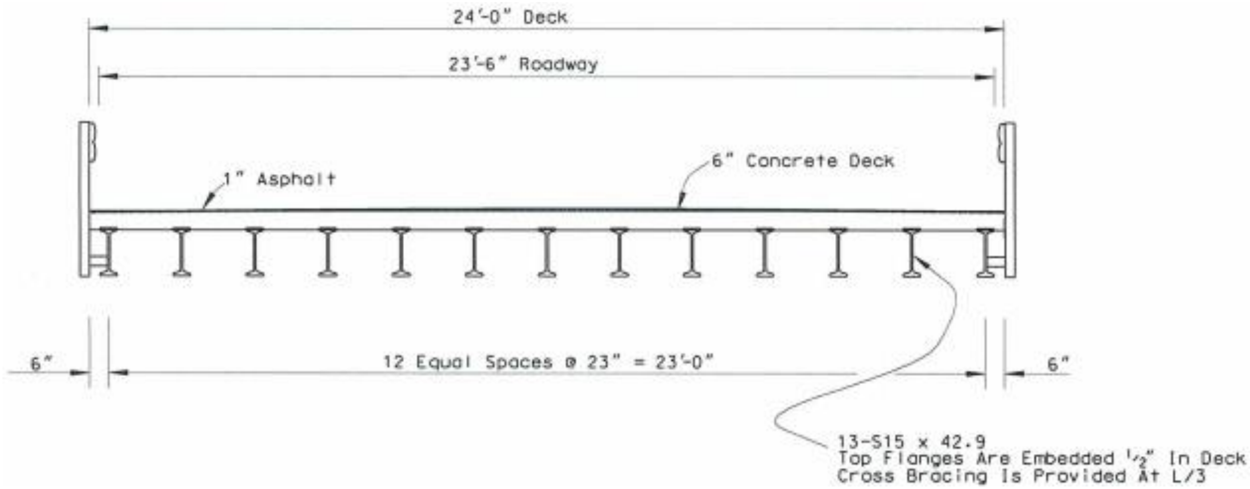

Figure 5.2. Transverse Section of Bridge SM-5 (TxDOT 2018a)

5.2.2 Sectional Properties of Steel Girder

Steel Girder Section: S15x42.9

Area: $A = 12.6 \text{ in}^2$

Major Axis Moment of Inertia: $I_x = 446 \text{ in}^4$

Elastic Section Modulus: $S_x = 59.4 \text{ in}^3$

Plastic Section Modulus: $Z_x = 69.2 \text{ in}^3$

Total Depth: $d = 15 \text{ in.}$

Web Height: $h_w = 13.756 \text{ in.}$

Web Thickness: $t_w = 0.411 \text{ in.}$

Flange Width: $b_f = 5.5 \text{ in.}$

Flange Thickness: $t_f = 0.622 \text{ in.}$

Web Area: $A_w = 5.654 \text{ in}^2$

Flange Area: $A_f = 3.421 \text{ in}^2$

Weak axis radius of gyration: $r_y = 1.06 \text{ in.}$

Web Depth in Compression: $D_c = 6.878 \text{ in.}$

5.2.3 Moment Capacity

Detailed in Section 5.1.3, the moment capacity M_n for an individual girder was determined to be:

$$M_n = 163.3 \text{ kip–ft}$$

5.2.4 Structural Analysis for Moment Demand

5.2.4.1 Applied Live Load Moment

For LFR, use the LLDF equations provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. As Bridge SM-5 is analyzed as a one-lane bridge, for an interior girder

$$
\begin{aligned}
DF &= \frac{\mathrm{S}}{7\ \mathrm{ft}} \qquad (5.30)\\
&= \frac{1.917\ \mathrm{ft}}{7\ \mathrm{ft}} = 0.274
\end{aligned}
$$

The Impact Factor I is applied to the live load effect to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to:

$$
\begin{aligned}
I &= \frac{50}{L+125} \leq 0.3 \qquad (5.31)\\
&= \frac{50}{40.167+125} \leq 0.3\\
&= 0.3 \leq 0.3\\
I &= 0.3
\end{aligned}
$$

From interpolation in the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1, the applied live load moment M_{HS20} without the application of the impact factor and the LLDF is equal to:

$$M_{HS20} = 226.4\ \frac{\mathrm{kip\text{-}ft}}{\mathrm{wheel\ line}}$$

Therefore, the distributed applied live load moment M_{LL} with dynamic effects, on an individual interior girder can be calculated as:

$$
\begin{aligned}
M_{LL} &= M_{HS20} DF(1+I) \qquad (5.32)\\
&= (226.4)(0.274)(1.3) = 80.6\ \mathrm{kip\text{-}ft}
\end{aligned}
$$

5.2.4.2 Dead Load Moment of Structural Components

Detailed in Section 5.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} \quad = \quad 37.7 \text{ kip–ft}$$

5.2.4.3 Superimposed Dead Load Moment

Detailed in Section 5.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} \quad = \quad 5.0 \text{ kip–ft}$$

5.2.4.4 Total Dead Load Moment

The total applied moment due to structural dead loads and superimposed dead loads M_{DL} is then equal to:

$$M_{DL} \quad = \quad 42.7 \text{ kip–ft}$$

5.2.5 LFR Load Rating for Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{5.33}$$

5.2.5.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$RF_I \quad = \quad \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.34}$$

$$= \quad \frac{163.3 - [(1.3)(42.7)]}{(2.17)(80.6)} = 0.62$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.35}$$

$$= \frac{163.3 - [(1.3)(42.7)]}{(1.3)(80.6)} = 1.03$$

Passes

5.2.5.2 Service Check

For non-composite sections, per AASHTO Standard Specifications Article 10.57.1 the service capacity C_{serv} is equal to:

$$C_{serv} = 0.8F_y \tag{5.36}$$

$$= (0.8)(33\ \text{ksi}) = 26.4\ \text{ksi}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$RF_I = \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \tag{5.37}$$

$$= \frac{26.4 - (1.0)\left[\frac{(42.7)(12)}{59.4}\right]}{(1.67)\left[\frac{(80.6)(12)}{59.4}\right]} = 0.65$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$RF_O = \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \tag{5.38}$$

$$= \frac{26.4 - (1.0)\left[\frac{(42.7)(12)}{59.4}\right]}{(1.0)\left[\frac{(80.6)(12)}{59.4}\right]} = 1.09$$

Passes.

5.2.6 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge SM-5 from the lane reduction analysis are equal to:

$$RF_I = 0.62$$

$$RF_O = 1.03$$

The basic load rating controlling RFs were 0.49 for Inventory and 0.81 for Operating. The new rating factors represent a 25.5 percent increase for Inventory and a 25.6 percent increase for Operating. Table 5.1 compares the controlling RFs determined using a reduction in number of lanes to the controlling RFs determined in the basic load rating.

Table 5.1. Lane Reduction RF Comparison

Rating Factor	Basic Load Rating	Load Rating with Lane Reduction	Lane Reduction/Basic Load Rating
Inventory	0.49	0.62	1.27
Operating	0.81	1.03	1.27

5.3 LOAD RATING ANALYSIS CONSIDERING ONLY COMPOSITE ACTION

This section shows an abbreviated example of a load rating performed for Bridge SM-5, a steel multi-girder bridge, considering interior girder flexure with partial composite action. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). In addition, a Level II Analysis (Section 2.3.2) for partial composite action is also performed to determine an increased moment capacity for the girder. This Level II Analysis is based on the results of the load test performed on the bridge in the field (Hueste et al. 2019b). The results of this load rating are compared to the results of the basic load rating analysis.

5.3.1 Bridge Characteristics

Bridge SM-5 is a two-lane bridge with a span length of 40'-2", a deck width of 24'-0", and a roadway width of 23'-6". The girders are braced at third points. The TxDOT HS-20 RFs for this bridge are 0.47 for Inventory and 0.79 for Operating. Figure 5.3 shows a transverse section of Bridge SM-5.

Steel Girder Section: S15x42.9

Yield Stress: $F_y = 33\ \text{ksi}$

Concrete Compressive Strength: $f'_c = 2.5\ \text{ksi}$

Concrete Density: $\gamma_c = 150\ \text{pcf}$

Deck Thickness: $t_d = 6\ \text{in.}$

Assumed Railing Linear Weight: $w_{rail} = 20\ \frac{\text{lb}}{\text{ft}}$

Span Length: $L = 40.167\ \text{ft}$

Stringer Spacing: $S = 1.917\ \text{ft}$

Deck Overhang: $S_{overhang} = 0.5\ \text{ft}$

Asphalt Density: $\gamma_{ws} = 140\ \text{pcf}$

Asphalt Thickness: $t_{ws} = 1\ \text{in.}$

Number of Girders: $N_G = 13$

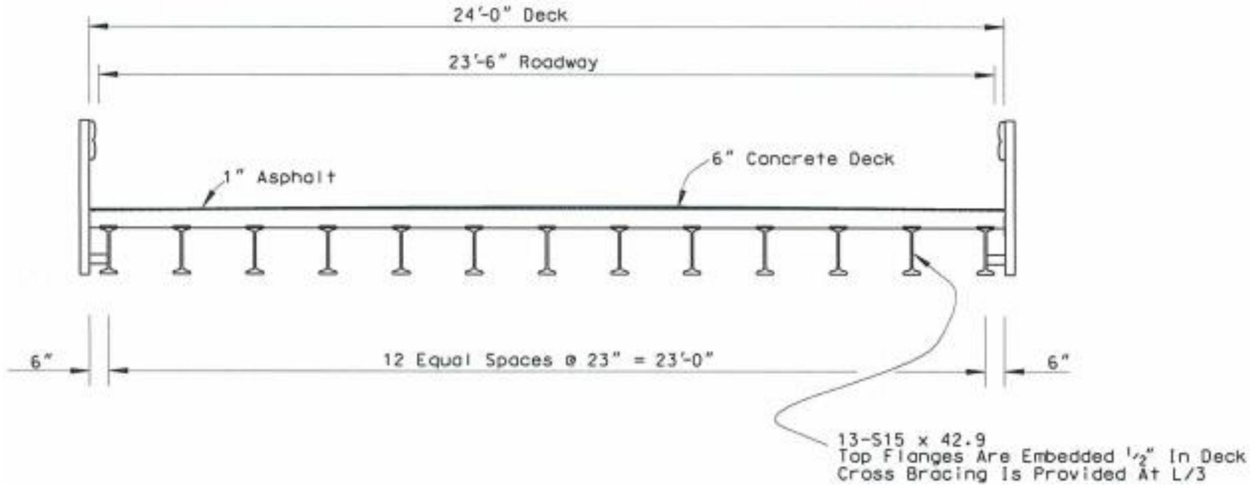

Figure 5.3. Transverse Section of Bridge SM-5 (TxDOT 2018a)

5.3.2 Sectional Properties of Steel Girder

Steel Girder Section: S15x42.9

Area: $A = 12.6\ \text{in}^2$

Major Axis Moment of Inertia: $I_x = 446\ \text{in}^4$

Elastic Section Modulus: $S_x = 59.4\ \text{in}^3$

Plastic Section Modulus: $Z_x = 69.2\ \text{in}^3$

Total Depth: $d = 15\ \text{in.}$

Web Height: $h_w = 13.756\ \text{in.}$

Web Thickness: $t_w = 0.411\ \text{in.}$

Flange Width: $b_f = 5.5\ \text{in.}$

Flange Thickness: $t_f = 0.622\ \text{in.}$

Web Area: $A_w = 5.654\ \text{in}^2$

Flange Area: $A_f = 3.421\ \text{in}^2$

Weak axis radius of gyration: $r_y = 1.06\ \text{in.}$

Web Depth in Compression: $D_c = 6.878\ \text{in.}$

5.3.3 Moment Capacity

5.3.3.1 Determination of Amount Composite

The theoretical composite moment of inertia of an interior girder and deck I_c can be found using a transformed section analysis as:

$$I_c = 1329\ \text{in}^4$$

The theoretical non-composite moment of inertia of the interior girder I_{nc} is that of the girder section:

$$I_{nc} = 446 \text{ in}^4$$

The theoretical composite deflection Δ_c of an individual interior girder, G7, under Middle Path loading can be found as:

$$\Delta_c = 0.131 \text{ in.}$$

The theoretical non-composite deflection Δ_{nc} of an individual interior girder, G7, under Middle Path loading can be found as:

$$\Delta_{nc} = 0.349 \text{ in.}$$

The measured test deflection Δ_{test} of Girder 7 under Middle Path loading is 0.145 in.

$$\Delta_{test} = 0.145 \text{ in.}$$

Therefore, the prorated deflection ratio $\Delta_{prorated}$ is:

$$\begin{aligned} \Delta_{prorated} &= \frac{\Delta_{nc} - \Delta_{test}}{\Delta_{nc} - \Delta_c} \\ &= \frac{0.349 \text{ in.} - 0.145 \text{ in.}}{0.349 \text{ in.} - 0.131 \text{ in.}} = 0.94 \end{aligned} \quad (5.39)$$

Therefore, the acting moment of inertia of the girder I_{equiv} can be approximated as:

$$\begin{aligned} I_{equiv} &= I_{nc} + \Delta_{proprated}(I_c - I_{nc}) \\ &= 446 + 0.94(1329 - 446) = 1272\ \text{in}^4 \end{aligned} \tag{5.40}$$

Substituting known values into Equation C-I3-4 in the 14th edition of the AISC Steel Construction Manual yields:

$$\begin{aligned} I_{equiv} &= I_{nc} + \sqrt{\frac{\sum Q_n}{C_f}}(I_c - I_{nc}) \xrightarrow{yields} \\ 1272 &= 446 + \sqrt{\frac{\sum Q_n}{C_f}}(1329 - 446) \end{aligned} \tag{5.41}$$

Solving for $\frac{\sum Q_n}{C_f}$ yields:

$$\frac{\sum Q_n}{C_f} = 0.88$$

This ratio is used to estimate the interface shear force for the composite section analysis.

5.3.3.2 Determination of Nominal Moment Capacity

Assume the slab is only 5.5 in. thick for nominal moment capacity calculations, as the girder flange is embedded 0.5 in. into the 6 in. slab.

The unreduced force in the slab C_i is equal to:

$$\begin{aligned} C_i &= 0.85 f'_c b_e t_d \\ &= 0.85(2.5\ \text{ksi})(23\ \text{in.})(5.5\ \text{in.}) = 268.8\ \text{kips} \end{aligned} \tag{5.42}$$

The unreduced force in the girder T_i is equal to:

$$\begin{aligned} T_i &= AF_y \\ &= (12.6\ \text{in}^2)(33\ \text{ksi}) = 415.8\ \text{kips} \end{aligned} \tag{5.43}$$

The slab force controls. Therefore, the reduced slab force C is equal to:

$$\begin{aligned} C &= \frac{\sum Q_n}{C_f} C_i \\ &= (0.88)(268.8\ \text{kips}) = 236.5\ \text{kips} \end{aligned} \tag{5.44}$$

The depth of the compressive stress block in the deck a is equal to:

$$\begin{aligned} a &= \frac{C}{0.85 f'_c b_e} \\ &= \frac{236.5\ \text{kips}}{0.85(2.5\ \text{ksi})(23\ \text{in.})} = 4.84\ \text{in.} \end{aligned} \tag{5.45}$$

Equation 10-126 in the AASHTO Standard Specifications gives the compressive force in the steel C' as:

$$\begin{aligned} C' &= \frac{AF_y - C}{2} \\ &= \frac{415.8\ \text{kips} - 236.5\ \text{kips}}{2} = 89.7\ \text{kips} \end{aligned} \tag{5.46}$$

Because $C' = 89.7\ \text{kips} < A_f F_y = 112.9\ \text{kips}$, Equation 10-127 in the AASHTO Standard Specifications gives the neutral axis location y as:

$$\begin{aligned} y &= \frac{C'}{A_f F_y} t_f \\ &= \frac{89.7\ \text{kips}}{(3.421\ \text{in}^2)(33\ \text{ksi})}(0.622\ \text{in.}) = 0.494\ \text{in.} \end{aligned} \tag{5.47}$$

This value is measured down from the top of the girder top flange.

By summing moments of all of the force components (girder in tension, girder in compression, reduced slab in compression) about this neutral axis location, the plastic moment capacity M_p can be found as:

$$M_p = 314.9 \text{ kip–ft}$$

From Equation 10-129a in the AASHTO Standard Specifications, the factor D' is equal to:

$$\begin{aligned} D' &= 0.9\frac{d + t_d}{7.5} \\ &= 0.9\frac{15 + 5.5}{7.5} = 2.46 \text{ in.} \end{aligned} \tag{5.48}$$

The distance from the top of the slab to the plastic neutral axis D_p is:

$$\begin{aligned} D_p &= t_d + y \\ &= 5.5 \text{ in.} + 0.494 \text{ in.} = 5.994 \text{ in.} \end{aligned} \tag{5.49}$$

The equivalent, partial composite section modulus S_{equiv} can be found as:

$$\begin{aligned} S_{equiv} &= \frac{I_{equiv}}{d - y} \\ &= \frac{1272 \text{ in}^4}{15 \text{ in.} - 0.494 \text{ in.}} = 87.7 \text{ in}^3 \end{aligned} \tag{5.50}$$

The elastic moment capacity of the section M_y can also be found as:

$$\begin{aligned} M_y &= S_{equiv} F_y \\ &= (87.7 \text{ in}^3)(33 \text{ ksi}) = 2893.7 \text{ kip–in.} = 241.1 \text{ kip–ft} \end{aligned} \tag{5.51}$$

From Equation 10-129c in the AASHTO Standard Specifications, the nominal moment capacity M_n can be found as:

$$\begin{aligned} M_n &= \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right) \qquad (5.52) \\ &= \frac{(5)(314.9) - (0.85)(241.1)}{4} + \frac{(0.85)(241.1) - 314.9}{4}\left(\frac{5.994}{2.46}\right) \\ M_n &= 275.4 \text{ kip–ft} \end{aligned}$$

This is approximately 97 percent of the fully composite moment capacity, calculated to be 284.6 kip-ft. It is a 68.6 percent increase from the non-composite moment capacity of 163.3 kip-ft.

5.3.4 Structural Analysis for Moment Demand

5.3.4.1 Applied Live Load Moment

Detailed in Section 5.1.4.1, the distributed applied live load moment with dynamic effects M_{LL} on an individual interior girder is:

$$M_{LL} = 102.4 \text{ kip–ft}$$

5.3.4.2 Dead Load Moment of Structural Components

Detailed in Section 5.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 37.7 \text{ kip–ft}$$

5.3.4.3 Superimposed Dead Load Moment

Detailed in Section 5.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 5.0 \text{ kip–ft}$$

5.3.4.4 Total Dead Load Moment

The total applied moment due to structural dead loads and superimposed dead loads M_{DL} is then equal to:

$$M_{DL} = 42.7 \text{ kip–ft}$$

5.3.5 LFR Load Rating for Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{5.53}$$

5.3.5.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$RF_I = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.54}$$

$$= \frac{275.4 - [(1.3)(42.7)]}{(2.17)(102.4)} = 0.99$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.55}$$

$$= \frac{275.4 - [(1.3)(42.7)]}{(1.3)(102.4)} = 1.65$$

Passes.

5.3.5.2 Service Check

For composite sections, per AASHTO Standard Specifications Article 10.57.2 the service capacity C_{serv} is equal to:

$$\begin{aligned} C_{serv} &= 0.95F_y \\ &= (0.95)(33 \text{ ksi}) = 31.35 \text{ ksi} \end{aligned} \tag{5.56}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$\begin{aligned} RF_I &= \frac{C_{serv} - A_1\left(\frac{M_{DC}}{S_x}\right) - A_1\left(\frac{M_{SDL}}{S_{equiv}}\right)}{A_2\left(\frac{M_{LL}}{S_{equiv}}\right)} \\ &= \frac{31.35 - (1.0)\left[\frac{(37.7)(12)}{59.4}\right] - (1.0)\left[\frac{(5.0)(12)}{87.7}\right]}{(1.67)\left[\frac{(102.4)(12)}{87.7}\right]} = 0.99 \end{aligned} \tag{5.57}$$

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$\begin{aligned} RF_O &= \frac{C_{serv} - A_1\left(\frac{M_{DC}}{S_x}\right) - A_1\left(\frac{M_{SDL}}{S_{equiv}}\right)}{A_2\left(\frac{M_{LL}}{S_{equiv}}\right)} \\ &= \frac{31.35 - (1.0)\left[\frac{(37.7)(12)}{59.4}\right] - (1.0)\left[\frac{(5.0)(12)}{87.7}\right]}{(1.0)\left[\frac{(102.4)(12)}{87.7}\right]} = 1.65 \end{aligned} \tag{5.58}$$

Passes.

5.3.6 Controlling Rating Factors

Therefore, the controlling LFR rating factors come from the strength and service check. The controlling rating factors for Bridge SM-5 when considering partial composite action are,

$$RF_I = 0.99$$

$$RF_O = 1.65$$

The basic load rating controlling RFs were 0.49 for Inventory and 0.81 for Operating. The new rating factors represent a 102 percent increase for Inventory and a 104 percent increase for Operating. Table 5.2 compares the controlling RFs determined using a Level II Analysis for partial composite action to the controlling RFs determined in the basic load rating.

Table 5.2. Partial Composite RF Comparison

Rating Factor	Basic Load Rating	Level II Partial Composite Load Rating	Level II Partial Composite/Basic Load Rating
Inventory	0.49	0.99	2.02
Operating	0.81	1.65	2.04

5.4 LOAD RATING ANALYSIS CONSIDERING ONLY END RESTRAINT

This section shows an abbreviated version of a load rating performed for Bridge SM-5, a steel multi-girder bridge, considering interior girder flexure with end restraint. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). In addition, a Level II Analysis (Section 2.4.2) for end restraint is also performed to determine updated live load and dead load moments for the girder being analyzed. This Level II Analysis is based on the results of the load test performed on the bridge in the field (Hueste et al. 2019b). The results of this load rating are compared to the results of the basic load rating analysis.

5.4.1 Bridge Characteristics

Bridge SM-5 is a two-lane bridge with a span length of 40'-2", a deck width of 24'-0", and a roadway width of 23'-6". The girders are braced at third points. The TxDOT HS-20 RFs for this bridge are 0.47 for Inventory and 0.79 for Operating. Figure 5.4 shows a transverse section of Bridge SM-5.

Steel Girder Section: S15x42.9	Span Length: $L = 40.167\ \mathrm{ft}$
Yield Stress: $F_y = 33\ \mathrm{ksi}$	Stringer Spacing: $S = 1.917\ \mathrm{ft}$
Concrete Compressive Strength: $f'_c = 2.5\ \mathrm{ksi}$	Deck Overhang: $S_{overhang} = 0.5\ \mathrm{ft}$
Concrete Density: $\gamma_c = 150\ \mathrm{pcf}$	Asphalt Density: $\gamma_{ws} = 140\ \mathrm{pcf}$
Deck Thickness: $t_d = 6\ \mathrm{in.}$	Asphalt Thickness: $t_{ws} = 1\ \mathrm{in.}$
Assumed Railing Linear Weight: $w_{rail} = 20\ \frac{\mathrm{lb}}{\mathrm{ft}}$	Number of Girders: $N_G = 13$

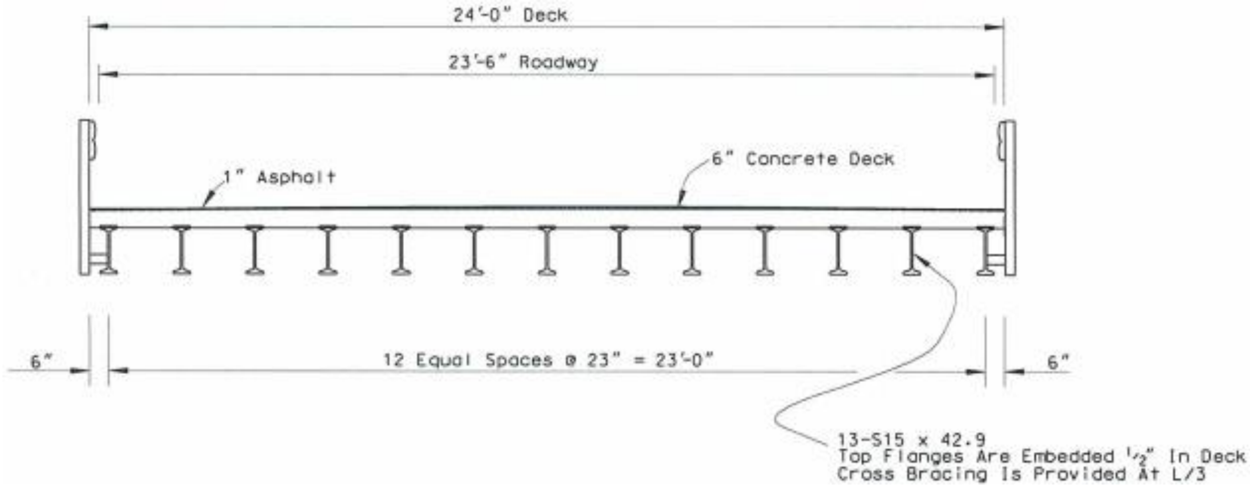

Figure 5.4. Transverse Section of Bridge SM-5 (TxDOT 2018a)

5.4.2 Sectional Properties of Steel Girder

Steel Girder Section: S15x42.9

Area: $A = 12.6 \text{ in}^2$

Major Axis Moment of Inertia: $I_x = 446 \text{ in}^4$

Elastic Section Modulus: $S_x = 59.4 \text{ in}^3$

Plastic Section Modulus: $Z_x = 69.2 \text{ in}^3$

Total Depth: $d = 15 \text{ in.}$

Web Height: $h_w = 13.756 \text{ in.}$

Web Thickness: $t_w = 0.411 \text{ in.}$

Flange Width: $b_f = 5.5 \text{ in.}$

Flange Thickness: $t_f = 0.622 \text{ in.}$

Web Area: $A_w = 5.654 \text{ in}^2$

Flange Area: $A_f = 3.421 \text{ in}^2$

Weak axis radius of gyration: $r_y = 1.06 \text{ in.}$

Web Depth in Compression: $D_c = 6.878 \text{ in.}$

5.4.3 Moment Capacity

Detailed in Section 5.1.3, the moment capacity M_n for an individual girder was determined to be:

$$M_n \quad = \quad 163.3 \text{ kip–ft}$$

5.4.4 Structural Analysis for Moment Demand

5.4.4.1 Applied Live Load Moment considering Simply Supported Boundary Conditions

For LFR, use the LLDF equations provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. Because Bridge SM-5 is being analyzed as a two-lane bridge, for an interior girder

$$\begin{aligned} DF &= \frac{\mathrm{S}}{5.5\ \mathrm{ft}} \\ &= \frac{1.917\ \mathrm{ft}}{5.5\ \mathrm{ft}} = 0.348 \end{aligned} \tag{5.59}$$

The Impact Factor I is applied to the live load effect to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact factor is equal to:

$$\begin{aligned} I &= \frac{50}{L + 125} \leq 0.3 \\ &= \frac{50}{40.167 + 125} \leq 0.3 \\ &= 0.3 \leq 0.3 \\ I &= 0.3 \end{aligned} \tag{5.60}$$

From interpolation in the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1, the applied live load moment M_{HS20} without the application of the impact factor and the LLDF is equal to:

$$M_{HS20} = 226.4\ \frac{\text{kip–ft}}{\text{wheel line}}$$

Therefore, the distributed applied live load moment M_{LL} with dynamic effects, on an individual interior girder considering simply supported boundary conditions can be calculated as:

$$\begin{aligned} M_{LL-simple} &= M_{HS20} DF(1 + I) \\ &= (226.4)(0.348)(1.3) = 102.4\ \text{kip–ft} \end{aligned} \tag{5.61}$$

5.4.4.2 Consideration of End Restraint

From load test results for an interior girder, the maximum compressive strain in the bottom flange during Middle Path loading was measured as 19.4 microstrain ($\mu\varepsilon$). This strain can be converted to a stress (σ) value using Hooke's law:

$$\begin{aligned} \sigma &= \frac{\mu\varepsilon}{10^6}(29{,}000 \text{ ksi}) \\ &= \frac{19.4}{10^6}(29{,}000 \text{ ksi}) = 0.563 \text{ ksi} \end{aligned} \tag{5.62}$$

This stress value can be converted to a moment value, giving the following restraining moment M_{fix} when considering a non-composite girder section:

$$\begin{aligned} M_{end} &= \sigma S_x \\ &= (0.563 \text{ ksi})(59.4 \text{ in}^3) = 33.4 \text{ kip–in.} = 2.8 \text{ kip–ft} \end{aligned} \tag{5.63}$$

Therefore, the new applied midspan live load moment considering end restraint is:

$$\begin{aligned} M_{LL} &= M_{LL-simple} - \frac{(M_{end1} + M_{end2})}{2} \\ &= 102.4 - \frac{2.8 + 2.8}{2} = 99.6 \text{ kip–ft} \end{aligned} \tag{5.64}$$

5.4.4.3 Dead Load Moment of Structural Components

Detailed in Section 5.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 37.7 \text{ kip–ft}$$

5.4.4.4 Superimposed Dead Load Moment

Detailed in Section 5.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 5.0 \text{ kip–ft}$$

5.4.4.5 Total Dead Load Moment

The total applied moment due to structural dead loads and superimposed dead loads M_{DL} is then equal to:

$$M_{DL} = 42.7 \text{ kip–ft}$$

5.4.5 LFR Load Rating for Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{5.65}$$

5.4.5.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$RF_I = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.66}$$

$$= \frac{163.3 - [(1.3)(42.7)]}{(2.17)(99.6)} = 0.50$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.67}$$

$$= \frac{163.3 - [(1.3)(42.7)]}{(1.3)(99.6)} = 0.83$$

Does not pass.

5.4.5.2 Service Check

For non-composite sections, per AASHTO Standard Specifications Article 10.57.1, the service capacity C_{serv} is equal to:

$$\begin{aligned} C_{serv} &= 0.8F_y \\ &= (0.8)(33\text{ ksi}) = 26.4\text{ ksi} \end{aligned} \tag{5.68}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$\begin{aligned} RF_I &= \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \\ &= \frac{26.4 - (1.0)\left[\frac{(42.7)(12)}{59.4}\right]}{(1.67)\left[\frac{(99.6)(12)}{59.4}\right]} = 0.53 \end{aligned} \tag{5.69}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$\begin{aligned} RF_O &= \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \\ &= \frac{26.4 - (1.0)\left[\frac{(42.7)(12)}{59.4}\right]}{(1.0)\left[\frac{(99.6)(12)}{59.4}\right]} = 0.88 \end{aligned} \tag{5.70}$$

Does not pass.

5.4.6 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge SM-5 from the Level II end restraint analysis are equal to:

$$RF_I = 0.50$$

$$RF_O = 0.83$$

The basic load rating controlling RFs were 0.49 for Inventory and 0.81 for Operating. The new rating factors represent a 2.0 percent increase for Inventory and a 2.5 percent increase for Operating. Table 5.3 compares the controlling RFs determined using a Level II end restraint analysis to the controlling RFs determined in the basic load rating.

Table 5.3. End Restraint RF Comparison

Rating Factor	Basic Load Rating	Load Rating with End Restraint	End Restraint/Basic Load Rating
Inventory	0.49	0.50	1.02
Operating	0.81	0.83	1.02

5.5 LOAD RATING ANALYSIS CONSIDERING COMPOSITE ACTION AND END RESTRAINT

This section shows an abbreviated version of a load rating performed for Bridge SM-5, a steel multi-girder bridge, considering both partial composite action and end restraint. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). In addition, a Level III Analysis for partial composite action and end restraint is also performed to determine the updated moment capacity for the girder being analyzed and the applied midspan moment. This Level III Analysis is based on the results of the load test performed on the bridge in the field. The results of this load rating are compared to the results of the basic load rating analysis.

5.5.1 Bridge Characteristics

Bridge SM-5 is a two-lane bridge with a span length of 40'-2", a deck width of 24'-0", and a roadway width of 23'-6". The girders are braced at third points. The TxDOT HS-20 RFs for this bridge are 0.47 for Inventory and 0.79 for Operating. Figure 5.5 shows a transverse section of Bridge SM-5.

Steel Girder Section: S15x42.9

Yield Stress: $F_y = 33 \text{ ksi}$

Concrete Compressive Strength: $f'_c = 2.5 \text{ ksi}$

Concrete Density: $\gamma_c = 150 \text{ pcf}$

Deck Thickness: $t_d = 6 \text{ in.}$

Assumed Railing Linear Weight: $w_{rail} = 20 \ \frac{\text{lb}}{\text{ft}}$

Span Length: $L = 40.167 \text{ ft}$

Stringer Spacing: $S = 1.917 \text{ ft}$

Deck Overhang: $S_{overhang} = 0.5 \text{ ft}$

Asphalt Density: $\gamma_{ws} = 140 \text{ pcf}$

Asphalt Thickness: $t_{ws} = 1 \text{ in.}$

Number of Girders: $N_G = 13$

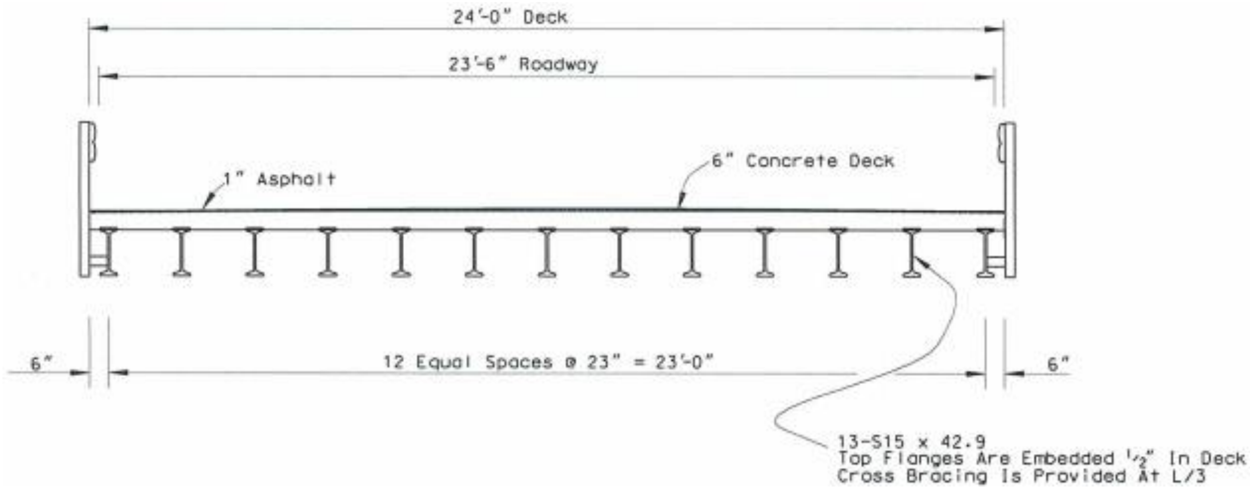

Figure 5.5. Transverse Section of Bridge SM-5 (TxDOT 2018a)

5.5.2 Sectional Properties of Steel Girder

Steel Girder Section: S15x42.9

Area: $A = 12.6 \text{ in}^2$

Major Axis Moment of Inertia: $I_x = 446 \text{ in}^4$

Elastic Section Modulus: $S_x = 59.4 \text{ in}^3$

Plastic Section Modulus: $Z_x = 69.2 \text{ in}^3$

Total Depth: $d = 15 \text{ in.}$

Web Height: $h_w = 13.756 \text{ in.}$

Web Thickness: $t_w = 0.411 \text{ in.}$

Flange Width: $b_f = 5.5 \text{ in.}$

Flange Thickness: $t_f = 0.622 \text{ in.}$

Web Area: $A_w = 5.654 \text{ in}^2$

Flange Area: $A_f = 3.421 \text{ in}^2$

Weak axis radius of gyration: $r_y = 1.06 \text{ in.}$

Web Depth in Compression: $D_c = 6.878 \text{ in.}$

5.5.3 Moment Capacity

5.5.3.1 Initial Moment Capacity Calculation

From Section 5.3.3.1, the acting moment of inertia of the girder I_{equiv} is:

$$I_{equiv} = 1272 \text{ in}^4$$

Also from Section 5.3.3.1, the $\frac{\sum Q_n}{C_f}$ ratio is then:

$$\frac{\sum Q_n}{C_f} = 0.88$$

Use this ratio to reduce the interface shear force in the composite section analysis.

From Section 5.3.3.2, the neutral axis location y as:

$$y = 0.494 \text{ in.}$$

This value is measured down from the top of the girder top flange.

Also from Section 5.3.3.2, the nominal moment capacity M_n is:

$$M_n = 275.4 \text{ kip–ft}$$

5.5.3.2 Consideration of End Restraint for Deflection

Using the restraining moment M_{end} determined in Section 5.4.4.2, Equation (5.63), the amount of upward midspan deflection caused by the end restraint observed during testing Δ_{fixity} can be found as:

$$\Delta_{fixity} = \frac{|M_{end}|L^2}{8EI_{equiv}} \tag{5.71}$$

$$= \frac{\left[|-2.8 \text{ kip–ft}|\left(\frac{12 \text{ in.}}{1 \text{ ft}}\right)\right]\left[(40.17 \text{ ft})\left(\frac{12 \text{ in.}}{1 \text{ ft}}\right)\right]^2}{8(29{,}000 \text{ ksi})(1272 \text{ in}^4)} = 0.026 \text{ in.}$$

Add the midspan deflection due to end restraint from the midspan deflection observed during testing to obtain a new midspan test deflection $\Delta_{test-fixity}$ considering the reduced downward deflection caused by end restraint.

$$\begin{aligned} \Delta_{test-fixity} &= \Delta_{test} + \Delta_{fixity} \\ &= 0.145 \text{ in.} + 0.026 \text{ in.} = 0.171 \text{ in.} \end{aligned} \tag{5.72}$$

This new midspan test deflection $\Delta_{test-fixity}$ is used to update the partial composite action calculations in order to not overestimate the level of composite action.

5.5.3.3 Iteration of Moment Capacity Calculation

Calculate a new moment capacity using the same procedure as laid out in Section 5.3.3.

The theoretical composite moment of inertia of an interior girder and deck I_c can be found as:

$$I_c = 1329 \text{ in}^4$$

The theoretical non-composite moment of inertia of the interior girder I_{nc} is:

$$I_{nc} = 446 \text{ in}^4$$

The theoretical composite deflection Δ_c of an individual interior girder, G7, under Middle Path loading can be found as:

$$\Delta_c = 0.131 \text{ in.}$$

The theoretical non-composite deflection Δ_{nc} of an individual interior girder, G7, under Middle Path loading can be found as:

$$\Delta_{nc} = 0.349 \text{ in.}$$

Therefore, the prorated deflection ratio $\Delta_{prorated}$ is:

$$\begin{aligned} \Delta_{prorated} &= \frac{\Delta_{nc} - \Delta_{test-fixity}}{\Delta_{nc} - \Delta_{c}} \\ &= \frac{0.349 \text{ in.} - 0.171 \text{ in.}}{0.349 \text{ in.} - 0.131 \text{ in.}} = 0.82 \end{aligned} \tag{5.73}$$

Therefore, the acting moment of inertia of the girder I_{equiv} can be approximated as:

$$\begin{aligned} I_{equiv} &= I_{nc} + \Delta_{proprated}(I_c - I_{nc}) \\ &= 446 + 0.82(1329 - 446) = 1170 \text{ in}^4 \end{aligned} \tag{5.74}$$

Substituting known values into Equation C-I3-4 in the 14th edition of the AISC Steel Construction Manual yields:

$$\begin{aligned} I_{equiv} &= I_{nc} + \sqrt{\frac{\sum Q_n}{C_f}}(I_c - I_{nc}) \xrightarrow{yields} \\ 1170 &= 446 + \sqrt{\frac{\sum Q_n}{C_f}}(1329 - 446) \end{aligned} \tag{5.75}$$

Solving for $\frac{\sum Q_n}{C_f}$ yields:

$$\frac{\sum Q_n}{C_f} = 0.82$$

Use this ratio to reduce the controlling force in a composite section analysis.

Assume the slab is only 5.5 in. thick for nominal moment capacity calculations, as the girder flange is embedded 0.5 in. into the 6 in. slab.

The unreduced force in the slab C_i is equal to:

$$\begin{aligned} C_i &= 0.85f'_c b_e t_d \\ &= 0.85(2.5\text{ ksi})(23\text{ in.})(5.5\text{ in.}) = 268.8\text{ kips} \end{aligned} \tag{5.76}$$

The unreduced force in the girder T_i is equal to:

$$\begin{aligned} T_i &= AF_y \\ &= (12.6\text{ in}^2)(33\text{ ksi}) = 415.8\text{ kips} \end{aligned} \tag{5.77}$$

The slab force controls. Therefore, the reduced slab force C is equal to:

$$\begin{aligned} C &= \frac{\sum Q_n}{C_f} C_i \\ &= (0.82)(268.8\text{ kips}) = 220.4\text{ kips} \end{aligned} \tag{5.78}$$

The depth of the compressive stress block in the deck a is equal to:

$$\begin{aligned} a &= \frac{C}{0.85f'_c b_e} \\ &= \frac{220.4\text{ kips}}{0.85(2.5\text{ ksi})(23\text{ in.})} = 4.51\text{ in.} \end{aligned} \tag{5.79}$$

Equation 10-126 in the AASHTO Standard Specifications gives the compressive force in the steel C' as:

$$\begin{aligned} C' &= \frac{AF_y - C}{2} \\ &= \frac{415.8\text{ kips} - 220.4\text{ kips}}{2} = 97.7\text{ kips} \end{aligned} \tag{5.80}$$

Since $C' = 97.7\text{ kips} < A_f F_y = 112.9\text{ kips}$, Equation 10-127 in the AASHTO Standard Specifications gives the neutral axis location y as:

$$\begin{aligned} y &= \frac{C'}{A_f F_y} t_f \\ &= \frac{97.7\text{ kips}}{(3.421\text{ in}^2)(33\text{ ksi})}(0.622\text{ in.}) = 0.538\text{ in.} \end{aligned} \tag{5.81}$$

This value is measured down from the top of the girder top flange.

By summing moments of all of the force components (girder in tension, girder in compression, reduced slab in compression) about this neutral axis location, the plastic moment capacity M_p can be found as:

$$M_p = 313.1\text{ kip–ft}$$

From Equation 10-129a in the AASHTO Standard Specifications, the factor D' is equal to:

$$\begin{aligned} D' &= 0.9\frac{d + t_d}{7.5} \\ &= 0.9\frac{15 + 5.5}{7.5} = 2.46\text{ in.} \end{aligned} \tag{5.82}$$

The distance from the top of the slab to the plastic neutral axis D_p is:

$$\begin{aligned} D_p &= t_d + y \\ &= 5.5\text{ in.} + 0.538\text{ in.} = 6.038\text{ in.} \end{aligned} \tag{5.83}$$

The equivalent, partial composite section modulus S_{equiv} can be found as:

$$S_{equiv} = \frac{I_{equiv}}{d-y} \tag{5.84}$$

$$= \frac{1272 \text{ in}^4}{15 \text{ in.} - 0.538 \text{ in.}} = 88.0 \text{ in}^3$$

The elastic moment capacity of the section M_y can also be found as:

$$M_y = S_{equiv} F_y \tag{5.85}$$

$$= (88.0 \text{ in}^3)(33 \text{ ksi}) = 2904.0 \text{ kip–in.} = 242.0 \text{ kip–ft}$$

From Equation 10-129c in the AASHTO Standard Specifications, the nominal moment capacity M_n can be found as:

$$M_n = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right) \tag{5.86}$$

$$= \frac{(5)(313.1) - (0.85)(242.0)}{4} + \frac{(0.85)(242.0) - 313.1}{4}\left(\frac{6.038}{2.46}\right)$$

$$M_n = 274.0 \text{ kip–ft}$$

This is approximately 96 percent of the fully composite moment capacity, calculated to be 284.6 kip-ft. It is a 67.8 percent increase from the non-composite moment capacity of 163.3 kip-ft. It is a 0.5 percent decrease from the moment capacity of 275.4 kip-ft found considering only partial composite action.

5.5.4 Structural Analysis for Moment Demand

5.5.4.1 Applied Live Load Moment considering Simply Supported Boundary Conditions

From Section 5.4.4.1, the applied midspan live load moment considering end restraint is:

$$M_{LL} = 99.6 \text{ kip–ft}$$

5.5.4.2 Dead Load Moment of Structural Components

From Section 5.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 37.7 \text{ kip–ft}$$

5.5.4.3 Superimposed Dead Load Moment

From Section 5.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 5.0 \text{ kip–ft}$$

5.5.4.4 Total Dead Load Moment

The total applied moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$M_{DL} = 42.7 \text{ kip–ft}$$

5.5.5 LFR Load Rating for Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{5.87}$$

5.5.5.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$RF_I = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.88}$$

$$= \frac{274.0 - [(1.3)(42.7)]}{(2.17)(99.6)} = 1.01$$

Passes.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{5.89}$$

$$= \frac{274.0 - [(1.3)(42.7)]}{(1.3)(99.6)} = 1.69$$

Passes.

5.5.5.2 Service Check

For composite sections, per AASHTO Standard Specifications Article 10.57.2 the service capacity C_{serv} is equal to:

$$C_{serv} = 0.95 F_y \tag{5.90}$$

$$= (0.95)(33 \text{ ksi}) = 31.35 \text{ ksi}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$RF_I = \frac{C_{serv} - A_1\left(\frac{M_{DC}}{S_x}\right) - A_1\left(\frac{M_{SDL}}{S_{equiv}}\right)}{A_2\left(\frac{M_{LL}}{S_{equiv}}\right)} \tag{5.91}$$

$$= \frac{31.35 - (1.0)\left[\frac{(37.7)(12)}{59.4}\right] - (1.0)\left[\frac{(5.0)(12)}{88.0}\right]}{(1.67)\left[\frac{(99.6)(12)}{88.0}\right]} = 1.02$$

Passes.

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$RF_O = \frac{C_{serv} - A_1\left(\frac{M_{DC}}{S_x}\right) - A_1\left(\frac{M_{SDL}}{S_{equiv}}\right)}{A_2\left(\frac{M_{LL}}{S_{equiv}}\right)} \tag{5.92}$$

$$= \frac{31.35 - (1.0)\left[\frac{(37.7)(12)}{59.4}\right] - (1.0)\left[\frac{(5.0)(12)}{88.0}\right]}{(1.0)\left[\frac{(99.6)(12)}{88.0}\right]} = 1.70$$

Passes.

5.5.6 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge SM-5 from the Level III partial composite action analysis are equal to:

$$RF_I = 1.01$$

$$RF_O = 1.69$$

The basic load rating controlling RFs were 0.49 for Inventory and 0.81 for Operating. The new rating factors represent a 106 percent increase for Inventory and a 109 percent increase for Operating. Table 5.4 compares the controlling RFs determined using a Level III Analysis for partial composite action and end restraint to the controlling RFs determined in the basic load rating.

Table 5.4. Level III Partial Composite Action Considering End Restraint RF Comparison

Rating Factor	Basic Load Rating	Level III Load Rating	Level III/Basic Load Rating
Inventory	0.49	1.01	2.06
Operating	0.81	1.69	2.09

6 LOAD RATING EXAMPLE: CONTINUOUS STEEL MULTI-GIRDER BRIDGE USING LFR METHOD

6.1 BASIC LOAD RATING ANALYSIS ASSUMING NON-COMPOSITE GIRDERS

This section shows an abbreviated version of the initial basic load rating performed for Bridge SC-12, a three-span continuous steel multi-girder bridge, considering interior girder flexure. This basic load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). The initial load rating is performed with the assumption that the bridge is acting without composite action at the girder-to-deck interface and the support conditions are all roller-supported. This initial load rating is used for comparison when conducting load ratings assuming partial composite action.

6.1.1 Bridge Characteristics

Bridge SC-12 is a two-lane, three-span continuous bridge with span lengths of 60'-75'-60', a deck width of 25'-6", and a roadway width of 24'-0". The girders are braced at quarter points. It also has a 9 x 3/8 in. cover plate on the top and bottom flange that is 10'-0"long centered over both interior supports. The TxDOT HS-20 RFs for it are 0.55 for Inventory and 0.92 for Operating. Figure 6.1 shows a transverse section of Bridge SC-12. The main bridge characteristics needed for load rating are summarized below.

Steel Girder Section: W30x108

Yield Stress: $F_y = 33\text{ ksi}$

Concrete Compressive Strength: $f'_c = 2.5\text{ ksi}$

Concrete Density: $\gamma_c = 150\text{ pcf}$

Deck Thickness: $t_d = 6\text{ in.}$

Assumed Railing Linear Weight: $w_{rail} = 20\ \frac{\text{lb}}{\text{ft}}$

Main Span Length: $L = 75\text{ ft}$

Stringer Spacing: $S = 6.67\text{ ft}$

Deck Overhang: $S_{overhang} = 2.75\text{ ft}$

Asphalt Density: $\gamma_{ws} = 140\text{ pcf}$

Asphalt Thickness: $t_{ws} = 2\text{ in.}$

Number of Girders: $N_G = 4$

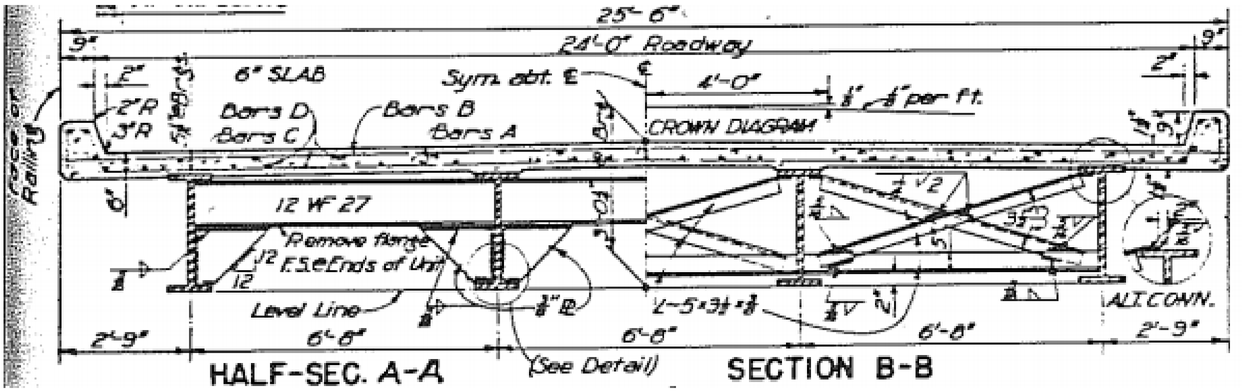

Figure 6.1. Transverse Section of Bridge SC-12 (TxDOT 2018a)

6.1.2 Sectional Properties of Steel Girder

Steel Girder Section: W30x108

Area: $A = 31.7 \text{ in}^2$

Major Axis Moment of Inertia: $I_x = 4470 \text{ in}^4$

Elastic Section Modulus: $S_x = 299 \text{ in}^3$

Plastic Section Modulus: $Z_x = 346 \text{ in}^3$

Total Depth: $d = 29.8 \text{ in.}$

Web Height: $h_w = 28.28 \text{ in.}$

Web Thickness: $t_w = 0.545 \text{ in.}$

Flange Width: $b_f = 10.5 \text{ in.}$

Flange Thickness: $t_f = 0.76 \text{ in.}$

Web Area: $A_w = 15.413 \text{ in}^2$

Flange Area: $A_f = 7.98 \text{ in}^2$

Weak axis radius of gyration: $r_y = 2.15 \text{ in.}$

Web Depth in Compression: $D_c = 14.14 \text{ in.}$

6.1.3 Moment Capacity in the Positive Moment Region

To determine the positive moment region capacity for an individual girder, the procedure laid out in Section 10.48 of the AASHTO Standard Specifications (AASHTO 2002) is followed. First, a compact section check is performed. This involves checking the compression flange and web for compactness, and the braced length against a limit. If the girder does not pass one of these checks, a braced noncompact section check is performed. This involves the same checks as before, with different limit values. Finally, if the girder does not pass the lateral bracing check, it is analyzed as a partially braced member.

6.1.3.1 Compact Section Check

Check if section is compact – AASHTO Standard Specifications Article 10.48.1.1.

a) Check compression flange

$$\frac{b_f}{t_f} \leq \frac{4110}{\sqrt{F_y}} \tag{6.1}$$

$$\frac{10.5 \text{ in.}}{0.76 \text{ in.}} \leq \frac{4110}{\sqrt{33{,}000}}$$

$$13.8 \leq 22.6$$

Compression flange is compact (OK).

b) Check web thickness

$$\frac{h_w}{t_w} \leq \frac{19230}{\sqrt{F_y}} \tag{6.2}$$

$$\frac{28.28 \text{ in.}}{0.545 \text{ in.}} \leq \frac{19{,}230}{\sqrt{33{,}000}}$$

$$51.9 \leq 105.9$$

Web is compact (OK).

c) Check spacing of lateral bracing for compression flange (moments are obtained from applied moment analysis later)

$$\frac{L_b}{r_y} \leq \frac{\left(3.6 - 2.2\left(\frac{M_1}{M_u}\right)\right)(10^6)}{F_y} \tag{6.3}$$

$$\frac{225 \text{ in.}}{2.15 \text{ in.}} \leq \frac{\left(3.6 - 2.2\left(\frac{1125}{952}\right)\right)(10^6)}{33{,}000}$$

$$104.7 \geq 42.4 \quad \text{(No Good)}$$

Where M_1 is the smaller moment at the end of the unbraced length of the member and M_u is equal to the plastic moment capacity.

Spacing of lateral bracing for compression flange is NOT OK.

6.1.3.2 Braced Non-Compact Section Check

Check if braced non-compact section – AASHTO Standard Specifications Article 10.48.2.

a) Check compression flange

$$\frac{b_f}{t_f} \leq 24 \tag{6.4}$$

$$\frac{10.5\text{ in.}}{0.76\text{ in.}} \leq 24$$

$$13.8 \leq 24$$

Compression flange is OK.

b) Check web thickness. Web thickness is OK per compact section check.

c) Check spacing of lateral bracing for compression flange

$$L_b \leq \frac{20{,}000{,}000A_f}{F_y d} \tag{6.5}$$

$$225\text{ in.} \leq \frac{(20{,}000{,}000)(7.98)}{(33{,}000)(29.8)}$$

$$225\text{ in.} \geq 162.3\text{ in.} \quad \text{No Good.}$$

Spacing of lateral bracing for compression flange is NOT OK.

6.1.3.3 Partially Braced Section Analysis

Analyze as partially braced section – AASHTO Standard Specifications Article 10.48.4.

The factor λ used in partially braced member moment capacity calculations can be taken as 15,400.

$$\lambda = 15400$$

The bending coefficient C_b can conservatively be taken as 1.0.

$$C_b = 1.0$$

The moment of inertia of the compression flange about the vertical axis I_{yc} is calculated as:

$$\begin{aligned} I_{yc} &= \frac{1}{12} t_f b_f^3 \qquad (6.6) \\ &= \left(\frac{1}{12}\right)(0.76\text{ in.})(10.5\text{ in.})^3 = 73.3\text{ in}^4 \end{aligned}$$

The radius of gyration of the compression flange about the vertical axis r_y' can be calculated as:

$$\begin{aligned} r'_y &= \sqrt{\frac{I_{yc}}{A_f}} \qquad (6.7) \\ &= \sqrt{\frac{73.3\text{ in}^4}{7.98\text{ in}^2}} = 3.03\text{ in.} \end{aligned}$$

The torsional property J may be computed as:

$$\begin{aligned} J &= \frac{2\left(b_f t_f^3\right) + \left(h_w t_w^3\right)}{3} \qquad (6.8) \\ &= \frac{2[(10.5)(0.76^3)] + [(28.28)(0.545^3)]}{3} = 4.60\text{ in}^4 \end{aligned}$$

Check,

$$\begin{aligned} \frac{D_c}{t_w} &\leq \frac{\lambda}{\sqrt{F_y}} \qquad (6.9) \\ \frac{14.14\text{ in.}}{0.545\text{ in.}} &\leq \frac{15400}{\sqrt{33{,}000}} \\ 25.9 &\leq 84.8 \end{aligned}$$

Check is OK. Therefore, flexural resistance M_r may be calculated as:

$$M_r = (91)(10^6)C_b\left(\frac{I_{yc}}{L_b}\right)\sqrt{\frac{0.722J}{I_{yc}} + 9.87\left(\frac{d}{L_b}\right)^2} \leq F_y S_x \tag{6.10}$$

$$= (91)(10^6)(1.0)\left(\frac{73.3}{225}\right)\sqrt{\frac{(0.722)(4.60)}{73.3} + 9.87\left(\frac{29.8}{225}\right)^2} \leq (33\text{ ksi})(299\text{ in}^3)$$

$$= 13856\text{ kip–in.} \leq 9867\text{ kip–in.} = 9867\text{ kip–in.}$$

$$M_r = 822.3\text{ kip–ft}$$

Calculate the bending capacity reduction factor R_b as:

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_f}\right)\left(\frac{D_c}{t_w} - \frac{\lambda}{\sqrt{\frac{M_r}{S_x}}}\right) \leq 1.0 \tag{6.11}$$

$$= 1 - 0.002\left[\frac{(14.14)(0.545)}{7.98}\right]\left(\frac{14.14}{0.545} - \frac{15400}{\sqrt{\frac{9867000}{299}}}\right) \leq 1.0$$

$$= 1.11 \leq 1.0$$

$$R_b = 1.0$$

Therefore, the moment capacity M_n in the positive moment region is equal to:

$$M_n = R_b M_r \tag{6.12}$$

$$= (1.0)(822.3\text{ kip–ft}) = 822.3\text{ kip–ft}$$

6.1.4 Moment Capacity in the Negative Moment Region

To determine the negative moment region capacity for an individual girder, the procedure laid out in Section 10.48 of the AASHTO Standard Specifications (AASHTO 2002) is followed. However, new section properties must first be calculated considering the 9 x 3/8 in. top and bottom cover plates. Calculation of the new section properties leads to:

Area: $A = 38.45 \text{ in}^2$

Major Axis Moment of Inertia: $I_x = 6007 \text{ in}^4$

Elastic Section Modulus: $S_x = 393 \text{ in}^3$

Plastic Section Modulus: $Z_x = 443 \text{ in}^3$

Total Depth: $d = 30.55 \text{ in.}$

Web Height: $h_w = 28.28 \text{ in.}$

Web Thickness: $t_w = 0.545 \text{ in.}$

Flange Width: $b_f = 10.5 \text{ in.}$

Flange Thickness: $t_f = 1.135 \text{ in.}$

Web Area: $A_w = 15.413 \text{ in}^2$

Flange Area: $A_f = 11.355 \text{ in}^2$

Weak axis radius of gyration: $r_y = 2.23 \text{ in.}$

Web Depth in Compression: $D_c = 14.14 \text{ in.}$

Weak Axis Moment of Inertia: $I_y = 192 \text{ in}^4$

6.1.4.1 Compact Section Check

Check if compact section – AASHTO Standard Specifications Article 10.48.1.1.

a) Check compression flange

$$\frac{b_f}{t_f} \leq \frac{4110}{\sqrt{F_y}} \tag{6.13}$$

$$\frac{10.5 \text{ in.}}{1.135 \text{ in.}} \leq \frac{4110}{\sqrt{33{,}000}}$$

$$9.3 \leq 22.6$$

Compression flange is OK.

b) Check web thickness

$$\frac{h_w}{t_w} \leq \frac{19230}{\sqrt{F_y}} \tag{6.14}$$

$$\frac{28.28 \text{ in.}}{0.545 \text{ in.}} \leq \frac{19{,}230}{\sqrt{33{,}000}}$$

$$51.9 \leq 105.9$$

Web thickness is OK.

c) Check spacing of lateral bracing for compression flange (moments are obtained from applied moment analysis later).

$$\frac{L_b}{r_y} \leq \frac{\left(3.6 - 2.2\left(\frac{M_1}{M_u}\right)\right)(10^6)}{F_y} \tag{6.15}$$

$$\frac{225 \text{ in.}}{2.23 \text{ in.}} \leq \frac{\left(3.6 - 2.2\left(\frac{436}{1218}\right)\right)(10^6)}{33{,}000}$$

$$100.9 \leq 85.2$$

Where M_1 is the smaller moment at the end of the unbraced length of the member and M_u is equal to the plastic moment capacity.

Spacing of lateral bracing for compression flange is NOT OK.

6.1.4.2 Braced Non-Compact Section Check

Check if braced non-compact section – AASHTO Standard Specifications Article 10.48.2.

a) Check compression flange

$$\frac{b_f}{t_f} \leq 24 \tag{6.16}$$

$$\frac{10.5 \text{ in.}}{1.135 \text{ in.}} \leq 24$$

$$9.3 \leq 24$$

Compression flange is OK.

b) Check web thickness. Web thickness is OK per compact section check.

c) Check spacing of lateral bracing for compression flange

$$L_b \leq \frac{20{,}000{,}000 A_f}{F_y d} \tag{6.17}$$

$$225 \text{ in.} \leq \frac{(20{,}000{,}000)(11.35)}{(33{,}000)(30.55)}$$

$$225 \text{ in.} \leq 225.2 \text{ in.}$$

Spacing of lateral bracing for compression flange is NOT OK.

6.1.4.3 Partially Braced Section Analysis

Analyze as partially braced section – AASHTO Standard Specifications Article 10.48.4.

The factor λ used in partially braced member moment capacity calculations can be taken as 15,400.

$$\lambda = 15400$$

The bending coefficient C_b can conservatively be taken as 1.0.

$$C_b = 1.0$$

The moment of inertia of the compression flange about the vertical axis I_{yc} is calculated as:

$$I_{yc} = \frac{1}{12} t_f b_f^3 \tag{6.18}$$

$$= \left(\frac{1}{12}\right)(1.135 \text{ in.})(10.5 \text{ in.})^3 = 109.5 \text{ in}^4$$

The radius of gyration of the compression flange about the vertical axis r_y' can be calculated as:

$$r'_y = \sqrt{\frac{I_{yc}}{A_f}} \tag{6.19}$$

$$= \sqrt{\frac{109.5 \text{ in}^4}{11.355 \text{ in}^2}} = 9.64 \text{ in.}$$

The torsional property J may be computed as:

$$J = \frac{2\left(b_f t_f^3\right) + (h_w t_w^3)}{3} \tag{6.20}$$

$$= \frac{2[(10.5)(1.135^3)] + [(28.28)(0.545^3)]}{3} = 11.76 \text{ in}^4$$

Check,

$$\frac{D_c}{t_w} \le \frac{\lambda}{\sqrt{F_y}} \tag{6.21}$$

$$\frac{14.14 \text{ in.}}{0.545 \text{ in.}} \le \frac{15400}{\sqrt{33{,}000}}$$

$$25.9 \le 84.8$$

Check is OK. Therefore, flexural resistance M_r may be calculated as:

$$M_r = (91)(10^6)C_b\left(\frac{I_{yc}}{L_b}\right)\sqrt{\frac{0.722J}{I_{yc}} + 9.87\left(\frac{d}{L_b}\right)^2} \le F_y S_x \tag{6.22}$$

$$= (91)(10^6)(1.0)\left(\frac{109.5}{225}\right)\sqrt{\frac{(0.722)(11.76)}{109.5} + 9.87\left(\frac{30.55}{225}\right)^2} \le (33 \text{ ksi})(393 \text{ in}^3)$$

$$= 22560 \text{ kip–in.} \le 12969 \text{ kip–in.} = 12969 \text{ kip–in.}$$

$$M_r = 1081 \text{ kip–ft}$$

Calculate the bending capacity reduction factor R_b as:

$$R_b = 1 - 0.002\left(\frac{D_c t_w}{A_f}\right)\left(\frac{D_c}{t_w} - \frac{\lambda}{\sqrt{\frac{M_r}{S_x}}}\right) \le 1.0 \tag{6.23}$$

$$= 1 - 0.002\left[\frac{(14.14)(0.545)}{11.355}\right]\left(\frac{14.14}{0.545} - \frac{15400}{\sqrt{\frac{12{,}969{,}000}{393}}}\right) \le 1.0$$

$$= 1.08 \le 1.0$$

$$R_b = 1.0$$

Therefore, the moment capacity M_n in the positive moment region is equal to:

$$\begin{aligned} M_n &= R_b M_r \\ &= (1.0)(1081 \text{ kip–ft}) = 1081 \text{ kip–ft} \end{aligned} \tag{6.24}$$

6.1.5 Structural Analysis for Moment Demand in the Positive Moment Region

6.1.5.1 Applied Positive Live Load Moment

For LFR, use the LLDF equations provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. As Bridge SC-12 is a two-lane bridge, for an interior girder

$$\begin{aligned} DF &= \frac{\text{S}}{5.5 \text{ ft}} \\ &= \frac{6.67 \text{ ft}}{5.5 \text{ ft}} = 1.212 \end{aligned} \tag{6.25}$$

The Impact Factor I is applied to the live load effect to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to:

$$\begin{aligned} I &= \frac{50}{L + 125} \leq 0.3 \\ &= \frac{50}{75 + 125} \leq 0.3 \\ &= 0.25 \leq 0.3 \\ I &= 0.25 \end{aligned} \tag{6.26}$$

From structural analysis representing the interior bridge girder as a three-span continuous beam, the applied live load moment M_{HS20} without the application of the Impact Factor and the LLDF is equal to:

$$M_{HS20} = 332 \, \frac{\text{kip–ft}}{\text{wheel line}}$$

Therefore, the distributed applied live load positive moment M_{LL} with dynamic effects, on an individual interior girder can be calculated as:

$$\begin{aligned} M_{LL} &= M_{HS20} DF(1+I) \\ &= (332)(1.212)(1.25) = 503 \text{ kip–ft} \end{aligned} \tag{6.27}$$

6.1.5.2 Dead Load Positive Moment of Structural Components

The girder distributed weight w_G for a W30x108 section is equal to:

$$w_G = 0.108\ \frac{\text{kip}}{\text{ft}}$$

The distributed load from the deck w_d on an individual girder can be calculated as:

$$\begin{aligned} w_d &= \gamma_c t_d S \\ &= (150\text{ pcf})\left(\frac{6\text{ in.}}{12\,\frac{\text{in.}}{\text{ft}}}\right)(6.67\text{ ft}) = 500\ \frac{\text{lb}}{\text{ft}} = 0.5\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{6.28}$$

The curbs are integral and are therefore included in the dead load of structural components section. Distributing the curbs evenly to all girders, the distributed load from the curbs w_{curb} on an individual girder can be found as:

$$\begin{aligned} w_{curb} &= \gamma_c A_{curb}\frac{2}{N_G} \\ &= (150\text{ pcf})\left(\frac{90\text{ in}^2}{144\,\frac{\text{in}^2}{\text{ft}^2}}\right)\left(\frac{2}{4}\right) = 46.9\ \frac{\text{lb}}{\text{ft}} = 0.047\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{6.29}$$

Therefore, the total distributed load due to dead load of structural components w_{DC} is equal to:

$$\begin{aligned} w_{DC} &= w_G + w_d + w_{curb} \\ &= \left(0.043\ \frac{\text{kip}}{\text{ft}}\right) + \left(0.144\frac{\text{kip}}{\text{ft}}\right) + \left(0.047\frac{\text{kip}}{\text{ft}}\right) = 0.655\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{6.30}$$

From structural analysis representing the interior bridge girder as a three-span continuous beam, the applied positive moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 140.8\text{ kip–ft}$$

6.1.5.3 Superimposed Dead Load Positive Moment

The superimposed dead load on an individual girder due to the railing w_{rail} can be calculated as:

$$\begin{aligned} w_{rail} &= w_{rail}\frac{2}{N_G} \\ &= \left(20\ \frac{\text{lb}}{\text{ft}}\right)\left(\frac{2}{4}\right) = 10\ \frac{\text{lb}}{\text{ft}} = 0.010\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{6.31}$$

The superimposed dead load on an individual girder due to the wearing surface w_{ws} can be calculated as:

$$\begin{aligned} w_{ws} &= \gamma_{ws} t_{ws} S \\ &= (140\ \text{pcf})\left(\frac{1\ \text{in.}}{12\frac{\text{in.}}{\text{ft}}}\right)(6.67\ \text{ft}) = 77.8\ \frac{\text{lb}}{\text{ft}} = 0.078\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{6.32}$$

Therefore, the total distributed load due to superimposed dead load w_{SDL} is equal to:

$$\begin{aligned} w_{SDL} &= w_{rail} + w_{ws} \\ &= \left(0.010\ \frac{\text{kip}}{\text{ft}}\right) + \left(0.078\frac{\text{kip}}{\text{ft}}\right) = 0.088\frac{\text{kip}}{\text{ft}} \end{aligned} \tag{6.33}$$

From structural analysis representing the interior bridge girder as a three-span continuous beam, the applied positive moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 36.2\ \text{kip–ft}$$

6.1.5.4 Total Dead Load Positive Moment

Therefore, the total applied positive moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$\begin{aligned} M_{DL} &= M_{DC} + M_{SDL} \\ &= (140.8\ \text{kip–ft}) + (36.2\ \text{kip–ft}) = 177\ \text{kip–ft} \end{aligned} \tag{6.34}$$

6.1.6 Structural Analysis for Moment Demand in the Negative Moment Region

6.1.6.1 Applied Negative Live Load Moment

For LFR, use the LLDF equations provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. As Bridge SC-12 is a two-lane bridge, for an interior girder

$$\begin{aligned} DF &= \frac{\mathrm{S}}{5.5\ \mathrm{ft}} \\ &= \frac{6.67\ \mathrm{ft}}{5.5\ \mathrm{ft}} = 1.212 \end{aligned} \tag{6.35}$$

The Impact Factor I is applied to the live load effect to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to:

$$\begin{aligned} I &= \frac{50}{L+125} \leq 0.3 \\ &= \frac{50}{75+125} \leq 0.3 \\ &= 0.25 \leq 0.3 \\ I &= 0.25 \end{aligned} \tag{6.36}$$

From structural analysis representing the interior bridge girder as a three-span continuous beam, the applied live load negative moment M_{HS20} without the application of the Impact Factor and the LLDF is equal to:

$$M_{HS20} = 218\ \frac{\mathrm{kip{-}ft}}{\mathrm{wheel\ line}}$$

Therefore, the distributed applied live load negative moment M_{LL} with dynamic effects, on an individual interior girder can be calculated as:

$$\begin{aligned} M_{LL} &= M_{HS20} DF(1+I) \\ &= (218)(1.212)(1.25) = 331\ \mathrm{kip{-}ft} \end{aligned} \tag{6.37}$$

6.1.6.2 Dead Load Negative Moment of Structural Components

From Section 6.1.5.2, the total distributed load due to dead load of structural components w_{DC} is equal to:

$$w_{DC} = 0.655 \frac{\text{kip}}{\text{ft}}$$

From structural analysis representing the interior bridge girder as a three-span continuous beam, the applied negative moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 303 \text{ kip–ft}$$

6.1.6.3 Superimposed Dead Load Negative Moment

From Section 6.1.5.3, the total distributed load due to superimposed dead load w_{SDL} is equal to:

$$w_{SDL} = 0.088 \frac{\text{kip}}{\text{ft}}$$

From structural analysis representing the interior bridge girder as a three-span continuous beam, the applied negative moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 41 \text{ kip–ft}$$

6.1.6.4 Total Dead Load Negative Moment

Therefore, the total applied negative moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$\begin{aligned} M_{DL} &= M_{DC} + M_{SDL} \\ &= (303 \text{ kip–ft}) + (41 \text{ kip–ft}) = 344 \text{ kip–ft} \end{aligned} \tag{6.38}$$

6.1.7 LFR Load Rating for Positive Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{6.39}$$

6.1.7.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \qquad (6.40) \\ &= \frac{822.3 - [(1.3)(177.0)]}{(2.17)(503.0)} = 0.54 \end{aligned}$$

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$\begin{aligned} RF_O &= \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \qquad (6.41) \\ &= \frac{822.3 - [(1.3)(177.0)]}{(1.3)(503.0)} = 0.91 \end{aligned}$$

6.1.7.2 Service Check

For non-composite sections, per AASHTO Standard Specifications Article 10.57.1 the service capacity C_{serv} is equal to:

$$C_{serv} = 0.8F_y \tag{6.42}$$
$$= (0.8)(33\text{ ksi}) = 26.4\text{ ksi}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$RF_I = \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \tag{6.43}$$
$$= \frac{26.4 - (1.0)\left[\frac{(177.0)(12)}{299.0}\right]}{(1.67)\left[\frac{(503.0)(12)}{299.0}\right]} = 0.57$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$RF_O = \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \tag{6.44}$$
$$= \frac{26.4 - (1.0)\left[\frac{(177.0)(12)}{299.0}\right]}{(1.0)\left[\frac{(503.0)(12)}{299.0}\right]} = 0.96$$

Does not pass.

6.1.8 LFR Load Rating for Negative Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{6.45}$$

6.1.8.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{1081.0 - [(1.3)(344.0)]}{(2.17)(331.0)} = 0.88 \end{aligned} \tag{6.46}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$\begin{aligned} RF_O &= \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{1081.0 - [(1.3)(344.0)]}{(1.3)(331.0)} = 1.47 \end{aligned} \tag{6.47}$$

Passes.

6.1.8.2 Service Check

For non-composite sections, per AASHTO Standard Specifications Article 10.57.1 the service capacity C_{serv} is equal to:

$$\begin{aligned} C_{serv} &= 0.8F_y \\ &= (0.8)(33\text{ ksi}) = 26.4\text{ ksi} \end{aligned} \tag{6.48}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$\begin{aligned} RF_I &= \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \\ &= \frac{26.4 - (1.0)\left[\frac{(344.0)(12)}{393.0}\right]}{(1.67)\left[\frac{(331.0)(12)}{393.0}\right]} = 0.94 \end{aligned} \tag{6.49}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$\begin{aligned} RF_O &= \frac{C_{serv} - A_1\left(\frac{M_{DL}}{S_x}\right)}{A_2\left(\frac{M_{LL}}{S_x}\right)} \\ &= \frac{26.4 - (1.0)\left[\frac{(344.0)(12)}{393.0}\right]}{(1.0)\left[\frac{(331.0)(12)}{393.0}\right]} = 1.57 \end{aligned} \tag{6.50}$$

Passes.

6.1.9 Controlling Rating Factors

The controlling LFR rating factors come from the strength check in the positive moment region. The controlling rating factors for Bridge SC-12 from the basic load rating analysis are equal to:

$$RF_I = 0.54$$

$$RF_O = 0.91$$

6.2 LOAD RATING CONSIDERING ONLY PARTIAL COMPOSITE ACTION

This section shows an abbreviated version of a load rating analysis performed for Bridge SC-12, a three-span continuous steel multi-girder bridge, considering interior girder flexure. In addition, partial composite action observed during load testing is considered in the load rating. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). However, a Level II Analysis for partial composite action is also performed to determine a new moment capacity for the girder being analyzed. This Level II Analysis is based on the results of the load test performed on the bridge in the field. The results of this load rating are compared to the results of the basic load rating analysis.

6.2.1 Bridge Characteristics

Bridge SC-12 is a two-lane, three-span continuous bridge with span lengths of 60'-75'-60', a deck width of 25'-6", and a roadway width of 24'-0". The girders are braced at quarter points. It also has a 9 x 3/8 in. cover plate on the top and bottom flange that is 10'-0" long centered over both interior supports. The TxDOT HS-20 RFs for it are 0.55 for Inventory and 0.92 for Operating. Figure 6.2 shows a transverse section of Bridge SC-12.

Steel Girder Section: W30x108

Yield Stress: $F_y = 33 \text{ ksi}$

Concrete Compressive Strength: $f'_c = 2.5 \text{ ksi}$

Concrete Density: $\gamma_c = 150 \text{ pcf}$

Deck Thickness: $t_d = 6 \text{ in.}$

Assumed Railing Linear Weight: $w_{rail} = 20 \ \frac{\text{lb}}{\text{ft}}$

Main Span Length: $L = 75 \text{ ft}$

Stringer Spacing: $S = 6.67 \text{ ft}$

Deck Overhang: $S_{overhang} = 2.75 \text{ ft}$

Asphalt Density: $\gamma_{ws} = 140 \text{ pcf}$

Asphalt Thickness: $t_{ws} = 2 \text{ in.}$

Number of Girders: $N_G = 4$

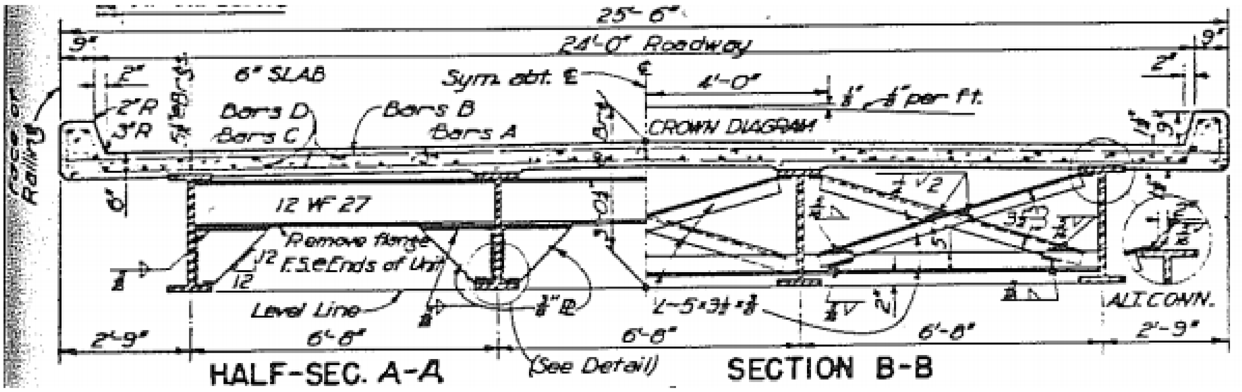

Figure 6.2. Transverse Section of Bridge SC-12 (TxDOT 2018a)

6.2.2 Sectional Properties of Steel Girder

Steel Girder Section: W30x108

Area: $A = 31.7 \text{ in}^2$

Major Axis Moment of Inertia: $I_x = 4470 \text{ in}^4$

Elastic Section Modulus: $S_x = 299 \text{ in}^3$

Plastic Section Modulus: $Z_x = 346 \text{ in}^3$

Total Depth: $d = 29.8 \text{ in.}$

Web Height: $h_w = 28.28 \text{ in.}$

Web Thickness: $t_w = 0.545 \text{ in.}$

Flange Width: $b_f = 10.5 \text{ in.}$

Flange Thickness: $t_f = 0.76 \text{ in.}$

Web Area: $A_w = 15.413 \text{ in}^2$

Flange Area: $A_f = 7.98 \text{ in}^2$

Weak axis radius of gyration: $r_y = 2.15 \text{ in.}$

Web Depth in Compression: $D_c = 14.14 \text{ in.}$

6.2.3 Moment Capacity in the Positive Moment Region

6.2.3.1 Determination of Amount Composite

The theoretical composite moment of inertia of an interior girder and deck I_c can be found as:

$$I_c = 11{,}300 \text{ in}^4$$

The theoretical non-composite moment of inertia of the interior girder I_{nc} can be found as:

$$I_{nc} = 4470 \text{ in}^4$$

The theoretical composite deflection Δ_c of an individual interior girder, G3, under Path 1 loading can be found as:

$$\Delta_c = 0.236 \text{ in.}$$

The theoretical non-composite deflection Δ_{nc} of an individual interior girder, G3, under Path 1 loading can be found as:

$$\Delta_{nc} = 0.438 \text{ in.}$$

The measured test deflection Δ_{test} of Girder 3 under Path 1 loading is 0.351 in.

$$\Delta_{test} = 0.351 \text{ in.}$$

Therefore, the prorated deflection ratio $\Delta_{prorated}$ is:

$$\begin{aligned} \Delta_{prorated} &= \frac{\Delta_{nc} - \Delta_{test}}{\Delta_{nc} - \Delta_c} \\ &= \frac{0.438 \text{ in.} - 0.351 \text{ in.}}{0.438 \text{ in.} - 0.236 \text{ in.}} = 0.43 \end{aligned} \tag{6.51}$$

Therefore, the acting moment of inertia of the girder I_{equiv} can be approximated as:

$$\begin{aligned} I_{equiv} &= I_{nc} + \Delta_{proprated}(I_c - I_{nc}) \\ &= 4470 + 0.43(11{,}300 - 4470) = 7407 \text{ in}^4 \end{aligned} \tag{6.52}$$

Substituting known values into Equation C-I3-4 in the 14th edition of the AISC Steel Construction Manual yields:

$$I_{equiv} = I_{nc} + \sqrt{\frac{\sum Q_n}{C_f}}(I_c - I_{nc}) \xrightarrow{yields} \tag{6.53}$$

$$7407 = 4470 + \sqrt{\frac{\sum Q_n}{C_f}}(11{,}300 - 4470)$$

Solving for $\frac{\sum Q_n}{C_f}$ yields:

$$\frac{\sum Q_n}{C_f} = 0.66$$

This ratio is used to reduce the controlling force in a composite section analysis.

6.2.3.2 Determination of Nominal Moment Capacity

The unreduced force in the slab C_i is equal to:

$$\begin{aligned} C_i &= 0.85 f'_c b_e t_d \\ &= 0.85(2.5\text{ ksi})(72\text{ in.})(6\text{ in.}) = 918\text{ kips} \end{aligned} \tag{6.54}$$

The unreduced force in the girder T_i is equal to:

$$\begin{aligned} T_i &= AF_y \\ &= (31.7\text{ in}^2)(33\text{ ksi}) = 1046\text{ kips} \end{aligned} \tag{6.55}$$

The slab force controls. Therefore, the reduced slab force C is equal to:

$$\begin{aligned} C &= \frac{\sum Q_n}{C_f} C_i \\ &= (0.66)(918\text{ kips}) = 605.9\text{ kips} \end{aligned} \tag{6.56}$$

The depth of the compressive stress block in the deck a is equal to:

$$a = \frac{C}{0.85f'_c b_e} \tag{6.57}$$

$$= \frac{605.9\text{ kips}}{0.85(2.5\text{ ksi})(72\text{ in.})} = 3.96\text{ in.}$$

Equation 10-126 in the AASHTO Standard Specifications gives the compressive force in the steel C' as:

$$C' = \frac{AF_y - C}{2} \tag{6.58}$$

$$= \frac{1046\text{ kips} - 605.9\text{ kips}}{2} = 220.1\text{ kips}$$

Since $C' = 220.1\ kips < A_f F_y = 263.3\ kips$, Equation 10-127 in the AASHTO Standard Specifications gives the neutral axis location y as:

$$y = \frac{C'}{A_f F_y} t_f \tag{6.59}$$

$$= \frac{220.1\text{ kips}}{(7.98\text{ in}^2)(33\text{ ksi})}(0.76\text{ in.}) = 0.635\text{ in.}$$

This value is measured down from the top of the girder top flange.

By summing moments of all of the components (girder in tension, girder in compression, reduced slab in compression) about this neutral axis, the plastic moment capacity M_p can be found as:

$$M_p = 1477.3\text{ kip–ft}$$

From Equation 10-129a in the AASHTO Standard Specifications, the factor D' is equal to:

$$D' = 0.9\frac{d + t_d}{7.5} \tag{6.60}$$

$$= 0.9\frac{29.8 + 6}{7.5} = 4.296\text{ in.}$$

The distance from the top of the slab to the plastic neutral axis D_p is:

$$\begin{aligned} D_p &= t_d + y \\ &= 6 \text{ in.} + 0.635 \text{ in.} = 6.635 \text{ in.} \end{aligned} \tag{6.61}$$

The equivalent, partial composite section modulus S_{equiv} can be found as:

$$\begin{aligned} S_{equiv} &= \frac{I_{equiv}}{d - y} \\ &= \frac{7407 \text{ in}^4}{29.8 \text{ in.} - 0.635 \text{ in.}} = 254.0 \text{ in}^3 \end{aligned} \tag{6.62}$$

The elastic moment capacity of the section M_y can also be found as:

$$\begin{aligned} M_y &= S_{equiv} F_y \\ &= (254.0 \text{ in}^3)(33 \text{ ksi}) = 8382 \text{ kip–in.} = 698.5 \text{ kip–ft} \end{aligned} \tag{6.63}$$

From Equation 10-129c in the AASHTO Standard Specifications, the nominal moment capacity M_n can be found as:

$$\begin{aligned} M_n &= \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right) \\ &= \frac{(5)(1477.3) - (0.85)(698.5)}{4} + \frac{(0.85)(698.5) - 1477.3}{4}\left(\frac{6.635}{4.296}\right) \\ M_n &= 1357 \text{ kip–ft} \end{aligned} \tag{6.64}$$

This is approximately 90 percent of the fully composite moment capacity, calculated to be 1514 kip-ft. It is a 65.0 percent increase from the non-composite moment capacity of 163.3 kip-ft.

6.2.4 Structural Analysis for Moment Demand in the Positive Moment Region

6.2.4.1 Applied Positive Live Load Moment

From Section 6.1.5.1, the distributed applied live load positive moment M_{LL} with dynamic effects, on an individual interior girder is:

$$M_{LL} = 503 \text{ kip–ft}$$

6.2.4.2 Dead Load Positive Moment of Structural Components

From Section 6.1.5.2, the applied positive moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 140.8 \text{ kip–ft}$$

6.2.4.3 Superimposed Dead Load Positive Moment

From Section 6.1.5.3, the applied positive moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 36.2 \text{ kip–ft}$$

6.2.4.4 Total Dead Load Positive Moment

The total applied positive moment due to structural dead loads and superimposed dead loads M_{DL} is:

$$M_{DL} = 177.0 \text{ kip–ft}$$

6.2.5 LFR Load Rating for Positive Flexure

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{6.65}$$

6.2.5.1 Strength Check

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \qquad (6.66) \\ &= \frac{1357.0 - [(1.3)(177.0)]}{(2.17)(503.0)} = 1.03 \end{aligned}$$

Passes.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$\begin{aligned} RF_O &= \frac{M_n - A_1 M_{DL}}{A_2 M_{LL}} \qquad (6.67) \\ &= \frac{1357.0 - [(1.3)(177.0)]}{(1.3)(503.0)} = 1.72 \end{aligned}$$

Passes.

6.2.5.2 Service Check

For composite sections, per AASHTO Standard Specifications Article 10.57.2 the service capacity C_{serv} is equal to:

$$\begin{aligned} C_{serv} &= 0.95 F_y \qquad (6.68) \\ &= (0.95)(33 \text{ ksi}) = 31.35 \text{ ksi} \end{aligned}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.67$

$$RF_I = \frac{C_{serv} - A_1\left(\frac{M_{DC}}{S_x}\right) - A_1\left(\frac{M_{SDL}}{S_{equiv}}\right)}{A_2\left(\frac{M_{LL}}{S_{equiv}}\right)} \tag{6.69}$$

$$= \frac{31.15 - (1.0)\left[\frac{(140.8)(12)}{299.0}\right] - (1.0)\left[\frac{(36.2)(12)}{254.0}\right]}{(1.67)\left[\frac{(503.0)(12)}{254.0}\right]} = 0.60$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.0$

Live Load Factor, $A_2 = 1.0$

$$RF_O = \frac{C_{serv} - A_1\left(\frac{M_{DC}}{S_x}\right) - A_1\left(\frac{M_{SDL}}{S_{equiv}}\right)}{A_2\left(\frac{M_{LL}}{S_{equiv}}\right)} \tag{6.70}$$

$$= \frac{31.15 - (1.0)\left[\frac{(140.8)(12)}{299.0}\right] - (1.0)\left[\frac{(36.2)(12)}{254.0}\right]}{(1.0)\left[\frac{(503.0)(12)}{254.0}\right]} = 1.01$$

Passes.

6.2.6 LFR Load Rating for Negative Flexure

Assume non-composite action is still occurring in the negative moment region. Therefore, nothing changes in the negative moment region load rating calculations.

6.2.6.1 Strength Check

Inventory Rating

$$RF_I = 0.88$$

Operating Rating

$$RF_O = 1.47$$

6.2.6.2 Service Check

Inventory Rating

$$RF_I = 0.94$$

Operating Rating

$$RF_O = 1.57$$

6.2.7 Controlling Rating Factors

The controlling LFR rating factors come from the service check in the positive moment region. The controlling rating factors for Bridge SC-12 from the Level II partial composite action load rating analysis are equal to:

$$RF_I = 0.60$$

$$RF_O = 1.01$$

The basic load rating controlling RFs were 0.54 for Inventory and 0.91 for Operating. The new rating factors represent an 11.1 percent increase for Inventory and an 11.0 percent increase for Operating. Table 6.1 compares the controlling RFs determined using a Level II Analysis for partial composite action to the controlling RFs determined in the basic load rating.

Table 6.1. Partial Composite RF Comparison

Rating Factor	Basic Load Rating	Level II Partial Composite Load Rating	Level II Partial Composite/Basic Load Rating
Inventory	0.54	0.60	1.11
Operating	0.91	1.01	1.11

7 FLEXURAL LOAD RATING ANALYSIS FOR AN INTERIOR GIRDER OF BRIDGE CM-5 USING THE LFR METHOD

7.1 BASIC LOAD RATING ANALYSIS

The basic load rating presented in Section 6.2 of the Volume 1 report (Hueste et al. 2019a) for Bridge CM-5 is based on information gathered from the standard drawing for concrete slab and girder bridges (pan form) provided on the TxDOT website titled “CG 30'-4" Spans” (TxDOT 2005). During the field load testing of Bridge CM-5, on-site measurements were taken and found to be different from what was shown on the standard drawing. The bottom web width of the interior girders were measured to be 7 in., versus 8 in. indicated on the standard drawing. In addition, the asphalt layer was 4.5 in. thick, which is more than the standard 2 in. thickness used for basic load rating analysis in Volume 1. Using Ground Penetrating Radar (GPR), it was found that the girders consisted of a single layer of tensile reinforcement located 3 in. from the bottom of the girder, as opposed to the two layers of tension reinforcement indicated on the standard drawings. This section shows the initial basic load rating performed for Bridge CM-5 updated based on measurements taken during the field test. This basic load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). This initial basic load rating was performed under the assumption that the girder ends are simply supported. The resulting rating factors are used for comparison when conducting load ratings assuming partial end restraint, updated material strengths, or a combination of the two.

7.1.1 Bridge Characteristics

Bridge CM-5 is a two-lane bridge with a total length of 30 ft. The bridge was designed as simply supported, and has a controlling span for load rating of 29 ft. The bridge width is 21 ft 8 in. with a roadway width of 21 ft. The TxDOT HS-20 RFs are 0.72 for Inventory and 1.00 for Operating. The transverse section of Bridge CM-5 is shown in Figure 7.1.

Span Length: $L = 29\ \text{ft}$ Bridge Width: $W = \ 21\ \text{ft}\ 8\ \text{in.}$

Concrete Compressive Strength: $f'_c = 4.0 \text{ ksi}$

Steel yield strength: $f_y = 33 \text{ ksi}$

Concrete Density: $\gamma_c = 150 \text{ pcf}$

Asphalt Density: $\gamma_{ws} = 144 \text{ pcf}$

Concrete Girder Section: 24 in. deep, 7 in. wide web

Girder Spacing: $S = 3 \text{ ft}$

Number of Girders: $N_G = 8$

Deck Thickness: $t_d = 4.5 \text{ in.}$

Asphalt Thickness: $t_{ws} = 4.5 \text{ in.}$

Girder Linear Weight: $w_{SG} = 429 \ \frac{\text{lb}}{\text{ft}}$

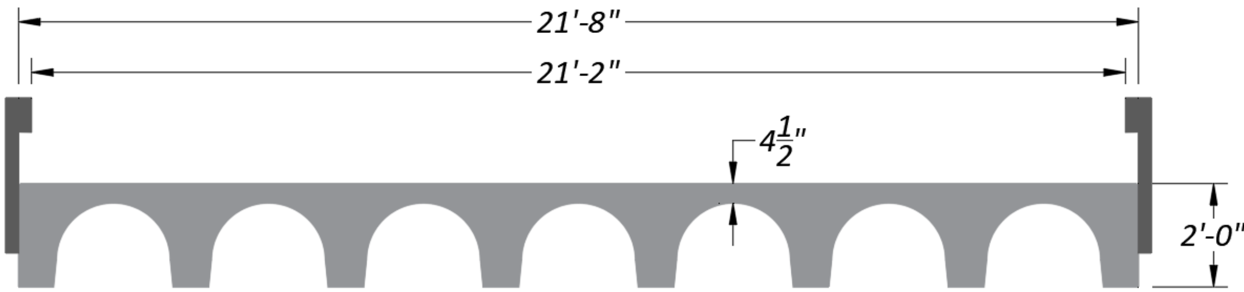

Figure 7.1. Transverse Section of Bridge CM-5 (TxDOT 2018a)

7.1.2 Sectional Properties

Ground Penetrating Radar (GPR) was used to verify the reinforcement locations within the girder sections. A single layer of longitudinal reinforcement was found located at 3 in. from the bottom of the girder and another layer of reinforcement was found at 21 in. from the bottom of the girder. The area of tension reinforcement is assumed to match that of one layer of steel (2 - #11 bars) in TxDOT's standard drawings for this bridge system (TxDOT 2005). The top layer of steel is neglected for flexural strength calculations. The girder web thickness given is the width at the bottom of the web.

Girder Web Thickness (bottom): $t_w = 7.0 \text{ in.}$

Assumed Tension Reinf. Area: $A_s = 3.12 \text{ in}^2$

Total Girder Depth: $h = 24.0 \text{ in.}$

Tension Reinforcement Depth: $d = 21 \text{ in.}$

7.1.3 Moment Capacity

The procedure outlined in Article 8.16 of the AASHTO Standard Specifications (AASHTO 2002) is followed to determine the moment capacity of an individual girder.

Calculate the flexural capacity of the girder according to AASHTO Standard Specifications Article 8.16. Ultimate strain in concrete, $\varepsilon_{cu} = 0.003$.

The stress block factor β_1 can be calculated as:

$$0.65 \leq \beta_1 = 0.85 - \left(0.05(f_c{}' - 4)\right) \leq 0.85 \tag{7.1}$$

$$\beta_1 = 0.85$$

The depth of equivalent stress block,

$$\begin{aligned} a &= \frac{A_s f_y}{0.85 f_c' b_f} \\ &= \frac{(3.12\ \text{in}^2)(33\ \text{ksi})}{(0.85)(4\ \text{ksi})(36\ \text{in.})} \\ &= 0.841\ \text{in.} \end{aligned} \tag{7.2}$$

The distance from extreme compression fiber to the neutral axis,

$$\begin{aligned} \text{c} &= \frac{a}{\beta_1} \\ &= \frac{0.841\ \text{in.}}{0.85} \\ &= 0.989\ \text{in.} \end{aligned} \tag{7.3}$$

Check if steel has yielded:

$$\begin{aligned} \varepsilon_s &= \varepsilon_{cu}\left(\frac{d - c}{c}\right) &&\geq \frac{f_y}{E_s} \\ &= 0.003\left(\frac{21.0\ \text{in.} - 0.989\text{in.}}{0.989\ \text{in.}}\right) &&\geq \frac{33\ \text{ksi}}{29{,}000\ \text{ksi}} \\ &= 0.0607 &&\geq 0.00114\quad \text{(Steel yields)} \end{aligned} \tag{7.4}$$

Flexural reduction factor, $\phi = 0.90$. Nominal moment capacity,

$$\begin{aligned}\phi M_n &= \phi A_s f_y \left(d - \frac{a}{2}\right) \qquad (7.5)\\ &= (0.90)(3.12\ \text{in}^2)(33\ \text{ksi})\left(21.0\ \text{in.} - \frac{0.841\ \text{in.}}{2}\right)\left(\frac{1\ \text{ft}}{12\ \text{in.}}\right)\\ &= 158.9\ \text{kip–ft}\end{aligned}$$

7.1.4 Structural Analysis for Moment Demand

7.1.4.1 Applied Live Load Moment

For LFR, use the LLDF for wheel loads provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. Bridge CM-5 is a two-lane bridge,

$$\begin{aligned}DF &= \frac{S}{6.0\ \text{ft}} \qquad (7.6)\\ &= \frac{3.0\ \text{ft}}{6.0\ \text{ft}}\\ &= 0.50\end{aligned}$$

The Impact Factor I is applied to the live load to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to,

$$\begin{aligned}I &= \frac{50}{L + 125} \leq 0.3 \qquad (7.7)\\ &= \frac{50}{29 + 125} \leq 0.3\\ &= 0.325 \leq 0.3\\ I &= 0.3\end{aligned}$$

The applied live load moment for wheel load without the impact factor M_{HS20} is obtained from the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1:

$$M_{HS20} = 133.50\ \frac{\text{kip–ft}}{\text{wheel line}}$$

Therefore, the distributed applied live load moment M_L with dynamic effects, on an individual interior girder can be calculated as:

$$\begin{aligned} M_{LL} &= M_{HS20} DF(1+I) \\ &= (133.50\ \text{kip–ft})(0.50)(1.3) \\ &= 86.8\ \text{kip–ft} \end{aligned} \tag{7.8}$$

7.1.4.2 Dead Load Moment of Structural Components

The cross-sectional area of the interior pan girder was determined to be $2.86\ \text{ft}^2$ and the self-weight of the cross-section w_{SG} is calculated as,

$$\begin{aligned} w_{ws} &= \gamma_c A_G \\ &= (150\ \text{pcf})(2.86\ \text{ft}^2) \\ &= 428.9\ \text{lb/ft} \end{aligned} \tag{7.9}$$

The applied moment due to dead load of structural components M_{DC} is,

$$\begin{aligned} M_{DC} &= \frac{w_{DC} L^2}{8} \\ &= \frac{\left(0.429\ \frac{\text{kip}}{\text{ft}}\right)(29\ \text{ft})^2}{8} \\ &= 45.1\ \text{kip–ft} \end{aligned} \tag{7.10}$$

7.1.4.3 Superimposed Dead Load Moment

The superimposed dead load on an individual girder due to the wearing surface w_{ws} can be calculated as,

$$\begin{aligned} w_{ws} &= \gamma_{ws} t_{ws} S \\ &= (144\ \text{pcf})\left(\frac{4.5\ \text{in.}}{12\,\frac{\text{in.}}{\text{ft}}}\right)(3\ \text{ft}) \\ &= 0.162\ \text{kip/ft} \end{aligned} \tag{7.11}$$

Therefore, the total distributed load due to superimposed dead load w_{SDL} is,

$$w_{SDL} = 0.162 \text{ kip/ft}$$

The applied moment due to superimposed dead load M_{SDL} is,

$$\begin{aligned} M_{SDL} &= \frac{w_{SDL} L^2}{8} \\ &= \frac{\left(0.162 \frac{\text{kip}}{\text{ft}}\right)(29 \text{ ft})^2}{8} \\ &= 17.0 \text{ kip–ft} \end{aligned} \tag{7.12}$$

7.1.4.4 Total Dead Load Moment

The total applied moment due to structural dead loads and superimposed dead loads M_{DL} is,

$$\begin{aligned} M_{DL} &= M_{DC} + M_{SDL} \\ &= (45.1 \text{ kip–ft}) + (17.0 \text{ kip–ft}) = 62.1 \text{ kip–ft} \end{aligned} \tag{7.13}$$

7.1.5 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2 (L + I)} \tag{7.14}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{158.9 - [(1.3)(62.1)]}{(2.17)(86.8)} \\ &= 0.42 \end{aligned} \tag{7.15}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{7.16}$$

$$= \frac{158.9 - [(1.3)(62.1)]}{(1.3)(86.8)}$$

$$= 0.69$$

Does not pass.

LFR rating factors for Bridge CM-5 from the basic load rating analysis are equal to:

$$RF_I = 0.42$$

$$RF_O = 0.69$$

TxDOT reports the rating factors for this bridge to be 0.72 for Inventory and 1.00 for Operating. Due to the poor condition rating of the substructure (Item 60 < 6), TxDOT's Off-System Load Rating flowchart (TxDOT 2018b) does not allow the posting to be removed. The bridge is currently posted at inventory level with an inspection frequency of less than two years. The calculated inventory and operating RFs are different from the TxDOT values. The reasons for the difference could not be confirmed due to lack of available information used to calculate these RFs. However, basic load rating calculations conducted for similar concrete multi-girder bridges with available details show good agreement with the TxDOT ratings, as summarized in Section 6.2 of Volume 1 report (Hueste et al. 2019).

7.2 LOAD RATING ANALYSIS CONSIDERING REDUCED NUMBER OF LANES

This example shows the load rating analysis performed for Bridge CM-5 considering a reduction in number of lanes used in analysis due to the relatively narrow width of the bridge and low traffic conditions. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). It was also performed under the assumption that the girder ends are simply-supported. This load rating considering a reduction in the number of lanes is then compared to the basic load rating.

7.2.1 Bridge Characteristics

Bridge CM-5 is a two-lane bridge with a total length of 30 ft. The bridge was designed as simply supported, and has a controlling span for load rating of 29 ft. The bridge width is 21 ft 8 in. with a roadway width of 21 ft. The TxDOT HS-20 RFs are 0.72 for Inventory and 1.00 for Operating. The transverse section of Bridge CM-5 is shown in Figure 7.2. Due to the lack of structural drawings for this bridge, field measurements were used and some assumptions were made, as described in Section 7.1.

Span Length: $L = 29\ \text{ft}$	Bridge Width: $W = \ 21\ \text{ft}\ 8\ \text{in.}$
Concrete Compressive Strength: $f'_c = 4.0\ \text{ksi}$	Girder Spacing: $S = 3\ \text{ft}$
Steel yield strength: $f_y = 33\ \text{ksi}$	Number of Girders: $N_G = 8$
Concrete Density: $\gamma_c = 150\ \text{pcf}$	Deck Thickness: $t_d = 4.5\ \text{in.}$
Asphalt Density: $\gamma_{ws} = 144\ \text{pcf}$	Asphalt Thickness: $t_{ws} = 4.5\ \text{in.}$
Concrete Girder Section: 24 in. deep, 7 in. wide web	Girder Linear Weight: $w_{SG} = 429\ \frac{\text{lb}}{\text{ft}}$

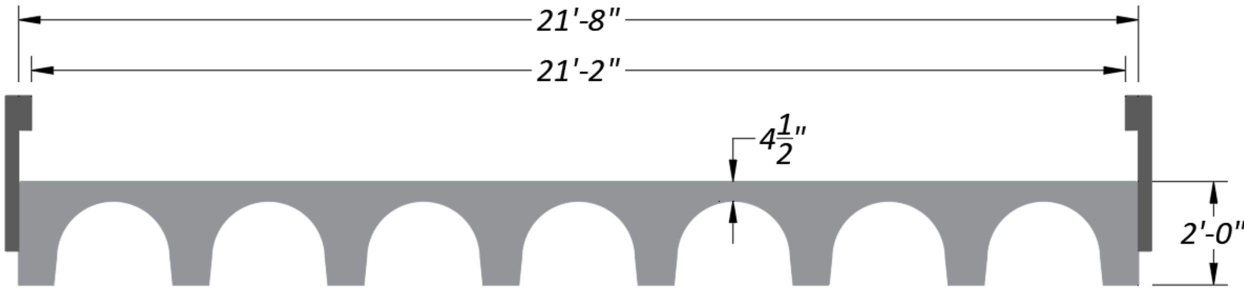

Figure 7.2. Transverse Section of Bridge CM-5 (TxDOT 2018a)

7.2.2 Sectional Properties

Ground Penetrating Radar (GPR) was used to verify the reinforcement locations within the girder sections. A single layer of longitudinal reinforcement was found located at 3 in. from the bottom of the girder and another layer of reinforcement was found at 21 in. from the bottom of the girder. The area of tension reinforcement is assumed to match that of one layer of steel (2 - #11 bars) in TxDOT's standard drawings for this bridge system (TxDOT 2005). The top layer of steel is neglected. The girder web thickness given is the width at the bottom of the web.

Girder Web Thickness (bottom): $t_w = 7.0$ in.	Total Girder Depth: $h = 24.0$ in.
Assumed Tension Reinf. Area: $A_s = 3.12\ \text{in}^2$	Tension Reinforcement Depth: $d = 21.0$ in.

7.2.3 Moment Capacity

Detailed in 7.1.3, the final reduced moment capacity ϕM_n for an individual girder was determined to be:

$$\phi M_n \quad = \quad 158.9\ \text{kip–ft}$$

7.2.4 Structural Analysis for Moment Demand

7.2.4.1 Applied Live Load Moment

For LFR, use the LLDF equations provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. As Bridge CM-5 is being analyzed as a one-lane bridge, for an interior girder

$$\begin{aligned} DF &= \frac{\mathrm{S}}{6.5\ \mathrm{ft}} \\ &= \frac{3.0\ \mathrm{ft}}{6.5\ \mathrm{ft}} \\ &= 0.462 \end{aligned} \tag{7.17}$$

The Impact Factor I is applied to the live load effect to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to:

$$\begin{aligned} I &= \frac{50}{L + 125} \leq 0.3 \\ &= \frac{50}{29 + 125} \leq 0.3 \\ &= 0.325 \leq 0.3 \\ I &= 0.3 \end{aligned} \tag{7.18}$$

The applied live load moment for wheel load without the impact factor M_{HS20} is obtained from the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1:

$$M_{HS20} = 133.50\ \frac{\mathrm{kip{-}ft}}{\mathrm{wheel\ line}}$$

Therefore, the distributed applied live load moment M_L with dynamic effects, on an individual interior girder can be calculated as:

$$\begin{aligned} M_{LL} &= M_{HS20} DF(1 + I) \\ &= (133.50\ \mathrm{kip{-}ft})(0.462)(1.3) \\ &= 80.1\ \mathrm{kip{-}ft} \end{aligned} \tag{7.19}$$

7.2.4.2 Dead Load Moment of Structural Components

Detailed in Section 7.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 45.1 \text{ kip–ft}$$

7.2.4.3 Superimposed Dead Load Moment

Detailed in Section 7.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 17.0 \text{ kip–ft}$$

7.2.4.4 Total Dead Load Moment

Detailed in Section 7.1.4.4, the total applied moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$M_{DL} = 62.1 \text{ kip–ft}$$

7.2.5 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{7.20}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{158.9 - [(1.3)(62.1)]}{(2.17)(80.1)} \\ &= 0.45 \end{aligned} \tag{7.21}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{7.22}$$

$$= \frac{158.9 - [(1.3)(62.1)]}{(1.3)(80.1)}$$

$$= 0.75$$

Does not pass.

LFR rating factors for Bridge CM-5 from the refined load rating analysis using lane reduction are equal to:

$$RF_I = 0.45$$

$$RF_O = 0.75$$

The basic load rating controlling RFs are 0.42 for Inventory and 0.69 for Operating. The new rating factors represent a 7.1 percent increase for Inventory and an 8.7 percent increase for Operating relative to the values computed in the basic load rating analysis. Table 5.1 compares the flexural strength RFs determined using a reduction in number of lanes to the flexural strength RFs determined in the basic load rating analysis.

Table 7.1. Lane Reduction RF Comparison

Rating Factor	Basic Load Rating	Load Rating with Lane Reduction	Lane Reduction/Basic Load Rating
Inventory	0.42	0.45	1.07
Operating	0.69	0.75	1.09

7.3 LOAD RATING ANALYSIS CONSIDERING ONLY MEASURED MATERIAL PROPERTIES

This example shows an abbreviated version of a load rating analysis performed for Bridge CM-5 when considering measured material properties. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). It was also performed under the assumption that the girder ends are simply-supported. The results of this load rating are compared to the results of the basic load rating analysis.

7.3.1 Bridge Characteristics

Bridge CM-5 is a two-lane bridge with a total length of 30 ft. The bridge was designed as simply supported, and has a controlling span for load rating of 29 ft. The bridge width is 21 ft 8 in. with a roadway width of 21 ft 0 in. The TxDOT HS-20 RFs are 0.72 for Inventory and 1.00 for Operating. The transverse section of Bridge CM-5 is shown in Figure 7.3.

Span Length: $L = 29\ \text{ft}$

Concrete Compressive Strength: $f'_c = 7.0\ \text{ksi}$

Steel yield strength: $f_y = 33\ \text{ksi}$

Concrete Density: $\gamma_c = 150\ \text{pcf}$

Asphalt Density: $\gamma_{ws} = 144\ \text{pcf}$

Concrete Girder Section: 24 in. deep, 7 in. wide web

Bridge Width: $W = 21\ \text{ft}\ 8\ \text{in.}$

Girder Spacing: $S = 3\ \text{ft}$

Number of Girders: $N_G = 8$

Deck Thickness: $t_d = 4.5\ \text{in.}$

Asphalt Thickness: $t_{ws} = 4.5\ \text{in.}$

Girder Linear Weight: $w_{SG} = 429\ \frac{\text{lb}}{\text{ft}}$

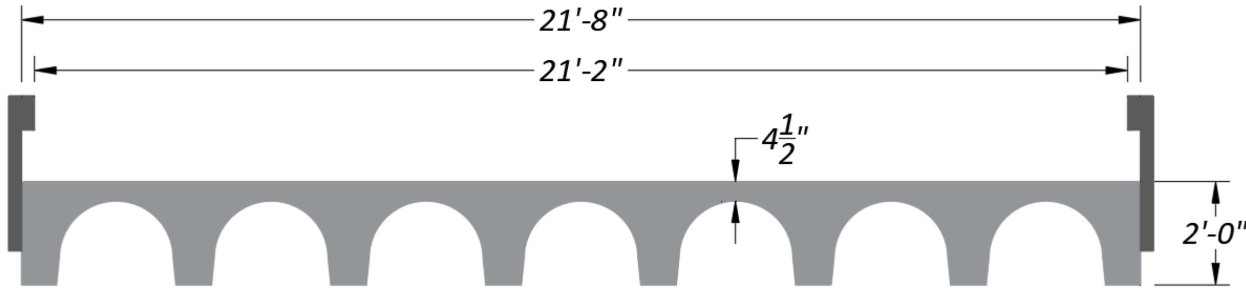

Figure 7.3. Transverse Section of Bridge CM-5 (TxDOT 2018a)

7.3.2 Sectional Properties

Ground Penetrating Radar (GPR) was used to verify the reinforcement locations within the girder sections. A single layer of longitudinal reinforcement was found located at 3 in. from the bottom of the girder and another layer of reinforcement was found at 21 in. from the bottom of the girder. The area of tension reinforcement is assumed to match that of one layer of steel (2 - #11 bars) in TxDOT's standard drawings for this bridge system (TxDOT 2005). The top layer of steel is neglected. The girder web thickness given is the width at the bottom of the web.

Girder Web Thickness (bottom): $t_w = 7.0\text{ in.}$ Total Girder Depth: $h = 24.0\text{ in.}$

Tension Reinf. Area: $A_s = 3.12\text{ in}^2$ Tension Reinforcement Depth: $d = 21.0\text{ in.}$

7.3.3 Moment Capacity

The procedure outlined in Article 8.16 of the AASHTO Standard Specifications (AASHTO 2002) is followed to determine the moment capacity of an individual girder. First, it is assumed that the tensile reinforcement yields and this assumption is verified. If the tensile reinforcement does not yield, then the tensile stress is calculated using Hooke's Law and this stress is used to determine the nominal moment capacity of the section.

Calculate the flexural capacity of the girder according to AASHTO Standard Specifications Article 8.16. Ultimate strain in concrete, $\varepsilon_{cu} = 0.003$.

The stress block factor for 7 ksi concrete is calculated using Equation (7.1) as,

$$\beta_1 = 0.70$$

The depth of equivalent stress block,

$$\begin{aligned} a &= \frac{A_s f_y}{0.85 f_c' b_f} && (7.23) \\ &= \frac{(3.12\ \text{in}^2)(33\ \text{ksi})}{(0.85)(7\ \text{ksi})(36\ \text{in.})} \\ &= 0.481\ \text{in.} \end{aligned}$$

The distance from extreme compression fiber to the neutral axis,

$$\begin{aligned} c &= \frac{a}{\beta_1} && (7.24) \\ &= \frac{0.481\ \text{in.}}{0.70} \\ &= 0.687\ \text{in.} \end{aligned}$$

Check if steel has yielded:

$$\begin{aligned} \varepsilon_s &= \varepsilon_{cu}\left(\frac{d-c}{c}\right) && \geq \frac{f_y}{E_s} && (7.25) \\ &= 0.003\left(\frac{(21.0\ \text{in.} - 0.687\ \text{in.})}{0.687\ \text{in.}}\right) && \geq \frac{33\ \text{ksi}}{29{,}000\ \text{ksi}} \\ &= 0.0887 && \geq 0.00114\ \text{(Steel yields)} \end{aligned}$$

Flexural reduction factor, $\phi = 0.90$. Nominal moment capacity,

$$\begin{aligned} \phi M_n &= \phi A_s f_y \left(d - \frac{a}{2}\right) && (7.26) \\ &= (0.90)(3.12\ \text{in}^2)(33\ \text{ksi})\left(21.0\ \text{in.} - \frac{0.481\ \text{in.}}{2}\right)\left(\frac{1\ \text{ft}}{12\ \text{in.}}\right) \\ &= 160.3\ \text{kip–ft} \end{aligned}$$

7.3.4 Structural Analysis for Moment Demand

7.3.4.1 Applied Live Load Moment

Detailed in Section 7.1.4.1, the distributed applied live load moment with dynamic effects M_{LL} on an individual interior girder is:

$$M_{LL} = 86.8 \text{ kip–ft}$$

7.3.4.2 Dead Load Moment of Structural Components

Detailed in Section 7.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 45.1 \text{ kip–ft}$$

7.3.4.3 Superimposed Dead Load Moment

Detailed in Section 7.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 17.0 \text{ kip–ft}$$

7.3.4.4 Total Dead Load Moment

Detailed in Section 7.1.4.4, the total applied moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$M_{DL} = 62.1 \text{ kip–ft}$$

7.3.5 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{7.27}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{160.3 - [(1.3)(62.1)]}{(2.17)(86.8)} \\ &= 0.42 \end{aligned}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$\begin{aligned} RF_O &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \qquad (7.28) \\ &= \frac{160.3 - [(1.3)(62.1)]}{(1.3)(86.8)} \\ &= 0.71 \end{aligned}$$

Does not pass.

LFR rating factors for Bridge CM-5 from the refined load rating analysis using the measured material properties are equal to:

$$RF_I = 0.42$$

$$RF_O = 0.71$$

The calculated basic load rating RFs are 0.42 for Inventory and 0.69 for Operating. The new rating factors represent a 0.0 percent increase for Inventory and a 2.9 percent increase for Operating, because the concrete compressive strength does not have a significant impact on the flexural strength of the girders. Table 7.2 compares the controlling RFs determined using measured material properties to the controlling RFs determined in the basic load rating.

Table 7.2. Measured Material Properties RF Comparison

Rating Factor	Basic Load Rating	Load Rating with Measured Material Properties	Measured Material Properties/Basic Load Rating
Inventory	0.42	0.42	1.00
Operating	0.69	0.71	1.03

7.4 LOAD RATING ANALYSIS CONSIDERING ONLY PARTIAL END RESTRAINT

This section shows an abbreviated version of a load rating performed for Bridge CM-5 when considering partial end restraint. This load rating was performed following Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). However, a Level II Analysis for end restraint is also performed to determine new applied live load and dead load moments for the girder being analyzed. This Level II Analysis is based on the calibrated FEM model results using load test performed on the bridge in the field (Hueste et al. 2019b). The results of this load rating are compared to the results of the basic load rating analysis.

7.4.1 Bridge Characteristics

Bridge CM-5 is a two-lane bridge with a total length of 30 ft. The bridge was designed as simply supported, and has a controlling span for load rating of 29 ft. The bridge width is 21 ft 8 in. with a roadway width of 21 ft. The transverse section of Bridge CM-5 is shown in Figure 7.4.

Span Length: $L = 29 \text{ ft}$

Concrete Compressive Strength: $f'_c = 4.0 \text{ ksi}$

Steel yield strength: $f_y = 33 \text{ ksi}$

Concrete Density: $\gamma_c = 150 \text{ pcf}$

Asphalt Density: $\gamma_{ws} = 144 \text{ pcf}$

Concrete Girder Section: 24 in. deep, 7 in. wide web

Bridge Width: $W = 21 \text{ ft } 8 \text{ in.}$

Girder Spacing: $S = 3 \text{ ft}$

Number of Girders: $N_G = 8$

Deck Thickness: $t_d = 4.5 \text{ in.}$

Asphalt Thickness: $t_{ws} = 4.5 \text{ in.}$

Girder Linear Weight: $w_{SG} = 429 \ \frac{\text{lb}}{\text{ft}}$

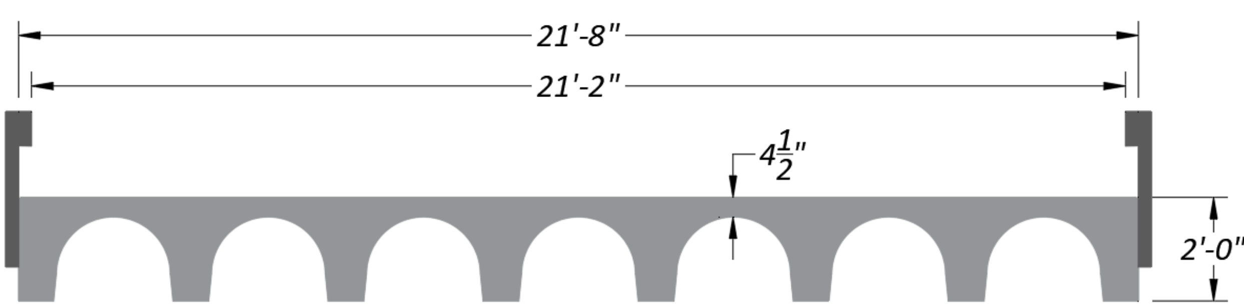

Figure 7.4. Transverse Section of Bridge CM-5 (TxDOT 2018a)

7.4.2 Sectional Properties

Ground Penetrating Radar (GPR) was used to verify the reinforcement locations within the girder sections. A single layer of longitudinal reinforcement was found located at 3 in. from the bottom of the girder and another layer of reinforcement was found at 21 in. from the bottom of the girder. The area of tension reinforcement is assumed to match that of one layer of steel (2 - #11 bars) in TxDOT's standard drawings for this bridge system (TxDOT 2005). The top layer of steel is neglected. The girder web thickness given is the width at the bottom of the web.

Girder Web Thickness (bottom): $t_w = 7.0$ in.	Total Girder Depth: $h = 24.0$ in.
Assumed Tension Reinf. Area: $A_s = 3.12\ \text{in}^2$	Tension Reinforcement Depth: $d = 21.0$ in.

7.4.3 Moment Capacity

Detailed in Section 7.1.3, the final reduced moment capacity ϕM_n for an individual girder was determined to be

$$\phi M_n = 158.9\ \text{kip–ft}$$

7.4.4 Structural Analysis for Moment Demand

7.4.4.1 Applied Live Load Moment Considering Simply Supported Boundary Conditions

For LFR, use the LLDF equations provided by the AASHTO Standard Specifications (2002) in Table 3.23.1. As Bridge CM-5 is being analyzed as a two-lane bridge, for an interior girder

$$\begin{aligned} DF &= \frac{\text{S}}{6.0\ \text{ft}} \\ &= \frac{3.0\ \text{ft}}{6.0\ \text{ft}} \\ &= 0.50 \end{aligned} \tag{7.29}$$

The Impact Factor I is applied to the live load effect to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to:

$$\begin{aligned} I &= \frac{50}{L+125} \leq 0.3 \\ &= \frac{50}{29+125} \leq 0.3 \\ &= 0.325 \leq 0.3 \\ I &= 0.3 \end{aligned} \tag{7.30}$$

The applied live load moment for wheel load without the impact factor M_{HS20} is obtained from the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1:

$$M_{HS20} = 133.50\ \frac{\text{kip–ft}}{\text{wheel line}}$$

Therefore, the distributed applied live load moment M_L with dynamic effects, on an individual interior girder can be calculated as:

$$\begin{aligned} M_{LL-simple} &= M_{HS20} DF(1+I) \\ &= (133.50\ \text{kip–ft})(0.50)(1.3) \\ &= 86.8\ \text{kip–ft} \end{aligned} \tag{7.31}$$

7.4.4.2 Consideration of End Restraint

The modulus of elasticity for concrete was calculated according to Article 8.7.2 of the AASHTO Standard Specifications (AASHTO 2002) as:

$$\begin{aligned} E_c &= 33 K_1 {w_c}^{1.5} \sqrt{{f_c}'} \\ &= \left[(33)(1.0)((150\ \text{lb/ft}^3)^{1.5})\sqrt{4000\ \text{psi}}\right]/1000 \\ &= 3834\ \text{ksi} \end{aligned} \tag{7.32}$$

From load test results for an interior girder, the maximum compressive strain at the bottom during Middle Path loading was found from the calibrated FEM to be 3.80 microstrain ($\mu\varepsilon$) at the west end and 3.65 microstrain ($\mu\varepsilon$) at the east end. This microstrain can be converted to a stress (σ) value using Hooke's law:

$$\begin{aligned} \sigma_1 &= \varepsilon E \\ &= (3.80)(10^{-6})(3834\text{ ksi}) \\ &= 0.015\text{ ksi} \end{aligned} \tag{7.33}$$

$$\begin{aligned} \sigma_2 &= \varepsilon E \\ &= (3.65)(10^{-6})(3834\text{ ksi}) \\ &= 0.014\text{ ksi} \end{aligned} \tag{7.34}$$

This stress value can be converted to a moment value, giving the restraining moment $M_{end-fix}$:

$$\begin{aligned} M_{end-fix1} &= \sigma \frac{I_x}{y} \\ &= (0.015\text{ ksi})\left(\frac{17{,}625\text{ in}^4}{15.65\text{ in.}}\right) \\ &= 1.37\text{ kip-ft} \end{aligned} \tag{7.35}$$

$$\begin{aligned} M_{end-fix2} &= \sigma \frac{I_x}{y} \\ &= (0.014\text{ ksi})\left(\frac{17{,}625\text{ in}^4}{15.65\text{ in.}}\right) \\ &= 1.31\text{ kip-ft} \end{aligned} \tag{7.36}$$

Therefore, the new applied midspan live load moment considering the observed partial end restraint is:

$$\begin{aligned} M_{LL} &= M_{LL-simple} - \frac{(M_{end1} + M_{end2})}{2} \\ &= 86.8 - \frac{1.37 + 1.31}{2} = 85.5\text{ kip–ft} \end{aligned} \tag{7.37}$$

7.4.4.3 Dead Load Moment of Structural Components

Detailed in Section 7.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 45.1 \text{ kip–ft}$$

7.4.4.4 Superimposed Dead Load Moment

Detailed in Section 7.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 17.0 \text{ kip–ft}$$

7.4.4.5 Total Dead Load Moment

Detailed in Section 7.1.4.4, the total applied moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$M_{DL} = 62.1 \text{ kip–ft}$$

7.4.5 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{7.38}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{158.9 \; - [(1.3)(62.1)]}{(2.17)(85.5)} \\ &= 0.42 \end{aligned} \tag{7.39}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$RF_O = \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \tag{7.40}$$

$$= \frac{158.9 \ - [(1.3)(62.1)]}{(1.3)(85.5)}$$

$$= 0.70$$

Passes.

LFR rating factors for Bridge CM-5 from the refined load rating analysis using Level II partial end restraint analysis are equal to:

$$RF_I = 0.42$$

$$RF_O = 0.70$$

The basic load rating controlling RFs are 0.42 for Inventory and 0.69 for Operating. The new rating factors represent a 0.0 percent increase for Inventory and a 1.4 percent increase for Operating. Table 7.3 compares the controlling RFs determined using Level II partial end restraint analysis to the controlling RFs determined in the basic load rating.

Table 7.3. Partial End Restraint RF Comparison

Rating Factor	Basic Load Rating	Load Rating with End Restraint	End Restraint/Basic Load Rating
Inventory	0.42	0.42	1.00
Operating	0.69	0.70	1.01

7.5 LOAD RATING ANALYSIS CONSIDERING ONLY FEM LIVE LOAD MOMENTS

This section shows an abbreviated version of a load rating performed for Bridge CM-5 when considering finite element method (FEM) live load moments. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). The results of this load rating are compared to the results of the basic load rating analysis.

7.5.1 Bridge Characteristics

Bridge CM-5 is a two-lane bridge with a total length of 30 ft. The bridge was designed as simply supported, and has a controlling span for load rating of 29 ft. The bridge width is 21 ft 8 in. with a roadway width of 21 ft. The TxDOT HS-20 RFs are 0.72 for Inventory and 1.00 for Operating. The transverse section of Bridge CM-5 is shown in Figure 7.5.

Span Length: $L = 29\ \mathrm{ft}$	Bridge Width: $W = \ 21\ \mathrm{ft}\ 8\ \mathrm{in.}$
Concrete Compressive Strength: $f'_c = 4.0\ \mathrm{ksi}$	Girder Spacing: $S = 3\ \mathrm{ft}$
Steel yield strength: $f_y = 33\ \mathrm{ksi}$	Number of Girders: $N_G = 8$
Concrete Density: $\gamma_c = 150\ \mathrm{pcf}$	Deck Thickness: $t_d = 4.5\ \mathrm{in.}$
Asphalt Density: $\gamma_{ws} = 144\ \mathrm{pcf}$	Asphalt Thickness: $t_{ws} = 4.5\ \mathrm{in.}$
Concrete Girder Section: 24 in. deep, 7 in. wide web	Girder Linear Weight: $w_{SG} = 429\ \frac{\mathrm{lb}}{\mathrm{ft}}$

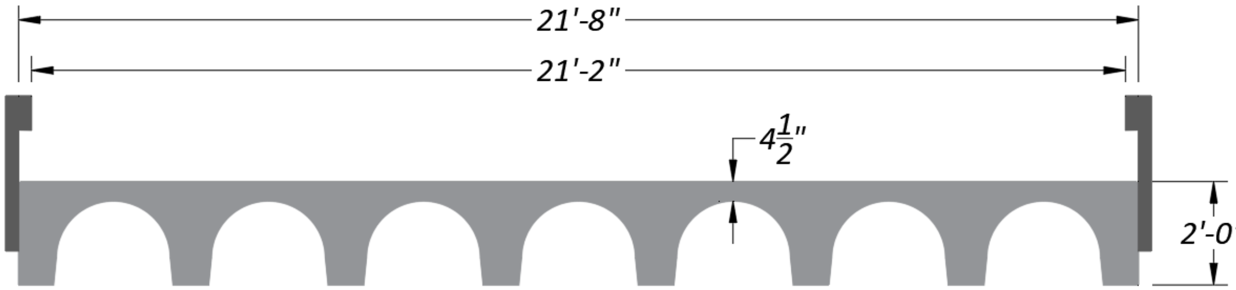

Figure 7.5. Transverse Section of Bridge CM-5 (TxDOT 2018a)

7.5.2 Sectional Properties

Ground Penetrating Radar (GPR) was used to verify the reinforcement locations within the girder sections. A single layer of longitudinal reinforcement was found located at 3 in. from the bottom of the girder and another layer of reinforcement was found at 21 in. from the bottom of the girder. The area of tension reinforcement is assumed to match that of one layer of steel (2 - #11 bars) in TxDOT's standard drawings for this bridge system (TxDOT 2005). The top layer of steel is neglected. The girder web thickness given is the width at the bottom of the web.

Girder Web Thickness (bottom): $t_w = 7.0\text{ in.}$ Total Girder Depth: $h = 24.0\text{ in.}$

Assumed Tension Reinf. Area: $A_s = 3.12\text{ in}^2$ Tension Reinforcement Depth: $d = 21.0\text{ in.}$

7.5.3 Moment Capacity

Detailed in Section 5.1.3, the final reduced moment capacity ϕM_n for an individual girder was determined to be

$$\phi M_n = 158.9\text{ kip–ft}$$

7.5.4 Structural Analysis for Moment Demand

7.5.4.1 Applied Live Load Moment

The Impact Factor I is applied to the live load to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to,

$$I = \frac{50}{L+125} \leq 0.3 \quad (7.41)$$

$$= \frac{50}{29+125} \leq 0.3$$

$$= 0.325 \leq 0.3$$

$$I = 0.3$$

The applied live load moment on an individual interior girder without the impact factor M_{HS20} is obtained from the original FEM model as:

$$M_{HS20} = 77.7 \text{ kip–ft}$$

Therefore, the distributed applied live load moment M_{LL} with dynamic effects, on an individual interior girder can be calculated as:

$$M_{LL} = M_{HS20}(1+I) \quad (7.42)$$

$$= (77.7 \text{ kip–ft})(1.3)$$

$$= 101.0 \text{ kip–ft}$$

7.5.4.2 Dead Load Moment of Structural Components

Detailed in Section 7.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 45.1 \text{ kip–ft}$$

7.5.4.3 Superimposed Dead Load Moment

Detailed in Section 7.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 17.0 \text{ kip–ft}$$

7.5.4.4 Total Dead Load Moment

Detailed in Section 7.1.4.4, the total applied moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$M_{DL} = 62.1 \text{ kip–ft}$$

7.5.5 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{7.43}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \qquad (7.44) \\ &= \frac{158.9 - [(1.3)(62.1)]}{(2.17)(101.0)} \\ &= 0.36 \end{aligned}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$\begin{aligned} RF_O &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \qquad (7.45) \\ &= \frac{158.9 - [(1.3)(62.1)]}{(1.3)(101.0)} \\ &= 0.60 \end{aligned}$$

Does not pass.

7.5.6 Controlling Rating Factors

Therefore, the controlling LFR rating factors for Bridge CM-5 from the developed FEM model live load are equal to:

$$RF_I = 0.36$$

$$RF_O = 0.60$$

The basic load rating controlling RFs are 0.42 for Inventory and 0.69 for Operating. The new rating factors represent a 14.3 percent decrease for Inventory and a 13.0 percent decrease for Operating. Table 7.4 compares the flexural strength RFs determined using the FEM model live load moments to the flexural strength RFs determined in the basic load rating.

Table 7.4. FEM Live Load RF Comparison

Rating Factor	Basic Load Rating	Load Rating with FEM Live Load	FEM Live Load /Basic Load Rating
Inventory	0.42	0.36	0.86
Operating	0.69	0.60	0.87

7.6 LOAD RATING ANALYSIS CONSIDERING CALIBRATED FEM LIVE LOAD MOMENTS

This example shows an abbreviated version of a load rating analysis performed for Bridge CM-5 when considering measured material properties and calibrated FEM live load moments. This load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). The results of this load rating are compared to the results of the basic load rating analysis.

7.6.1 Bridge Characteristics

Bridge CM-5 is a two-lane bridge with a total length of 30 ft. The bridge was designed as simply supported, and has a controlling span for load rating of 29 ft. The bridge width is 21 ft 8 in. with a roadway width of 21 ft. The TxDOT HS-20 RFs are 0.72 for Inventory and 1.00 for Operating. The transverse section of Bridge CM-5 is shown in Figure 7.6.

Span Length: $L = 29\ \mathrm{ft}$	Bridge Width: $W = 21\ \mathrm{ft}\ 8\ \mathrm{in.}$
Concrete Compressive Strength: $f'_c = 7.0\ \mathrm{ksi}$	Girder Spacing: $S = 3\ \mathrm{ft}$
Steel yield strength: $f_y = 33\ \mathrm{ksi}$	Number of Girders: $N_G = 8$
Concrete Density: $\gamma_c = 150\ \mathrm{pcf}$	Deck Thickness: $t_d = 4.5\ \mathrm{in.}$
Asphalt Density: $\gamma_{ws} = 144\ \mathrm{pcf}$	Asphalt Thickness: $t_{ws} = 4.5\ \mathrm{in.}$
Concrete Girder Section: 24 in. deep, 7 in. wide web	Girder Linear Weight: $w_{SG} = 429\ \frac{\mathrm{lb}}{\mathrm{ft}}$

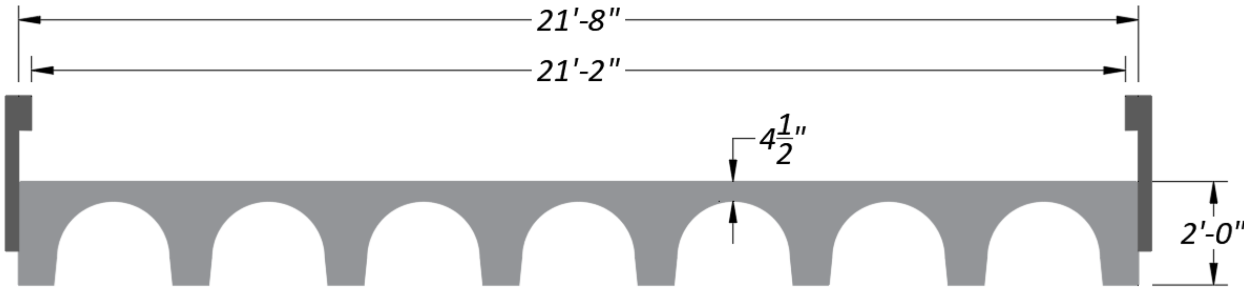

Figure 7.6. Transverse Section of Bridge CM-5 (TxDOT 2018a)

7.6.2 Sectional Properties

Ground Penetrating Radar (GPR) was used to verify the reinforcement locations within the girder sections. A single layer of longitudinal reinforcement was found located at 3 in. from the bottom of the girder and another layer of reinforcement was found at 21 in. from the bottom of the girder. The area of tension reinforcement is assumed to match that of one layer of steel (2 - #11 bars) in TxDOT's standard drawings for this bridge system (TxDOT 2005). The top layer of steel is neglected. The girder web thickness given is the width at the bottom of the web.

Girder Web Thickness (bottom): $t_w = 7.0\text{ in.}$	Total Girder Depth: $h = 24.0\text{ in.}$
Tension Reinf. Area: $A_s = 3.12\text{ in}^2$	Tension Reinforcement Depth: $d = 21.0\text{ in.}$

7.6.3 Moment Capacity

Detailed in Section 7.3.3, final reduced moment capacity ϕM_n for an individual girder was determined to be

$$\phi M_n \quad = \quad 160.3\text{ kip–ft}$$

7.6.4 Structural Analysis for Moment Demand

7.6.4.1 Applied Live Load Moment

The Impact Factor I is applied to the live load to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the Impact Factor is equal to,

$$
\begin{aligned}
I &= \frac{50}{L+125} \leq 0.3 \\
&= \frac{50}{29+125} \leq 0.3 \\
&= 0.325 \leq 0.3 \\
I &= 0.3
\end{aligned} \tag{7.46}
$$

The applied live load moment on an individual interior girder without the impact factor M_{HS20} is obtained from the calibrating FEM model as:

$$M_{HS20} = 70.1 \text{ kip–ft}$$

Therefore, the distributed applied live load moment M_{LL} with dynamic effects, on an individual interior girder can be calculated as:

$$
\begin{aligned}
M_{LL} &= M_{HS20}(1+I) \\
&= (70.1 \text{ kip–ft})(1.3) \\
&= 91.13 \text{ kip–ft}
\end{aligned} \tag{7.47}
$$

7.6.4.2 Dead Load Moment of Structural Components

Detailed in Section 7.1.4.2, the applied moment due to dead load of structural components M_{DC} is:

$$M_{DC} = 45.1 \text{ kip–ft}$$

7.6.4.3 Superimposed Dead Load Moment

Detailed in Section 7.1.4.3, the applied moment due to superimposed dead load M_{SDL} is:

$$M_{SDL} = 17.0 \text{ kip–ft}$$

7.6.4.4 Total Dead Load Moment

Detailed in Section 7.1.4.4, the total applied moment due to structural dead loads and superimposed dead loads M_{DL} is equal to:

$$M_{DL} = 62.1 \text{ kip–ft}$$

7.6.5 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{7.48}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

$$\begin{aligned} RF_I &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{160.3 - [(1.3)(62.1)]}{(2.17)(91.1)} \\ &= 0.40 \end{aligned} \tag{7.49}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

$$\begin{aligned} RF_O &= \frac{\phi M_n - A_1 M_{DL}}{A_2 M_{LL}} \\ &= \frac{160.3 - [(1.3)(62.1)]}{(1.3)(91.1)} \\ &= 0.67 \end{aligned} \tag{7.50}$$

Does not pass.

7.6.6 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge CM-5 from the calibrated FEM model live load are equal to:

$$RF_I = 0.40$$

$$RF_O = 0.67$$

The basic load rating controlling RFs are 0.42 for Inventory and 0.69 for Operating. The new rating factors represent a 4.8 percent decrease for Inventory and a 2.9 percent decrease for Operating. Table 7.5 compares the controlling RFs determined using the calibrated FEM live load moment to the controlling RFs determined in the basic load rating.

Table 7.5. Calibrated FEM Live Load RF Comparison

Rating Factor	Basic Load Rating	Load Rating with Calibrated FEM Live Load	Calibrated FEM Live Load /Basic Load Rating
Inventory	0.42	0.40	0.95
Operating	0.69	0.67	0.97

8 FLEXURAL LOAD RATING ANALYSIS OF BRIDGE CS-9 USING THE LFR METHOD

8.1 BASIC LOAD RATING ANALYSIS

This section shows the initial basic load rating analysis performed for Bridge CS-9. This basic load rating analysis was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). This initial basic load rating was performed under the assumption that the bridge ends are simply supported. The resulting rating factors are used for comparison when conducting load ratings assuming partial end restraint, updated material strengths, or a combination of the two.

8.1.1 Bridge Characteristics

Bridge CS-9 is a two-lane bridge with a total length of 75 ft. The bridge was designed to include three simply supported spans with a controlling span for load rating of 25 ft. The bridge width is 21 ft 4 in. with a roadway width of 20 ft. The TxDOT HS-20 RFs are 0.445 for Inventory and 0.935 for Operating. The transverse section of Bridge CS-9 is shown in Figure 8.1.

Span Length: $L = 24 \text{ ft}$

Bridge Width: $W = 21 \text{ ft } 4 \text{ in.}$

Concrete Compressive Strength: $f'_c = 2.5 \text{ ksi}$

Steel yield strength: $f_y = 33 \text{ ksi}$

Concrete Density: $\gamma_c = 150 \text{ pcf}$

Asphalt Density: $\gamma_{ws} = 144 \text{ pcf}$

Asphalt Thickness: $t_{ws} = 3 \text{ in.}$

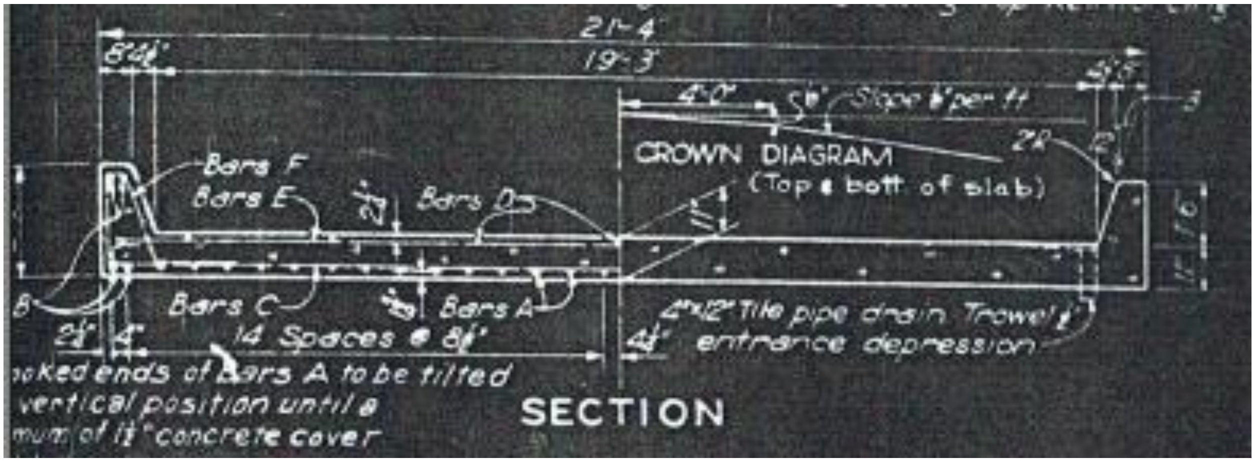

Figure 8.1. Transverse Section of Bridge CS-9 (TxDOT 2018a)

8.1.2 Properties of Concrete Slab Section

Design Slab Width: $b = 12.0$ in.

Tensile Reinforcement Diameter: $d_b = 1.0$ in.

Tensile Reinforcement Area: $A_s = 1.115\ \frac{\text{in}^2}{\text{ft}}$

Slab Depth: $t_{slab} = 11.0$ in.

Reinforcement Spacing: $s = 8.5$ in.

Bottom Cover: $c_{bot} = 1.75$ in.

8.1.3 Properties of Concrete Curb

Top width: $W_{top} = 8.00$ in.

Tension Reinforcement Dimension: 1.25 in. (sq.)

Tension Steel Area: $A_{s-curb} = 3.125\ \text{in}^2$

Compression Reinf. Dimension: 1.25 in. (sq.)

Compression Steel Area: $A'_{s-curb} = 3.125\ \text{in}^2$

Curb height: $h = 18.00$ in.

Bottom width: $W_{bot} = 12.50$ in.

No. of Reinforcing Bars: $N_b = 2$

Bottom Cover: $c_{bot-curb} = 1.75$ in.

No. of Reinforcing Bars: $N_b = 2$

Top Cover: $c_{top-curb} = 2.25$ in.

8.1.4 Moment Capacity

The procedure outlined in Article 8.16 of the AASHTO Standard Specifications (AASHTO 2002) is followed to determine the moment capacity for a unit width of slab. First, it is assumed that the tensile reinforcement yields and this assumption is verified.

8.1.4.1 Slab Capacity

Calculate the flexural capacity per unit width of slab according to AASHTO Standard Specifications Article 8.16. Ultimate strain in concrete, $\varepsilon_{cu} = 0.003$.

The stress block factor for 2.5 ksi concrete as,

$$0.65 \leq \beta_1 = 0.85 - \left(0.05(f_c{}' - 4)\right) \leq 0.85 \tag{8.1}$$

$$\beta_1 = 0.85$$

The depth of the equivalent stress block,

$$\begin{aligned} a &= \frac{A_s f_y}{0.85 f_c' b} \\ &= \frac{\left(1.115 \frac{\text{in}^2}{\text{ft}}\right)(33\text{ ksi})}{(0.85)(2.5\text{ ksi})(12\text{ in.})} \\ &= 1.443\text{ in.} \end{aligned} \tag{8.2}$$

The distance from extreme compression fiber to the neutral axis,

$$\begin{aligned} \mathrm{c} &= \frac{a}{\beta_1} \\ &= \frac{1.443\text{ in.}}{0.85} \\ &= 1.698\text{ in.} \end{aligned} \tag{8.3}$$

Check if steel has yielded:

$$\begin{aligned} \varepsilon_s &= \varepsilon_{cu}\left(\frac{d-c}{c}\right) && \geq \frac{f_y}{E_s} \\ &= 0.003\left(\frac{(9.25\text{ in.} - 1.698\text{ in.}}{1.698\text{ in.}}\right) && \geq \frac{33\text{ ksi}}{29{,}000\text{ ksi}} \\ &= 0.0133 && \geq 0.00114\text{ (Steel yields)} \end{aligned} \tag{8.4}$$

Flexural reduction factor, $\phi = 0.90$. Nominal moment capacity of the slab section may be calculated as:

$$\begin{aligned}\phi M_n &= \phi A_s f_y \left(d - \frac{a}{2}\right) && (8.5)\\ &= (0.90)\left(1.115\,\frac{\text{in}^2}{\text{ft}}\right)(33\text{ ksi})\left(9.25\text{ in.} - \frac{1.443\text{ in.}}{2}\right)\\ &= 23.5\,\frac{\text{kip-ft}}{\text{ft}}\end{aligned}$$

8.1.4.2 Curb Capacity

The slope of the trapezoidal curb sections is calculated as:

$$\begin{aligned}C_{slope} &= \frac{W_{bot} - W_{top}}{h} && (8.6)\\ &= \frac{12.5\text{ in.} - 8\text{ in.}}{18\text{ in.}}\\ &= 0.25\end{aligned}$$

The effective area of reinforcement from the slab section $A_{s,eff}$ is calculated for a width of $4t_{slab}$ as:

$$\begin{aligned}A_{s,eff} &= A_s(4t_{slab}) && (8.7)\\ &= \left(1.115\,\frac{\text{in}^2}{\text{ft}}\right)\left(4(11\text{ in.})\right)\left(\frac{1\text{ ft}}{12\text{ in.}}\right)\\ &= 4.09\text{ in}^2\end{aligned}$$

Effective depth of the curb section may be calculated as:

$$\begin{aligned}d_{curb} &= h + t_{slab} - c_{bot-curb} && (8.8)\\ &= 18\text{ in.} + 11\text{ in.} - 1.75\text{ in.}\\ &= 27.25\text{ in.}\end{aligned}$$

Article 8.16 of the AASHTO Standard Specifications suggests the following equation to check if the compression steel has yielded for sections with compressive reinforcement. The equation was developed for rectangular webs, when applying to the trapezoidal curb section it is conservative to use the minimum width at the top of the curb.

$$\left(\frac{A_{s-curb} + A_{s,eff} - A'_{s-curb}}{W_{top} d_{curb}}\right) \geq 0.85\beta_1 \left(\frac{f_c' c_{ccurb}}{f_y d_{curb}}\right)\left(\frac{87}{87 - f_y}\right) \tag{8.9}$$

$$\left(\frac{3.125 \text{ in}^2 + 4.09 \text{ in}^2 - 3.125 \text{ in}^2}{(8 \text{ in.})(27.25 \text{ in.})}\right) \geq 0.85(0.85)\left(\frac{(2.5 \text{ ksi})(2.25 \text{ in.})}{(33 \text{ ksi})(27.25 \text{ in.})}\right)\left(\frac{87}{87 - 33 \text{ ksi}}\right)$$

$$0.0855 \geq 0.00728 \text{ (compression steel yields)}$$

The total tensile reinforcement $A_{s,total}$ in the L-curb section is calculated as:

$$\begin{aligned} A_{s,total} &= A_{s,eff} + A_{s-curb} \\ &= 4.09 \text{ in}^2 + 3.125 \text{ in}^2 \\ &= 7.215 \text{ in}^2 \end{aligned} \tag{8.10}$$

Depth of stress block in the curb can be calculated as:

$$\begin{aligned} (A_{s,total} - A'_{s-curb})f_y &= 0.85 f_c' W_{top} a_{curb} + \frac{0.85 f_c' C_{slope}}{2} {a_{curb}}^2 && \leq h \\ (7.215 \text{ in}^2 - 3.125 \text{ in}^2)(33 \text{ ksi}) &= 0.85(2.5 \text{ ksi})(8 \text{ in.})(a_{curb}) + \frac{0.85(2.5 \text{ ksi})(0.25)}{2} {a_{curb}}^2 && \leq 18 \text{ in.} \\ a_{curb} &= 7.14 \text{ in.} && \leq 18 \text{ in.} \end{aligned} \tag{8.11}$$

The distance from top of curb to the centroid of the stress block is calculated as:

$$\begin{aligned} y_{curb} &= \frac{3 a_{curb} W_{top} + 2 C_{slope} {a_{curb}}^2}{6 W_{top} + 3 C_{slope} a_{curb}} \\ &= \frac{3(7.14 \text{ in.})(8 \text{ in.}) + 2(0.25)(7.14 \text{ in.})^2}{6(8 \text{ in.}) + 3(0.25)(7.14 \text{ in.})} \\ &= 3.69 \text{ in.} \end{aligned} \tag{8.12}$$

The total nominal moment of the curb section is calculated by adding the nominal moment capacity of the two components as:

$$\begin{aligned}\phi M_{n,L-curb} &= \phi\left(A_{s,curb} + A_{s,eff} - A'_{s,curb}\right)(f_y)(d_{curb} - y_{curb}) \\ &\quad +\phi A'_{s,curb}(f_y - 0.85f'_c)\left(d_{curb} - c_{top-curb}\right) \\ &= (0.90)(3.125\text{ in}^2 + 4.09\text{ in}^2 - 3.125\text{ in}^2)(33\text{ ksi}) \\ &\quad (27.25\text{ in.} - 3.69\text{ in.}) + (0.90)(3.125\text{ in}^2) \\ &\quad \left(33\text{ ksi} - 0.85(2.5\text{ ksi})\right)(27.25\text{ in.} - 2.25\text{ in.}) \\ &= 419.4\text{ kip–ft}\end{aligned} \tag{8.13}$$

8.1.5 Structural Analysis for Moment Demand

8.1.5.1 Applied Live Load Moment

For LFR, use the Illinois Bulletin 346 (Jenson et al. 1943) approach to determine the distribution of live load across the bridge. The triangular region and rectangular region used to calculate the moment of inertia of the curb are shown in Figure 8.2.

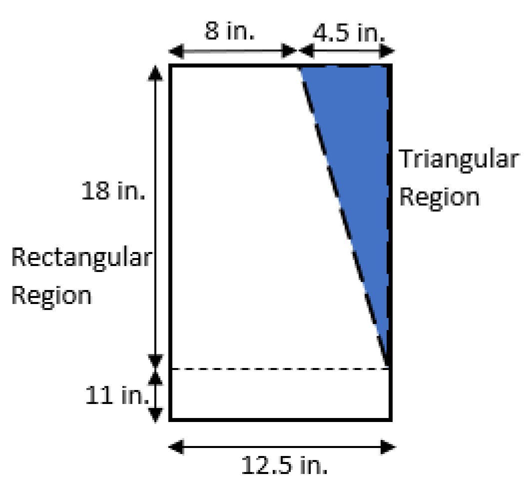

Figure 8.2. Curb Section of Bridge CS-9

The moment of inertia for the curb sections are calculated as:

Area of rectangular section, A_{rect}

$$A_{rect} = W_{bot}(h + t_{slab}) \tag{8.14}$$
$$= (12.5 \text{ in.})(18 \text{ in.} + 11 \text{ in.})$$
$$= 362.5 \text{ in}^2$$

Centroid of rectangular section, y_{rect}

$$y_{rect} = \frac{h + t_{slab}}{2} \tag{8.15}$$
$$= \frac{(18 \text{ in.} + 11 \text{ in.})}{2}$$
$$= 14.5 \text{ in.}$$

Moment of inertia of rectangular section, I_{rect}

$$I_{rect} = W_{bot}\frac{(h + t_{slab})^3}{12} \tag{8.16}$$
$$= (12.5 \text{ in.})\frac{(18 \text{ in.} + 11 \text{ in.})^3}{12}$$
$$= 25{,}410 \text{ in}^4$$

Area of triangular section, A_{tri}

$$A_{tri} = (W_{bot} - W_{top})\frac{h}{2} \tag{8.17}$$
$$= (12.5 \text{ in.} - 8 \text{ in.})\frac{18 \text{ in.}}{2}$$
$$= 40.5 \text{ in}^2$$

Centroid of triangular section, y_{tri}

$$y_{tri} = t_{slab} + \frac{2}{3}h \tag{8.18}$$
$$= 11 \text{ in.} + \frac{2}{3}(18 \text{ in.})$$
$$= 23 \text{ in.}$$

Moment of inertia of triangular section, I_{tri}

$$I_{tri} = (W_{bot} - W_{top})\frac{h^3}{36} \tag{8.19}$$
$$= (12.5 \text{ in.} - 8 \text{ in.})\frac{(18 \text{ in.})^3}{36}$$
$$= 729 \text{ in}^4$$

Area of curb, A_{curb}

$$A_{curb} = A_{rect} - A_{tri} \tag{8.20}$$
$$= 362.5 \text{ in}^2 - 40.5 \text{ in}^2$$
$$= 322 \text{ in}^2$$

$$\text{Centroid of curb, } y_{curb} = \frac{(A_{rect}y_{rect} - A_{tri}y_{tri})}{A_{curb}} \tag{8.21}$$

$$= \frac{((362.5\text{ in}^2)(14.5\text{ in.}))((40.5\text{ in}^2)(23\text{ in.}))}{322\text{ in}^2}$$

$$= 13.4\text{ in.}$$

$$\text{Moment of inertia of curb, } I_{Curb} = (I_{rect} + (A_{rect}(y_{rect} - y_{curb})^2)) - (I_{tri} + (A_{tri}(y_{tri} - y_{curb})^2)) \tag{8.22}$$

$$= \left((25{,}410\text{ in}^4) + (362.5\text{ in}^2)(14.5\text{ in.} - 13.4\text{ in.})^2\right) - \left((729\text{ in}^4) + (40.5\text{ in}^2)(23\text{ in.} - 13.4\text{ in.})^2\right)$$

$$= 21{,}380\text{ in}^4$$

The following factors for the curb sections are calculated based on the guidelines provided in IB346. The axle spacing of the test truck is $v = 6.0$ ft and the clear span of the bridge is $a = 24$ ft.

$$\text{Dimensionless stiffness factor, } G = L\frac{{t_{slab}}^3}{12I_{Curb}} \tag{8.23}$$

$$= (288\text{ in.})\frac{(11\text{ in.})^3}{(12)(21{,}380\text{ in}^4)}$$

$$= 1.49$$

$$\text{Dimensionless coefficient, } C_1 = \left(\frac{12}{2.5+G}\right)\left(\frac{\left(4-\frac{v}{a}\right)}{4+28\left(\frac{v}{a}\right)}\right) \tag{8.24}$$

$$= \left(\frac{12}{2.5+1.49}\right)\left(\frac{\left(4-\frac{6\text{ ft}}{24\text{ ft}}\right)}{4+28\left(\frac{6\text{ ft}}{24\text{ ft}}\right)}\right)$$

$$= 1.025$$

$$\text{Dimensionless coefficient, } C_2 = \frac{0.5\frac{L}{(W-2W_{bot})}}{0.47G + \sqrt[3]{1.15+\left(\frac{L}{(W-2W_{bot})}\right)^3}} \tag{8.25}$$

$$= \frac{0.5\frac{24\text{ ft}}{((21.33\text{ ft})-2(1.04\text{ ft}))}}{(0.47)(1.49)+\sqrt[3]{1.15+\left(\frac{24\text{ ft}}{((21.33\text{ ft})-2(1.04\text{ ft}))}\right)^3}}$$

$$= 0.289$$

Dimensionless coefficient, C_3

$$= \frac{\sqrt[3]{1.15+\left(\frac{L}{(W-2W_{bot})}\right)^3}}{0.47G+\sqrt[3]{1.15+\left(\frac{L}{(W-2W_{bot})}\right)^3}} \qquad (8.26)$$

$$= \frac{\sqrt[3]{1.15+\left(\frac{24\text{ ft}}{((21.33\text{ ft})-2(1.04\text{ ft}))}\right)^3}}{(0.47)(1.49)+\sqrt[3]{1.15+\left(\frac{24\text{ ft}}{((21.33\text{ ft})-2(1.04\text{ ft}))}\right)^3}}$$

$$= 0.675$$

The Impact Factor I is applied to the live load to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the impact factor is equal to,

$$I = \frac{50}{L+125} \leq 0.3 \qquad (8.27)$$

$$= \frac{50}{24+125} \leq 0.3$$

$$= 0.336 \leq 0.3$$

$$I = 0.3$$

The applied live load moment for wheel load without the impact factor M_{HS20} is obtained from the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1:

$$M_{HS20} = 96.3\ \frac{\text{kip–ft}}{\text{wheel line}}$$

The equivalent force P_{eq} due to applied live load moment is calculated as,

$$\begin{aligned} P_{eq} &= \frac{4M_{HS20}}{L} \\ &= \frac{4(96.3\text{ kip-ft})}{24\text{ ft}} \\ &= 16.1\text{ kips} \end{aligned} \tag{8.28}$$

The share of the live load moment per unit width taken by the concrete slab is calculated as:

$$\begin{aligned} M_{Slab,LL} &= (2(N_L - 0.75C_1))\frac{P_{eq}L}{4(W - 2W_{bot})} \\ &= \left(2\left(2-(0.75)(1.025)\right)\right)\frac{(16.1\text{ kips})(24\text{ ft})}{4\left((21.33\text{ ft}) - 2(1.04\text{ ft})\right)} \\ &= 12.4\frac{\text{kip-ft}}{\text{ft}} \end{aligned} \tag{8.29}$$

The portion of the live load moment acting on the composite L-curb is calculated as:

$$\begin{aligned} M_{LL,L-curb} &= \left(C_1\frac{P_{eq}L}{4}\right) + \left(4t_s M_{Slab,LL}\right) \\ &= \left((1.025)\frac{(16.1\text{ kips})(24\text{ ft})}{4}\right) + \left(4(0.917\text{ ft})\left(12.4\ \frac{\text{kip-ft}}{\text{ft}}\right)\right) \\ &= 144.5\text{ kip-ft} \end{aligned} \tag{8.30}$$

Therefore, the applied live load moment with dynamic effects for the slab and curb can be calculated as:

$$\begin{aligned} M_{Slab,LL} &= M_{Slab,LL}(1+I) \\ &= (12.4)(1.3) \\ &= 16.1\ \frac{\text{kip-ft}}{\text{ft}} \end{aligned} \tag{8.31}$$

$$\begin{aligned} M_{LL,L-curb} &= M_{LL,L-curb}(1+I) \\ &= (144.5)(1.3) \\ &= 187.8\text{ kip-ft} \end{aligned} \tag{8.32}$$

8.1.5.2 Dead Load Moment of Structural Components

The slab self-weight w_S is,

$$\begin{aligned} w_S &= \gamma_C t_{Slab} b \\ &= (150 \text{ pcf}) \left(\frac{11 \text{ in.}}{12 \frac{\text{in.}}{\text{ft}}} \right) (1 \text{ ft}) \\ &= 0.167 \ \frac{\text{kip}}{\text{ft}} \text{ per ft width of slab} \end{aligned}$$

The self-weight of the curbs w_{curb} including the slab directly below the curb is determined per the guidelines provided by IB346:

$$\begin{aligned} w_{curb} &= \left(\frac{(W_{top} + W_{bot})}{2} h + W_{bot} t_{slab} \right) \gamma_C \qquad (8.33) \\ &= \left(\frac{(8 \text{ in.} + 12.5 \text{ in.})}{2} \left(\frac{18 \text{ in.}}{12 \frac{\text{in.}}{\text{ft}}} \right) + \left(\frac{12.5 \text{ in.}}{12 \frac{\text{in.}}{\text{ft}}} \right) \left(\frac{11 \text{ in.}}{12 \frac{\text{in.}}{\text{ft}}} \right) \right) (0.15 \text{kcf}) \\ &= 0.335 \ \frac{\text{kip}}{\text{ft}} \end{aligned}$$

8.1.5.3 Superimposed Dead Load Moment

The superimposed dead load per unit width due to the wearing surface w_{SDL} can be calculated as,

$$\begin{aligned} w_{SDL} &= \gamma_{ws} t_{ws} b \qquad (8.34) \\ &= (0.144 \text{ kcf}) \left(\frac{3 \text{ in.}}{12 \frac{\text{in.}}{\text{ft}}} \right) (1 \text{ ft}) \\ &= 0.036 \ \frac{\text{kip}}{\text{ft}} \text{ per ft width of slab} \end{aligned}$$

8.1.5.4 Total Dead Load Moment

Therefore, the total applied moment due to structural dead loads and superimposed dead loads $M_{DL,Slab}$ per ft width for the slab is,

$$\begin{aligned} M_{DL,Slab} &= (1-2C_2)\frac{(w_S+w_{SDL})L^2}{8} \\ &\quad +2(1-C_3)\frac{w_{Curb}L^2}{8(W-2W_{bot})} \\ &= (1-2(0.289))\frac{\left(0.167\ \frac{\text{kip}}{\text{ft}}+0.036\ \frac{\text{kip}}{\text{ft}}\right)(24\text{ ft})^2}{8} \\ &\quad +2(1-0.675)\frac{\left(0.335\ \frac{\text{kip}}{\text{ft}}\right)(24\text{ ft})^2}{8\big((21.33\text{ ft})-2(1.04\text{ ft})\big)} \\ &= 6.1\ \frac{\text{kip-ft}}{\text{ft}} \end{aligned} \tag{8.35}$$

Therefore, the total applied moment due to structural dead loads and superimposed dead loads $M_{DL,L-curb}$ for the L-curb is,

$$\begin{aligned} M_{DL,L-curb} &= C_2(W-2W_{bot})\frac{(w_S+w_{SDL})L^2}{8}+C_3\frac{w_{Curb}L^2}{8} \\ &\quad +\big(4t_sM_{Slab,DL}\big) \\ &= (0.289)\big(21.33\text{ ft})-2(1.04\text{ ft})\big) \\ &\quad \frac{\left(0.167\ \frac{\text{kip}}{\text{ft}}+0.036\ \frac{\text{kip}}{\text{ft}}\right)(24\text{ ft})^2}{8} \\ &\quad +(0.675)\frac{\left(0.335\ \frac{\text{kip}}{\text{ft}}\right)(24\text{ ft})^2}{8}+\left(4(0.917\text{ ft})\left(6.1\ \frac{\text{kip-ft}}{\text{ft}}\right)\right) \\ &= 108.0\text{ kip–ft} \end{aligned} \tag{8.36}$$

8.1.6 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF=\frac{C-A_1D}{A_2(L+I)} \tag{8.37}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

Slab Rating:

$$RF_{ISlab} = \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \tag{8.38}$$

$$= \frac{23.5 - [(1.3)(6.1)]}{(2.17)(16.1\ )}$$

$$= 0.45$$

Curb Rating:

$$RF_{IL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \tag{8.39}$$

$$= \frac{419.4 - [(1.3)(108.0)]}{(2.17)(187.8\ )}$$

$$= 0.68$$

The inventory rating for Bridge CS-9 is the minimum of the two RFs determined for flexural strength of the slab and L-curb (TxDOT 2018b).

$$RF_I = Min(RF_{ISlab}, RF_{IL-curb}) \tag{8.40}$$

$$= Min(0.45, 0.68)$$

$$= 0.45$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

Slab Rating:

$$RF_{OSlab} = \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \tag{8.41}$$

$$= \frac{23.5 - [(1.3)(6.1)]}{(1.3)(16.1\ )}$$

$$= 0.74$$

Curb Rating:

$$RF_{OL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \tag{8.42}$$

$$= \frac{419.4 - [(1.3)(108.0)]}{(1.3)(187.8)}$$

$$= 1.14$$

TxDOT considers the operating rating for Bridge CS-9 as the weighted average of the L-curb and slab rating proportionate to their tributary width (TxDOT 2018b):

$$RF_O = \frac{2\left(RF_{OL-curb}(W_{bot} + 4h)\right) + \left(RF_{OSlab}\left(W - 2(W_{bot} + 4h)\right)\right)}{W} \tag{8.43}$$

$$= \frac{2\left(1.14\left(12.5 + 4(11)\right)\right) + \left(0.74\left(255.6 - 2\left(12.5 + 4(11)\right)\right)\right)}{255.6}$$

$$= 0.92$$

Does not pass.

The controlling LFR flexural strength rating factors for Bridge CS-9 using the basic load rating analysis, along with the IB346 methodology and TxDOT practices, are equal to:

$$RF_I = 0.45$$

$$RF_O = 0.92$$

Note that the above values are very close to those reported by TxDOT (0.45 for Inventory and 0.95 for Operating). For both inventory and operating ratings, the RFs are less than 1.0, and therefore do not pass these load rating according the AASHTO MBE.

8.2 LOAD RATING ANALYSIS USING AASHTO LRFD EQUIVALENT WIDTH APPROACH FOR MID-SLAB REGION

This section shows an alternative load rating analysis performed for Bridge CS-9 using the equivalent width formula in the *AASHTO LRFD Specifications* (AASHTO 2017) for estimating the live load distribution to the mid-slab region. This refined load rating analysis was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018), but adopting the LRFD equivalent width for slab bridges based on the results of field testing documented in Volume 2 (Hueste et al. 2019b). This refined load rating was performed under the assumption that the bridge ends are simply supported. The results of this refined load rating analysis are compared to the results of the basic load rating analysis.

8.2.1 Bridge Characteristics

Bridge CS-9 is a two-lane bridge with a total length of 75 ft. The bridge was designed to include three simply supported spans with a controlling span for load rating of 25 ft. The bridge width is 21 ft 4 in. with a roadway width of 20 ft. The TxDOT HS-20 RFs are 0.445 for Inventory and 0.935 for Operating. The transverse section of Bridge CS-9 is shown in Figure 8.1.

Span Length: $L = 24 \text{ ft}$

Bridge Width: $W = 21.33 \text{ ft}$

Concrete Compressive Strength: $f'_c = 2.5 \text{ ksi}$

Steel yield strength: $f_y = 33 \text{ ksi}$

Concrete Density: $\gamma_c = 150 \text{ pcf}$

Asphalt Density: $\gamma_{ws} = 144 \text{ pcf}$

Asphalt Thickness: $t_{ws} = 3 \text{ in.}$

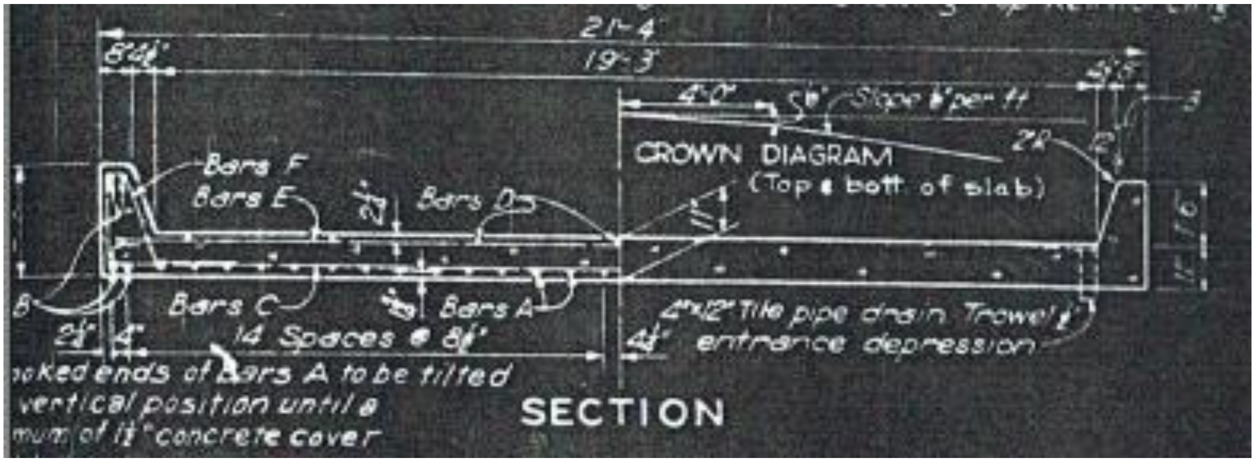

Figure 8.3. Transverse Section of Bridge CS-9 (TxDOT 2018a)

8.2.2 Properties of Concrete Slab Section

Design Slab Width: $b = 12.0$ in.

Tensile Reinforcement Diameter: $d_b = 1.0$ in.

Tensile Reinforcement Area: $A_s = 1.115\ \frac{\text{in}^2}{\text{ft}}$

Slab Depth: $t_{slab} = 11.0$ in.

Reinforcement Spacing: $s = 8.5$ in.

Bottom Cover: $c_{bot} = 1.75$ in.

8.2.3 Properties of Concrete Curb

Top width: $W_{top} = 8.00$ in.

Tensile Reinforcement Dimension: 1.25 in. (sq.)

Tension Steel Area: $A_{s-curb} = 3.125\ \text{in}^2$

Compression Reinf. Dimension: 1.25 in. (sq.)

Compression Steel Area: $A'_{s-curb} = 3.125\ \text{in}^2$

Curb height: $h = 18.00$ in.

Bottom width: $W_{bot} = 12.50$ in.

No. of Reinforcing Bars: $N_b = 2$

Bottom Cover: $c_{bot-curb} = 1.75$ in.

No. of Reinforcing Bars: $N_b = 2$

Top Cover: $c_{top-curb} = 2.25$ in.

8.2.4 Moment Capacity

Detailed in Section 8.1.4, the final reduced moment capacity ϕM_n of the slab section is:

$$\phi M_n = 23.5\ \text{kip–ft}$$

The total reduced nominal moment capacity of the curb section is:

$$\phi M_{n,L-curb} = 419.4 \text{ kip–ft}$$

8.2.5 Structural Analysis for Moment Demand

8.2.5.1 Applied Live Load Moment

The live load moments are distributed over the equivalent strip width *E* (in.) defined in *AASHTO LRFD Specifications* (AASHTO 2017) Article 4.6.2.3, where Equation (8.44) is for a multi-lane-loaded condition:

$$\text{Equivalent width, } E = 84.0 + 1.44\sqrt{L_1 W_1} \leq \frac{12.0W}{N_L} \tag{8.44}$$

$$= 84.0 + 1.44\sqrt{(24)(21.33)} \leq \frac{12.0(21.33)}{2}$$

$$= 116.58 \text{ in.} \leq 127.98 \text{ in.}$$

$$E = 9.7 \text{ ft}$$

where:

L_1 = Modified span length (ft), minimum of actual span or 60 ft

W_1 = Modified edge-to-edge width of bridge, minimum of actual width or 60 ft for multi-lane loading, or 30 ft for single-lane loading (ft)

W = Actual edge-to-edge width of bridge (ft)

N_L = Number of design lanes

The Impact Factor I is applied to the live load to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the impact factor is equal to,

$$I = \frac{50}{L + 125} \leq 0.3 \tag{8.45}$$

$$= \frac{50}{24 + 125} \leq 0.3$$

$$= 0.336 \leq 0.3$$

$$I = 0.3$$

The applied live load moment per lane (two lines of wheels) without the impact factor M_{HS20} is obtained from the AASHTO Manual for Bridge Evaluation (2018) Table C6B-1:

$$M_{HS20} = 192.6\ \frac{\text{kip–ft}}{\text{truck}}$$

The share of the live load moment per unit width taken by the concrete slab can be calculated as:

$$\begin{aligned} M_{Slab,LL} &= \frac{M_{HS20}}{E} \\ &= \frac{192.6}{9.7} \\ &= 19.86\ \frac{\text{kip-ft}}{\text{ft}} \end{aligned} \tag{8.46}$$

The portion of the live load moment acting on the composite L-curb is calculated using Illinois Bulletin 346 (Jenson et al. 1943) approach as detailed in Section 8.1.5.1 and found as:

$$M_{LL,L-curb} = 187.8\ \text{kip-ft}$$

8.2.5.2 Dead Load Moment of Structural Components

Detailed in Section 8.3.5.2, the slab self-weight w_s is:

$$w_s = 0.167\ \frac{\text{kip}}{\text{ft}}$$

The self-weight of the curbs w_{curb} is:

$$w_{curb} = 0.335\ \frac{\text{kip}}{\text{ft}}$$

8.2.5.3 Superimposed Dead Load Moment

The superimposed dead load per unit width due to the wearing surface w_{SDL} is:

$$w_{SDL} = 0.036\ \frac{\text{kip}}{\text{ft}}$$

8.2.5.4 Total Dead Load Moment

Detailed in Section 8.1.5.4, the total applied moment due to structural dead loads and superimposed dead loads $M_{Slab,DL}$ for the slab is:

$$M_{DL,slab} \quad = \quad 6.1\ \frac{\text{kip-ft}}{\text{ft}}$$

The total applied moment due to structural dead loads and superimposed dead loads $M_{DL,L-curb}$ for the L-curb is:

$$M_{DL,L-curb} \quad = \quad 108.0\ \text{kip–ft}$$

8.2.6 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{8.47}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

Slab Rating:

$$\begin{aligned} RF_{ISlab} &= \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \\ &= \frac{23.5 - [(1.3)(6.1)]}{(2.17)(19.86\ )} \\ &= 0.36 \end{aligned} \tag{8.48}$$

Curb Rating:

$$RF_{IL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \quad (8.49)$$

$$= \frac{419.4 - [(1.3)(108.0)]}{(2.17)(187.8\ )}$$

$$= 0.68$$

The inventory rating for Bridge CS-9 is the minimum of the two RFs determined for flexural strength of the slab and L-curb (TxDOT 2018b).

$$RF_I = Min(RF_{ISlab}, RF_{IL-curb}) \quad (8.50)$$

$$= Min(0.36, 0.68)$$

$$= 0.36$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

Slab Rating:

$$RF_{OSlab} = \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \quad (8.51)$$

$$= \frac{23.5 - [(1.3)(6.1)]}{(1.3)(19.86\ )}$$

$$= 0.60$$

Curb Rating:

$$RF_{OL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \quad (8.52)$$

$$= \frac{419.4 - [(1.3)(108.0)]}{(1.3)(187.8)}$$

$$= 1.14$$

TxDOT considers the operating rating for Bridge CS-9 as the weighted average of the L-curb and slab rating proportionate to their tributary width (TxDOT 2018b):

$$RF_O = \frac{2\left(RF_{OL-curb}(W_{bot}+4h)\right)+\left(RF_{OSlab}\left(W-2(W_{bot}+4h)\right)\right)}{W} \quad (8.53)$$

$$= \frac{2\left(1.14(12.5+4(11))\right)+\left(0.60\left(255.6-2(12.5+4(11))\right)\right)}{255.6}$$

$$= 0.84$$

Does not pass.

The controlling LFR flexural strength rating factors for Bridge CS-9 using the refined load rating analysis that considers live load distribution to the mid-slab region based on the *AASHTO LRFD Specifications* (AASHTO 2017), along with the IB346 methodology for the live load distribution to the the L-Curb section, are equal to:

$$RF_I = 0.36$$

$$RF_O = 0.84$$

Note that the above values are smaller than those reported by TxDOT (0.45 for Inventory and 0.95 for Operating) because the *AASHTO LRFD Specifications* provide a more conservative estimate of the live load distribution. For both inventory and operating ratings, the RFs are less than 1.0, and therefore do not pass these load rating according the AASHTO MBE.

The basic load rating controlling RFs were 0.45 for Inventory and 0.92 for Operating. The new rating factors represent a 20 percent decrease for Inventory and a 9 percent decrease for Operating. Table 8.3 compares the controlling RFs determined using the LRFD equivalent width formula for the slab to the controlling RFs determined in the basic load rating.

Table 8.1. End Restraint RF Comparison

Rating Factor	Basic Load Rating	Load Rating with AASHTO LRFD	AASHTO LRFD/Basic Load Rating
Inventory	0.45	0.36	0.80
Operating	0.92	0.84	0.91

8.3 LOAD RATING ANALYSIS CONSIDERING MEASURED MATERIAL PROPERTIES

This section shows a refined load rating analysis performed for Bridge CS-9 considering measured concrete compressive strength. This refined load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018) except the use of measured concrete compressive strength. It was also performed under the assumption that the bridge ends are simply-supported. The results of this refined load rating analysis are compared to the results of the basic load rating analysis.

8.3.1 Bridge Characteristics

Bridge CS-9 is a two-lane bridge with a total length of 75 ft. The bridge was designed to include three simply supported spans with a controlling span for load rating of 25 ft. The bridge width is 21 ft 4 in. with a roadway width of 20 ft. The TxDOT HS-20 RFs are 0.445 for Inventory and 0.935 for Operating. The transverse section of Bridge CS-9 is shown in Figure 8.4. The concrete compressive strength is updated as 5.2 ksi based on the NDE material tests.

Span Length: $L = 24\ \text{ft}$

Concrete Compressive Strength: $f'_c = 5.2\ \text{ksi}$

Concrete Density: $\gamma_c = 150\ \text{pcf}$

Asphalt Thickness: $t_{ws} = 3\ \text{in.}$

Bridge Width: $W = 21.33\ \text{ft}$

Steel yield strength: $f_y = 33\ \text{ksi}$

Asphalt Density: $\gamma_{ws} = 144\ \text{pcf}$

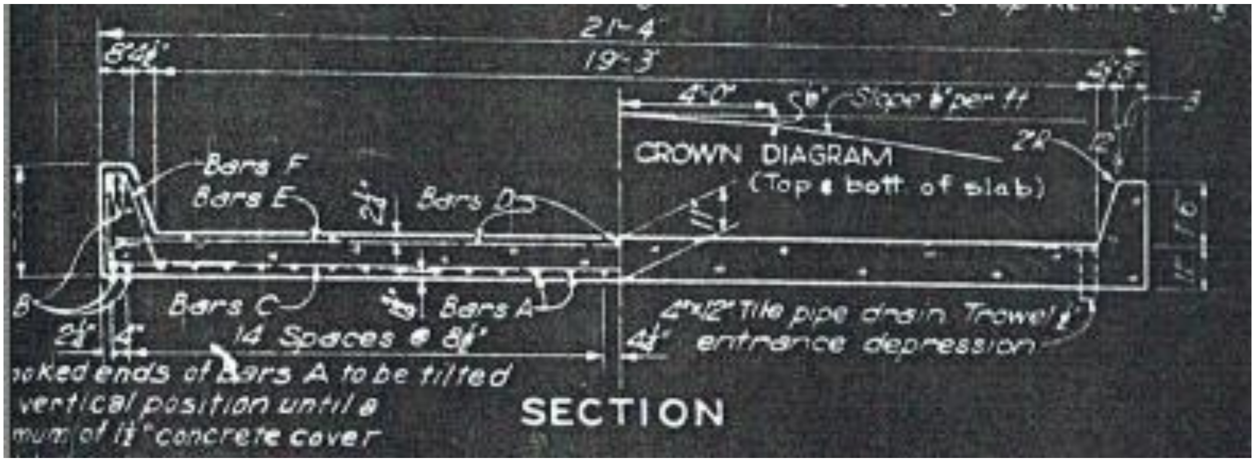

Figure 8.4. Transverse Section of Bridge CS-9 (TxDOT 2018a)

8.3.2 Properties of Concrete Slab

Design Slab Width: $b = 12.0$ in.

Tensile Reinforcement Diameter: $d_b = 1.0$ in.

Tensile Reinforcement Area: $A_s = 1.115\ \frac{\text{in}^2}{\text{ft}}$

Slab Depth: $t_{slab} = 11.0$ in.

Reinforcement Spacing: $s = 8.5$ in.

Bottom Cover: $c_{bot} = 1.75$ in.

8.3.3 Properties of Concrete Curb

Top width: $W_{top} = 8.00$ in.

Tensile Reinforcement Dimension: 1.25 in. (sq)

Tension Steel Area: $A_{s-curb} = 3.125\ \text{in}^2$

Compression Reinf. Dimension: 1.25 in. (sq)

Compression Steel Area: $A'_{s-curb} = 3.125\ \text{in}^2$

Curb height: $h = 18.00$ in.

Bottom width: $W_{bot} = 12.50$ in.

No. of Reinforcing Bars: $N_b = 2$

Bottom Cover: $c_{bot-curb} = 1.75$ in.

No. of Reinforcing Bars: $N_b = 2$

Top Cover: $c_{top-curb} = 2.25$ in.

8.3.4 Moment Capacity

The procedure outlined in Article 8.16 of the AASHTO Standard Specifications (AASHTO 2002) is followed to determine the moment capacity for a unit width of slab. First, it is assumed that the tensile reinforcement yields and this assumption is verified. If the tensile reinforcement does not yield, then the tensile stress is calculated using Hooke's Law and this stress is used to determine the nominal moment capacity of the section.

8.3.4.1 Slab Capacity

Calculate the flexural capacity per unit width of slab according to AASHTO Standard Specifications Article 8.16. Ultimate strain in concrete, $\varepsilon_{cu} = 0.003$.

The stress block factor for 5.2 ksi concrete is calculated as,

$$0.65 \leq \beta_1 = 0.85 - \left(0.05(f_c{}' - 4)\right) \leq 0.85 \tag{8.54}$$

$$\beta_1 = 0.79$$

The depth of equivalent stress block,

$$\begin{aligned} \mathrm{a} &= \frac{A_s f_y}{0.85 f_c' b} \qquad (8.55) \\ &= \frac{\left(1.115 \frac{\mathrm{in}^2}{\mathrm{ft}}\right)(33 \text{ ksi})}{(0.85)(5.2 \text{ ksi})(12 \text{ in.})} \\ &= 0.694 \text{ in.} \end{aligned}$$

The distance from extreme compression fiber to the neutral axis,

$$\begin{aligned} \mathrm{c} &= \frac{a}{\beta_1} \qquad (8.56) \\ &= \frac{0.694 \text{ in.}}{0.79} \\ &= 0.878 \text{ in.} \end{aligned}$$

Check if steel has yielded:

$$\begin{aligned} \varepsilon_s &= \varepsilon_{cu}\left(\frac{d-c}{c}\right) &\geq \frac{f_y}{E_s} \qquad (8.57) \\ &= 0.003\left(\frac{9.25 \text{ in.} - 0.878 \text{ in.}}{0.878 \text{ in.}}\right) &\geq \frac{33 \text{ ksi}}{29000 \text{ ksi}} \\ &= 0.0286 &\geq 0.00114 \text{ (Steel yields)} \end{aligned}$$

Flexural reduction factor, $\phi = 0.90$. Nominal moment capacity of the slab section may be calculated as:

$$\begin{aligned} \phi M_n &= \phi A_s f_y \left(d - \frac{a}{2}\right) \\ &= (0.90)\left(1.115 \frac{\text{in}^2}{\text{ft}}\right)(33\text{ ksi})\left(9.25\text{ in.} - \frac{0.694\text{ in.}}{2}\right) \\ &= 24.6\ \frac{\text{kip-ft}}{\text{ft}} \end{aligned} \tag{8.58}$$

8.3.4.2 Curb Capacity

The slope of the trapezoidal curb sections is calculated as:

$$\begin{aligned} C_{slope} &= \frac{W_{bot} - W_{top}}{h} \\ &= \frac{12.5\text{ in.} - 8\text{ in.}}{18\text{ in.}} \\ &= 0.25 \end{aligned} \tag{8.59}$$

The effective area of reinforcement from the slab section $A_{s,eff}$ is calculated for a width of $4t_{slab}$ as:

$$\begin{aligned} A_{s,eff} &= A_s(4t_{slab}) \\ &= \left(1.115\ \frac{\text{in}^2}{\text{ft}}\right)\left(4(11\text{ in.})\right) \\ &= 4.09\text{ in}^2 \end{aligned} \tag{8.60}$$

Effective depth of the curb section may be calculated as:

$$\begin{aligned} d_{curb} &= h + t_{slab} - c_{tcurb} \\ &= 18\text{ in.} + 11\text{ in.} - 1.75\text{ in.} \\ &= 27.25\text{ in.} \end{aligned} \tag{8.61}$$

Article 8.16 of the AASHTO Standard Specifications suggests the following for sections with compressive reinforcement.

$$\left(\frac{A_{s-curb} + A_{s,eff} - A'_{s-curb}}{W_{top}d_{curb}}\right) \geq 0.85\beta_1\left(\frac{f'_c c_{ccurb}}{f_y d_{curb}}\right)\left(\frac{87}{87 - f_y}\right) \tag{8.62}$$

$$\left(\frac{3.125\ \text{in}^2 + 4.09\ \text{in}^2 - 3.125\ \text{in}^2}{(8\ \text{in.})(27.25\ \text{in.})}\right) \geq 0.85(0.85)\left(\frac{(5.2\ \text{ksi})(2.25\ \text{in.})}{(33\ \text{ksi})(27.25\ \text{in.})}\right)\left(\frac{87}{87 - 33\ \text{ksi}}\right)$$

$$0.0855 \geq 0.0151\ \text{(compression steel yields)}$$

The total tensile reinforcement $A_{s,total}$ in the L-curb section is calculated as:

$$\begin{aligned} A_{s,total} &= A_{s,eff} + A_{s-curb} \\ &= 4.09\ \text{in}^2 + 3.125\ \text{in}^2 \\ &= 7.215\ \text{in}^2 \end{aligned} \tag{8.63}$$

Depth of stress block in the curb can be calculated as:

$$\begin{aligned} \left(A_{s,total} - A'_{s-curb}\right)f_y &= 0.85f'_c W_{top} a_{curb} + \frac{0.85f'_c C_{slope}}{2}{a_{curb}}^2 && \leq h \\ (7.215\ \text{in}^2 - 3.125\ \text{in}^2)(33\ \text{ksi}) &= 0.85(5.2\ \text{ksi})(8\ \text{in.})a_{curb} + \frac{0.85(5.2\ \text{ksi})(0.25)}{2}{a_{curb}}^2 && \leq 18\ \text{in.} \\ a_{curb} &= 3.61\ \text{in.} && \leq 18\ \text{in.} \end{aligned} \tag{8.64}$$

The distance from top of curb to the centroid of the stress block is calculated as:

$$\begin{aligned} y_{curb} &= \frac{3a_{curb}W_{top} + 2C_{slope}{a_{curb}}^2}{6W_{top} + 3C_{slope}a_{curb}} \\ &= \frac{3(3.61\ \text{in.})(8\ \text{in.}) + 2(0.25)(3.61\ \text{in.})^2}{6(8\ \text{in.}) + 3(0.25)(3.61\ \text{in.})} \\ &= 1.84\ \text{in.} \end{aligned} \tag{8.65}$$

The total nominal moment of the curb section is calculated by adding the nominal moment capacity of the two components as:

$$\begin{aligned}
\phi M_{n,L-curb} &= \phi\left(A_{s,curb} + A_{s,eff} - A'_{s,curb}\right) f_y \left(d_{curb} - y_{curb}\right) \\
&\quad + \phi A'_{s,curb}\left(f_y - 0.85 f'_c\right)\left(d_{curb} - c_{top-curb}\right) \\
&= (0.90)(3.125\text{ in}^2 + 4.09\text{ in}^2 - 3.125\text{ in}^2)(33\text{ ksi}) \\
&\quad (27.25\text{ in.} - 1.84\text{ in.}) + (0.90)(3.125\text{ in}^2) \\
&\quad \left(33\text{ ksi} - 0.85(5.2\text{ ksi})\right)(27.25\text{ in.} - 2.25\text{ in.}) \\
&= 424.7\text{ kip–ft}
\end{aligned} \tag{8.66}$$

8.3.5 Structural Analysis for Moment Demand

8.3.5.1 Applied Live Load Moment

Detailed in Section 8.1.5.1, the applied live load moment $M_{Slab,LL}$ with dynamic effects, per unit width of slab is:

$$M_{Slab,LL} = 16.1\ \frac{\text{kip-ft}}{\text{ft}}$$

The portion of the live load moment acting on the composite L-curb is:

$$M_{LL,L-curb} = 187.8\text{ kip-ft}$$

8.3.5.2 Dead Load Moment of Structural Components

Detailed in Section 8.3.5.2, the slab self-weight w_s is:

$$w_s = 0.167\ \frac{\text{kip}}{\text{ft}}$$

The self-weight of the curbs w_{curb} is:

$$w_{curb} = 0.335\ \frac{\text{kip}}{\text{ft}}$$

8.3.5.3 Superimposed Dead Load Moment

The superimposed dead load per unit width due to the wearing surface w_{SDL} is:

$$w_{SDL} = 0.036\ \frac{\text{kip}}{\text{ft}}$$

8.3.5.4 Total Dead Load Moment

Detailed in Section 8.1.5.4, the total applied moment due to structural dead loads and superimposed dead loads $M_{Slab,DL}$ for the slab is:

$$M_{DL,slab} = 6.1\ \frac{\text{kip-ft}}{\text{ft}}$$

The total applied moment due to structural dead loads and superimposed dead loads $M_{DL,L-curb}$ for the L-curb is:

$$M_{DL,L-curb} = 108.0\ \text{kip–ft}$$

8.3.6 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{8.67}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

Slab Rating:

$$\begin{aligned} RF_{ISlab} &= \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \\ &= \frac{24.6 - [(1.3)(6.1)]}{(2.17)(16.1\ )} \\ &= 0.48 \end{aligned} \tag{8.68}$$

Curb Rating:

$$RF_{IL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \tag{8.69}$$

$$= \frac{424.7 - [(1.3)(108.0)]}{(2.17)(187.8\ )}$$

$$= 0.70$$

The inventory rating for Bridge CS-9 is the minimum of the two RFs (TxDOT 2018b).

$$RF_I = Min(RF_{ISlab}, RF_{IL-curb}) \tag{8.70}$$

$$= Min(0.48{,}0.70)$$

$$= 0.48$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

Slab Rating:

$$RF_{OSlab} = \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \tag{8.71}$$

$$= \frac{24.6 - [(1.3)(6.1)]}{(1.3)(16.1\ )}$$

$$= 0.80$$

Curb Rating:

$$RF_{OL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \tag{8.72}$$

$$= \frac{424.7 - [(1.3)(108.0)]}{(1.3)(187.8\ )}$$

$$= 1.16$$

The operating rating for Bridge CS-9 is calculated as (TxDOT 2018b):

$$RF_O = \frac{2\left(RF_{OL-curb}(W_{bot}+4h)\right)+\left(RF_{OSlab}\left(W-2(W_{bot}+4h)\right)\right)}{W} \quad (8.73)$$

$$= \frac{2\left(1.16(12.5+4(11))\right)+\left(0.80\left(255.6-2(12.5+4(11))\right)\right)}{255.6}$$

$$= 0.96$$

Does not pass.

8.3.7 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge CS-9 from the measured material properties are equal to:

$$RF_I = 0.48$$

$$RF_O = 0.96$$

The basic load rating controlling RFs were 0.45 for Inventory and 0.92 for Operating. The new rating factors represent a 6.7 percent increase for Inventory and a 4.3 percent increase for Operating. Table 7.2 compares the controlling RFs determined using measured material properties to the controlling RFs determined in the basic load rating.

Table 8.2. Measured Material Properties RF Comparison

Rating Factor	Basic Load Rating	Load Rating with Measured Material Properties	Measured Material Properties/Basic Load Rating
Inventory	0.45	0.48	1.07
Operating	0.92	0.96	1.04

8.4 LOAD RATING ANALYSIS CONSIDERING END RESTRAINT

This section shows a refined load rating analysis performed for Bridge CS-9 considering end restraint based on load testing while keeping the concrete strength same as the basic load rating analysis. This refined load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). However, a Level II Analysis for end restraint is also performed to determine updated applied live load moments. This Level II Analysis is based on the results of the calibrated FEM model using the load test performed on the bridge in the field. The results of this refined load rating analysis are compared to the results of the basic load rating analysis.

8.4.1 Bridge Characteristics

Bridge CS-9 is a two-lane bridge with a total length of 75 ft. The bridge was designed to include three simply supported spans with a controlling span for load rating of 25 ft. The bridge width is 21 ft 4 in. with a roadway width of 20 ft. The TxDOT HS-20 RFs are 0.445 for Inventory and 0.935 for Operating. The transverse section of Bridge CS-9 is shown in Figure 8.1.

Span Length: $L = 24\ \text{ft}$

Bridge Width: $W = 21.33\ \text{ft}$

Concrete Compressive Strength: $f'_c = 2.5\ \text{ksi}$

Steel yield strength: $f_y = 33\ \text{ksi}$

Concrete Density: $\gamma_c = 150$ pcf

Asphalt Density: $\gamma_{ws} = 144$ pcf

Asphalt Thickness: $t_{ws} = 3\ \text{in.}$

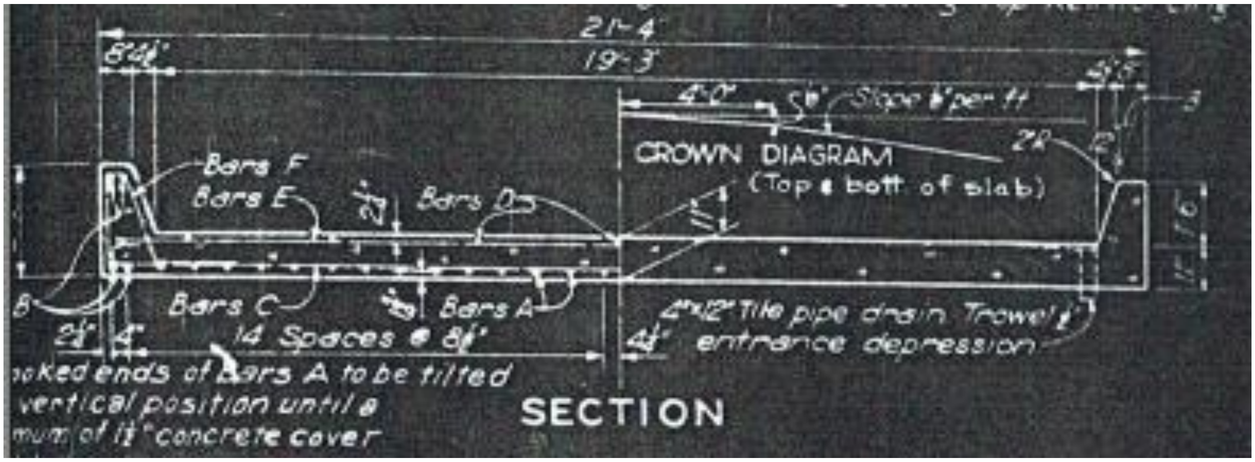

Figure 8.5. Transverse Section of Bridge CS-9 (TxDOT 2018a)

8.4.2 Properties of Concrete Slab

Design Slab Width: $b = 12.0$ in.

Tensile Reinforcement Diameter: $d_b = 1.0$ in.

Tensile Reinforcement Area: $A_s = 1.115\ \frac{\text{in}^2}{\text{ft}}$

Slab Depth: $t_{slab} = 11.0$ in.

Reinforcement Spacing: $s = 8.5$ in.

Bottom Cover: $c_{bot} = 1.75$ in.

8.4.3 Properties of Concrete Curb

Top width: $W_{top} = 8.00$ in.

Tensile Reinforcement Dimension: 1.25 in. (sq)

Tension Steel Area: $A_{s-curb} = 3.125\ \text{in}^2$

Compression Reinf. Dimension: 1.25 in. (sq)

Compression Steel Area: $A'_{s-curb} = 3.125\ \text{in}^2$

Curb height: $h = 18.00$ in.

Bottom width: $W_{bot} = 12.50$ in.

No. of Reinforcing Bars: $N_b = 2$

Bottom Cover: $c_{bot-curb} = 1.75$ in.

No. of Reinforcing Bars: $N_b = 2$

Top Cover: $c_{top-curb} = 2.25$ in.

8.4.4 Moment Capacity

Detailed in Section 8.1.4, the final reduced moment capacity ϕM_n of the slab section is:

$$\phi M_n = 23.5\ \text{kip–ft}$$

The total reduced nominal moment of the curb section is:

$$\phi M_{n,L-curb} = 419.4 \text{ kip–ft}$$

8.4.5 Structural Analysis for Moment Demand

8.4.5.1 Applied Live Load Moment

Detailed in Section 8.1.5.1, the applied live load moment per 1 ft slab with dynamic effects is calculated as 16.1 kip-ft/ft. During the field testing, strain measurement were taken for 2.583 ft slab sections, which gives an applied live load moment $M_{LL,slab-simple}$ with dynamic effect as

$$M_{LL,slab-simple} = 2.583 * 16.1 = 41.59 \text{ kip-ft}$$

Consideration of End Restraint:

The modulus of elasticity for concrete was calculated according to Article 8.7.2 of the AASHTO Standard Specifications (AASHTO 2002) as:

$$\begin{aligned} E_c &= 33K_1{w_c}^{1.5}\sqrt{f_c{}'} \quad (8.74) \\ &= (33)(1.0)((150\text{lbs/ft}^3)^{1.5})\sqrt{2500\text{psi}}/1000 \\ &= 3031.2 \text{ ksi} \end{aligned}$$

From load test results for an interior section, the maximum compressive strain in the bottom during Middle Path loading was measured as 1.54 microstrain ($\mu\varepsilon$) at the West end and 1.77 microstrain ($\mu\varepsilon$) at the East end. This microstrain can be converted to a stress (σ) value:

$$\begin{aligned} \sigma_1 &= \varepsilon E \quad (8.75) \\ &= (1.54)(10^{-6})(3031.2 \text{ ksi}) \\ &= 0.005 \text{ ksi} \end{aligned}$$

$$\begin{aligned} \sigma_2 &= \varepsilon E \quad (8.76) \\ &= (1.77)(10^{-6})(3031.2 \text{ ksi}) \\ &= 0.005 \text{ ksi} \end{aligned}$$

This stress value can be converted to a moment value, giving the restraining moment $M_{end-fix}$:

$$M_{end-fix,1} = \sigma \frac{I_x}{y} = (0.005\text{ ksi})\left(\frac{6682.4\text{ in}^4}{8.29\text{ in.}}\right) = 0.31\text{ kip-ft} \tag{8.77}$$

$$M_{end-fix,2} = \sigma \frac{I_x}{y} = (0.005\text{ ksi})\left(\frac{6682.4\text{ in}^4}{8.29\text{ in.}}\right) = 0.36\text{ kip-ft} \tag{8.78}$$

Therefore, the new applied midspan live load moment considering end restraint is:

$$M_{Slab,LL} = M_{LL,slab-simple} - \frac{(M_{end1} + M_{end2})}{2} = 41.59 - \frac{0.31 + 0.36}{2} = 41.25\text{ kip–ft} \tag{8.79}$$

The new applied per foot width midspan live load moment, including impact, for the slab considering end restraint is:

$$M_{Slab,LL} = \frac{M_{Slab,LL}}{2.583} = \frac{41.25\text{ kip–ft}}{2.583} = 15.98\ \frac{\text{kip-ft}}{\text{ft}} \tag{8.80}$$

The portion of the live load moment acting on the composite L-curb is:

$$M_{LL,L-curb,simple} = 187.8\text{ kip-ft}$$

From load test results for an exterior section, the maximum compressive strain in the bottom during Middle Path loading was measured as 7.51 microstrain ($\mu\varepsilon$) at the West end and 1.12 microstrain ($\mu\varepsilon$) at the East end. This microstrain can be converted to a stress (σ) value:

$$\begin{aligned} \sigma_1 &= \varepsilon E \\ &= (7.51)(10^{-6})(3031.2 \text{ ksi}) \\ &= 0.023 \text{ ksi} \end{aligned} \tag{8.81}$$

$$\begin{aligned} \sigma_2 &= \varepsilon E \\ &= (1.12)(10^{-6})(3031.2 \text{ ksi}) \\ &= 0.003 \text{ ksi} \end{aligned} \tag{8.82}$$

This stress value can be converted to a moment value, giving the restraining moment $M_{end-fix}$:

$$\begin{aligned} M_{end-fix,1} &= \sigma \frac{I_x}{y} \\ &= (0.023 \text{ ksi})\left(\frac{39{,}713.8 \text{ in}^4}{21.43 \text{ in.}}\right) \\ &= 3.5 \text{ kip-ft} \end{aligned} \tag{8.83}$$

$$\begin{aligned} M_{end-fix,2} &= \sigma \frac{I_x}{y} \\ &= (0.003 \text{ ksi})\left(\frac{39{,}713.8 \text{ in}^4}{21.43 \text{ in.}}\right) \\ &= 0.5 \text{ kip-ft} \end{aligned} \tag{8.84}$$

Therefore, the new applied midspan live load moment for the L-curb considering end restraint is:

$$\begin{aligned} M_{LL,L-curb} &= M_{L-curb,LL,simple} - \frac{(M_{end1} + M_{end2})}{2} \\ &= 187.8 - \frac{3.5 + 0.5}{2} \\ &= 185.8 \text{ kip–ft} \end{aligned} \tag{8.85}$$

8.4.5.2 Dead Load Moment of Structural Components

Detailed in Section 8.3.5.2, the slab self-weight w_S is:

$$w_S = 0.167\ \frac{\text{kip}}{\text{ft}}$$

The self-weight of the curbs w_{Curb} is:

$$w_{Curb} = 0.335\ \frac{\text{kip}}{\text{ft}}$$

8.4.5.3 Superimposed Dead Load Moment

The superimposed dead load per unit width due to the wearing surface w_{SDL} is:

$$w_{SDL} = 0.036\ \frac{\text{kip}}{\text{ft}}$$

8.4.5.4 Total Dead Load Moment

Detailed in Section 8.1.5.4, the total applied moment due to structural dead loads and superimposed dead loads $M_{DL,Slab}$ for the slab is:

$$M_{DL,Slab} = 6.1\ \frac{\text{kip-ft}}{\text{ft}}$$

The total applied moment due to structural dead loads and superimposed dead loads $M_{DL,L-curb}$ for the L-curb is:

$$M_{DL,L-curb} = 108.0\ \text{kip}-\text{ft}$$

8.4.6 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{8.86}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

Slab Rating:

$$RF_{ISlab} = \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \tag{8.87}$$

$$= \frac{23.5 - [(1.3)(6.1)]}{(2.17)(15.98\ )}$$

$$= 0.45$$

Curb Rating:

$$RF_{IL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \tag{8.88}$$

$$= \frac{419.4 - [(1.3)(108.0)]}{(2.17)(185.8\ )}$$

$$= 0.69$$

The inventory rating for Bridge CS-9 is the minimum of the two RFs (TxDOT 2018b).

$$RF_I = Min(RF_{ISlab}, RF_{IL-curb}) \tag{8.89}$$

$$= Min(0.52, 0.69)$$

$$= 0.45$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

Slab Rating:

$$RF_{OSlab} = \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \tag{8.90}$$

$$= \frac{23.5 - [(1.3)(6.1)]}{(1.3)(15.98)}$$

$$= 0.75$$

Curb Rating:

$$RF_{OL-curb} = \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \tag{8.91}$$

$$= \frac{419.4 - [(1.3)(108.0)]}{(1.3)(185.8)}$$

$$= 1.16$$

The operating rating for Bridge CS-9 is calculated as (TxDOT 2018b):

$$RF_O = \frac{2\left(RF_{OL-curb}(W_{bot} + 4h)\right) + \left(RF_{OSlab}\left(W - 2(W_{bot} + 4h)\right)\right)}{W} \tag{8.92}$$

$$= \frac{2\left(1.16(12.5 + 4(11))\right) + \left(0.75\left(255.6 - 2(12.5 + 4(11))\right)\right)}{255.6}$$

$$= 0.93$$

Does not pass.

8.4.7 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge CS-9 from the Level II end restraint analysis are equal to:

$$RF_I = 0.45$$

$$RF_O = 0.93$$

The basic load rating controlling RFs were 0.45 for Inventory and 0.92 for Operating. The new rating factors are almost same as the basic load rating factors because the measured end restraint was very small. Table 8.3 compares the controlling RFs determined Level II end restraint analysis to the controlling RFs determined in the basic load rating.

Table 8.3. End Restraint RF Comparison

Rating Factor	Basic Load Rating	Load Rating with End Restraint	End Restraint/Basic Load Rating
Inventory	0.45	0.45	1.00
Operating	0.92	0.93	1.01

8.5 LOAD RATING ANALYSIS CONSIDERING FEM LIVE LOAD MOMENTS

This section shows a refined load rating analysis performed for Bridge CS-9 considering live load moments from an FEM model that assumes simply supported boundary conditions and the same concrete compressive strength as the basic load rating analysis. This refined load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018) except that the live load moment demands are taken from FEM analysis. The results of this refined load rating analysis are compared to the results of the basic load rating analysis.

8.5.1 Bridge Characteristics

Bridge CS-9 is a two-lane bridge with a total length of 75 ft. The bridge was designed to include three simply supported spans with a controlling span for load rating of 25 ft. The bridge width is 21 ft 4 in. with a roadway width of 20 ft. The TxDOT HS-20 RFs are 0.445 for Inventory and 0.935 for Operating. The transverse section of Bridge CS-9 is shown in Figure 8.1.

Span Length: $L = 24\text{ ft}$

Bridge Width: $W = 21.33\text{ ft}$

Concrete Compressive Strength: $f'_c = 2.5\text{ ksi}$

Steel yield strength: $f_y = 33\text{ ksi}$

Concrete Density: $\gamma_c = 150\text{ pcf}$

Asphalt Density: $\gamma_{ws} = 144\text{ pcf}$

Asphalt Thickness: $t_{ws} = 3\text{ in.}$

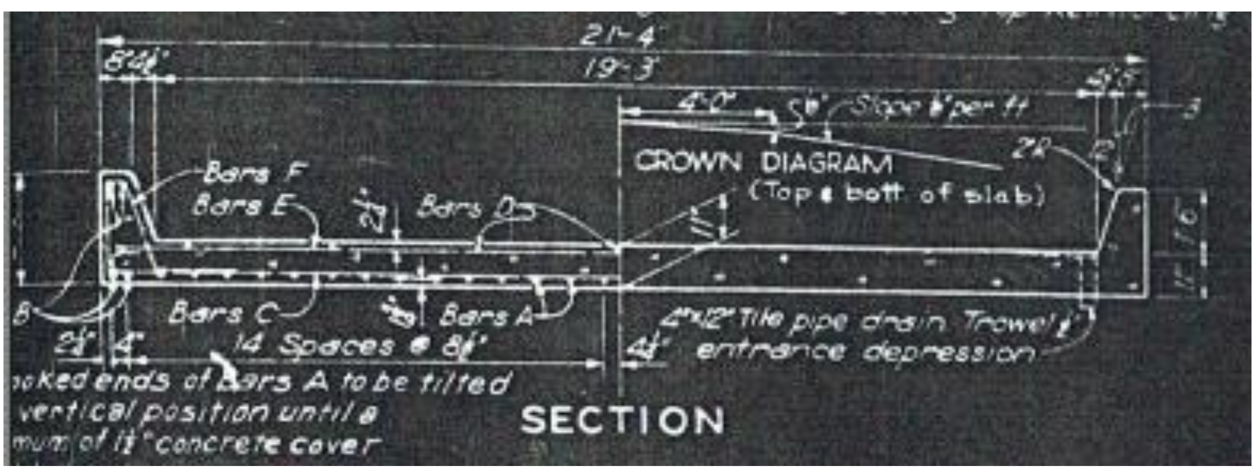

Figure 8.6. Transverse Section of Bridge CS-9 (TxDOT 2018a)

8.5.2 Properties of Concrete Slab

Design Slab Width: $b = 12.0$ in.

Tensile Reinforcement Diameter: $d_b = 1.0$ in.

Tensile Reinforcement Area: $A_s = 1.115\ \frac{\text{in}^2}{\text{ft}}$

Slab Depth: $t_{slab} = 11.0$ in.

Reinforcement Spacing: $s = 8.5$ in.

Bottom Cover: $c_{bot} = 1.75$ in.

8.5.3 Properties of Concrete Curb

Top width: $W_{top} = 8.00$ in.

Tensile Reinforcement Dimension: 1.25 in. (sq)

Tension Steel Area: $A_{s-curb} = 3.125\ \text{in}^2$

Compression Reinf. Dimension: 1.25 in. (sq)

Compression Steel Area: $A'_{s-curb} = 3.125\ \text{in}^2$

Curb height: $h = 18.00$ in.

Bottom width: $W_{bot} = 12.50$ in.

No. of Reinforcing Bars: $N_b = 2$

Bottom Cover: $c_{bot-curb} = 1.75$ in.

No. of Reinforcing Bars: $N_b = 2$

Top Cover: $c_{top-curb} = 2.25$ in.

8.5.4 Moment Capacity

Detailed in Section 8.1.4, the final reduced moment capacity ϕM_n of the slab section is:

$$\phi M_n = 23.5\ \frac{\text{kip-ft}}{\text{ft}}$$

The total reduced nominal moment of the curb section is:

$$\phi M_{n,L-curb} = 419.4\ \text{kip–ft}$$

8.5.5 Structural Analysis for Moment Demand

8.5.5.1 Applied Live Load Moment

The Impact Factor I is applied to the live load to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the impact factor is equal to,

$$I = \frac{50}{L + 125} \leq 0.3 \tag{8.93}$$

$$
\begin{aligned}
&= \frac{50}{24+125} \leq 0.3 \\
&= 0.336 \leq 0.3 \\
I &= 0.3
\end{aligned}
$$

The applied live load moment on the interior slab without the impact factor $M_{Slab,LL}$ is obtained from the original FEM model as:

$$M_{Slab,LL} = 120.5 \text{ kip–ft}$$

The applied live load moment on the L-curb section without the impact factor $M_{LL,L-curb}$ is obtained from the original FEM model as:

$$M_{LL,L-curb} = 149.1 \text{ kip–ft}$$

Therefore, the applied live load moment with dynamic effects for the slab and curb can be calculated as:

$$\begin{aligned} M_{Slab,LL} &= \frac{M_{Slab,LL}}{(W-2(W_{bot}+4t_{slab}))}(1+I) \\ &= \left(\frac{120.5 \text{ kip–ft}}{\left((21.33 \text{ ft})-2(1.04 \text{ ft}+3.67 \text{ ft})\right)}\right)(1.3) \\ &= 13.1\ \frac{\text{kip-ft}}{\text{ft}} \end{aligned} \tag{8.94}$$

$$\begin{aligned} M_{LL,L-curb} &= M_{LL,L-curb}(1+I) \\ &= (149.1)(1.3) \\ &= 193.8 \text{ kip-ft} \end{aligned} \tag{8.95}$$

8.5.5.2 Dead Load Moment of Structural Components

Detailed in Section 8.3.5.2, the slab self-weight w_S is:

$$w_S = 0.167\ \frac{\text{kip}}{\text{ft}}$$

The self-weight of the curbs w_{Curb} is:

$$w_{Curb} = 0.335\ \frac{\text{kip}}{\text{ft}}$$

8.5.5.3 Superimposed Dead Load Moment

The superimposed dead load per unit width due to the wearing surface w_{SDL} is:

$$w_{SDL} = 0.036\ \frac{\text{kip}}{\text{ft}}$$

8.5.5.4 Total Dead Load Moment

Detailed in Section 8.1.5.4, the total applied moment due to structural dead loads and superimposed dead loads $M_{DL,Slab}$ for the slab is:

$$M_{DL,Slab} \quad = \quad 6.1\ \frac{\text{kip-ft}}{\text{ft}}$$

The total applied moment due to structural dead loads and superimposed dead loads $M_{L-curb,DL}$ for the L-curb is:

$$M_{DL,L-curb} \quad = \quad 108.0\ \text{kip–ft}$$

8.5.6 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{8.96}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

Slab Rating:

$$RF_{ISlab} \quad = \quad \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \tag{8.97}$$

$$= \quad \frac{23.5 - [(1.3)(6.1)]}{(2.17)(13.1\ )}$$

$$= \quad 0.55$$

Curb Rating:

$$RF_{IL-curb} \quad = \quad \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \tag{8.98}$$

$$= \quad \frac{419.4 - [(1.3)(108.0)]}{(2.17)(193.8\ )}$$

$$= \quad 0.66$$

The inventory rating for Bridge CS-9 is the minimum of the two RFs (TxDOT 2018b).

$$\begin{aligned} RF_I &= Min(RF_{ISlab}, RF_{IL-curb}) \\ &= Min(0.54, 0.66) \\ &= 0.54 \end{aligned} \tag{8.99}$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

Slab Rating:

$$\begin{aligned} RF_{OSlab} &= \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \\ &= \frac{23.5 - [(1.3)(6.1)]}{(1.3)(13.1)} \\ &= 0.91 \end{aligned} \tag{8.100}$$

Curb Rating:

$$\begin{aligned} RF_{OL-curb} &= \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \\ &= \frac{419.4 - [(1.3)(108.0)]}{(1.3)(193.8)} \\ &= 1.11 \end{aligned} \tag{8.101}$$

The operating rating for Bridge CS-9 is calculated as (TxDOT 2018b):

$$\begin{aligned} RF_O &= \frac{2\big(RF_{OL-curb}(W_{bot} + 4h)\big) + \Big(RF_{OSlab}\big(W - 2(W_{bot} + 4h)\big)\Big)}{W} \\ &= \frac{2\Big(1.11\big(12.5 + 4(11)\big)\Big) + \Big(0.91\Big(255.6 - 2\big(12.5 + 4(11)\big)\Big)\Big)}{255.6} \\ &= 1.0 \end{aligned} \tag{8.102}$$

Passes.

8.5.7 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge CS-9 from the Level II end restraint analysis are equal to:

$$RF_I = 0.54$$

$$RF_O = 1.0$$

The basic load rating controlling RFs were 0.45 for Inventory and 0.92 for Operating. The new rating factors represent a 20 percent increase for Inventory and an 8.7 percent increase for Operating. Table 8.4 compares the controlling RFs using the FEM model live load moments to the controlling RFs determined in the basic load rating. TxDOT's On-System Load Rating flowchart (TxDOT 2018b) allows the posting to be removed when the HS20 operating factor is greater than or equal to one and the substructure is in good condition based on NBI substructure condition rating items (item 58, 59 60, and 62). Therefore, the posting could be removed for Bridge CS-9 based on this refined load rating analysis.

Table 8.4. FEM Live Load RF Comparison

Rating Factor	Basic Load Rating	Load Rating with FEM Live Load	FEM Live Load /Basic Load Rating
Inventory	0.45	0.54	1.20
Operating	0.92	1.0	1.09

8.6 LOAD RATING ANALYSIS CONSIDERING FEM LIVE LOAD MOMENTS AND UPDATED MATERIAL PROPERTIES

This section shows a refined load rating analysis performed for Bridge CS-9 considering measured concrete compressive strength in moment capacity calculations, and live load moment demands from a calibrated FEM model that considers certain level of end restraint based on field measurements. This refined load rating was performed following the Load Factor Rating (LFR) procedures laid out in the AASHTO Manual for Bridge Evaluation (MBE) (AASHTO 2018). The results of this refined load rating are compared to the results of the basic load rating analysis.

8.6.1 Bridge Characteristics

Bridge CS-9 is a two-lane bridge with a total length of 75 ft. The bridge was designed to include three simply supported spans with a controlling span for load rating of 25 ft. The bridge width is 21 ft 4 in. with a roadway width of 20 ft. The TxDOT HS-20 RFs are 0.445 for Inventory and 0.935 for Operating. The transverse section of Bridge CS-9 is shown in Figure 8.1.

Span Length: $L = 24$ ft

Bridge Width: $W = 21.33$ ft

Concrete Compressive Strength: $f'_c = 2.5$ ksi

Steel yield strength: $f_y = 33$ ksi

Concrete Density: $\gamma_c = 150$ pcf

Asphalt Density: $\gamma_{ws} = 144$ pcf

Asphalt Thickness: $t_{ws} = 3$ in.

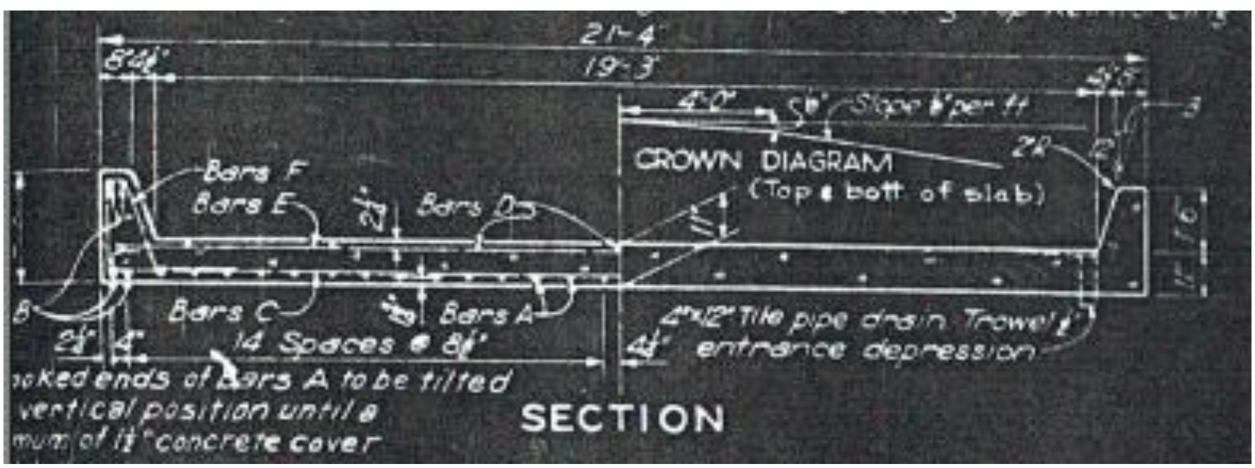

Figure 8.7. Transverse Section of Bridge CS-9 (TxDOT 2018a)

8.6.2 Properties of Concrete Slab

Design Slab Width: $b = 12.0$ in.

Slab Depth: $t_{slab} = 11.0$ in.

Tensile Reinforcement Diameter: $d_b = 1.0$ in.

Reinforcement Spacing: $s = 8.5$ in.

Tensile Reinforcement Area: $A_s = 1.115\ \frac{\text{in}^2}{\text{ft}}$

Bottom Cover: $c_{bot} = 1.75$ in.

8.6.3 Properties of Concrete Curb

Top width: $W_{top} = 8.00$ in.

Bottom width: $W_{bot} = 12.50$ in.

Tensile Reinforcement Dimension: 1.25 in. (sq)

No. of Reinforcing Bars: $N_b = 2$

Tension Steel Area: $A_{s-curb} = 3.125\ \text{in}^2$

Bottom Cover: $c_{bot-curb} = 1.75$ in.

Compression Reinf. Dimension: 1.25 in. (sq)

No. of Reinforcing Bars: $N_b = 2$

Compression Steel Area: $A'_{s-curb} = 3.125\ \text{in}^2$

Top Cover: $c_{top-curb} = 2.25$ in.

Curb height: $h = 18.00$ in.

8.6.4 Moment Capacity

Detailed in Section 8.3.4, the final reduced moment capacity ϕM_n of the slab section is:

$$\phi M_n = 24.6\ \frac{\text{kip-ft}}{\text{ft}}$$

The total reduced nominal moment of the curb section is:

$$\phi M_{n,L-curb} = 424.7\ \text{kip–ft}$$

8.6.5 Structural Analysis for Moment Demand

8.6.5.1 Applied Live Load Moment

The Impact Factor I is applied to the live load to allow for dynamic, vibratory, and impact effects. From AASHTO Standard Specifications (2002) Article 3.8.2.1, the impact factor is equal to,

$$I = \frac{50}{L + 125} \leq 0.3 \tag{8.103}$$

$$= \frac{50}{24+125} \leq 0.3$$

$$= 0.336 \leq 0.3$$

$$I = 0.3$$

The applied live load moment on the mid-slab region between L-curb sections without the impact factor $M_{Slab,LL}$ is obtained from the original FEM model as:

$$M_{Slab,LL} = 120.0 \text{ kip–ft}$$

The applied live load moment on the L-curb section without the impact factor $M_{LL,L-curb}$ is obtained from the original FEM model as:

$$M_{LL,L-curb} = 149.0 \text{ kip–ft}$$

Therefore, the applied live load moment with dynamic effects for the slab and curb can be calculated as:

$$\begin{aligned} M_{Slab,LL} &= \frac{M_{Slab,LL}}{(W - 2(W_{bot} + 4t_{slab}))}(1 + I) \\ &= \left(\frac{120.0 \text{ kip–ft}}{\left((21.33 \text{ ft}) - 2(1.04 \text{ ft} + 3.67 \text{ ft})\right)}\right)(1.3) \\ &= 13.1 \ \frac{\text{kip-ft}}{\text{ft}} \end{aligned} \tag{8.104}$$

$$\begin{aligned} M_{LL,L-curb} &= M_{LL,L-curb}(1 + I) \\ &= (149.0)(1.3) \\ &= 193.7 \text{ kip-ft} \end{aligned} \tag{8.105}$$

8.6.5.2 Dead Load Moment of Structural Components

Detailed in Section 8.3.5.2, the slab self-weight w_S is:

$$w_S = 0.167 \ \frac{\text{kip}}{\text{ft}}$$

The self-weight of the curbs w_{Curb} is:

$$w_{Curb} = 0.335 \ \frac{\text{kip}}{\text{ft}}$$

8.6.5.3 Superimposed Dead Load Moment

The superimposed dead load per unit width due to the wearing surface w_{SDL} is:

$$w_{SDL} = 0.036 \ \frac{\text{kip}}{\text{ft}}$$

8.6.5.4 Total Dead Load Moment

Detailed in Section 8.1.5.4, the total applied moment due to structural dead loads and superimposed dead loads $M_{DL,Slab}$ for the slab is:

$$M_{DL,Slab} \quad = \quad 6.1 \ \frac{\text{kip-ft}}{\text{ft}}$$

The total applied moment due to structural dead loads and superimposed dead loads $M_{DL,L-curb}$ for the L-curb is:

$$M_{DL,L-curb} \quad = \quad 108.0 \text{ kip–ft}$$

8.6.6 LFR Load Rating for Flexural Strength

Rating Factor Equation:

$$RF = \frac{C - A_1 D}{A_2(L + I)} \tag{8.106}$$

Inventory Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 2.17$

Slab Rating:

$$\begin{aligned} RF_{ISlab} &= \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \qquad (8.107) \\ &= \frac{24.6 - [(1.3)(6.1)]}{(2.17)(13.1\ )} \\ &= 0.58 \end{aligned}$$

Curb Rating:

$$\begin{aligned} RF_{IL-curb} &= \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \qquad (8.108) \\ &= \frac{424.7 - [(1.3)(108.0)]}{(2.17)(193.7)} \\ &= 0.68 \end{aligned}$$

The inventory rating for Bridge CS-9 is the minimum of the two RFs (TxDOT 2018b).

$$
\begin{aligned}
RF_I &= Min(RF_{ISlab}, RF_{IL-curb}) \\
&= Min(0.56, 0.71) \\
&= 0.58
\end{aligned}
\tag{8.109}
$$

Does not pass.

Operating Rating

Dead Load Factor, $A_1 = 1.3$

Live Load Factor, $A_2 = 1.3$

Slab Rating:

$$
\begin{aligned}
RF_{OSlab} &= \frac{\phi M_n - A_1 M_{DL,slab}}{A_2 M_{LL,slab}} \\
&= \frac{24.6 - [(1.3)(6.1)]}{(1.3)(13.1)} \\
&= 0.98
\end{aligned}
\tag{8.110}
$$

Curb Rating:

$$
\begin{aligned}
RF_{OL-curb} &= \frac{\phi M_{n,L-curb} - A_1 M_{DL,L-curb}}{A_2 M_{LL,L-curb}} \\
&= \frac{424.7 - [(1.3)(108.0)]}{(1.3)(193.7)} \\
&= 1.13
\end{aligned}
\tag{8.111}
$$

The operating rating for Bridge CS-9 is calculated as (TxDOT 2018b):

$$
\begin{aligned}
RF_O &= \frac{2\left(RF_{OL-curb}(W_{bot} + 4h)\right) + \left(RF_{OSlab}\left(W - 2(W_{bot} + 4h)\right)\right)}{W} \\
&= \frac{2\left(1.13(12.5 + 4(11))\right) + \left(0.98\left(255.6 - 2(12.5 + 4(11))\right)\right)}{255.6} \\
&= 1.05
\end{aligned}
\tag{8.112}
$$

Passes.

8.6.7 Controlling Rating Factors

The controlling LFR rating factors come from the strength check. The controlling rating factors for Bridge CS-9 from the Level II end restraint analysis are equal to:

$$RF_I = 0.58$$

$$RF_O = 1.05$$

The basic load rating controlling RFs were 0.45 for Inventory and 0.94 for Operating. The new rating factors represent a 24 percent increase for Inventory and a 14 percent increase for Operating. Table 8.5 compares the controlling RFs using the calibrated FEM live load moment to the controlling RFs determined in the basic load rating. TxDOT's On-System Load Rating flowchart (TxDOT 2018b) allows the posting to be removed when the HS20 operating factor is greater than or equal to one and the substructure is in good condition based on NBI substructure condition rating items (item 58, 59 60, and 62). Therefore, the posting could be removed for Bridge CS-9 based on this refined load rating analysis.

Table 8.5. Calibrated FEM Live Load RF Comparison

Rating Factor	Basic Load Rating	Load Rating with Calibrated FEM Live Load	Calibrated FEM Live Load /Basic Load Rating
Inventory	0.45	0.58	1.29
Operating	0.92	1.05	1.14

9 SUMMARY AND CONCLUSIONS

The objective of this research project was to determine appropriate strategies for bridge load rating to reduce uncertainty, which can lead to removal of load postings for Texas bridges posted at load levels below the legal limit. The refined load rating calculations using more accurate information and techniques presented in this research are expected to provide better accuracy in load rating and can potentially eliminate load postings or increase the allowable loads on load posted bridges. In particular, this project focused on substandard for load only (SSLO) bridges in Texas. SSLO bridges are a subset of the load posted bridge inventory and, while they have a load capacity below the legal limit, they are not considered structurally deficient or functionally obsolete.

The Volume 1 Report (Hueste et al. 2019a) fully documents a review of the state-of-the-practice and state-of-the-art for load rating of existing bridges, a review and synthesis of the bridge characteristics of load posted bridges in Texas, and the basic load rating analysis for selected representative bridges to identify the controlling limit states and areas of opportunities that likely lead to a reduced operating load for typical bridge structures.

The Volume 2 Report (Hueste et al. 2019b) presents the refined analysis procedures and results for more accurate LLDF predictions, fully documents the field-testing and measured results, and discusses FEM model updating and calibration for the selected typical bridge types.

This Volume 3 Report documents the details of the developed refined load rating guidelines for the four selected typical SSLO bridges types, and provides detailed refined load rating examples for each bridge type. The effect of each refinement on the revised load ratings has been evaluated and their implications for potentially increasing the posted loads or removal of load posting have been discussed.

9.1 RECOMMENDED PROCEDURES FOR REFINED LOAD RATING ANALYSIS

The following recommendations have been developed for the four different bridge types reviewed in detail in this research. Note that the four specific bridges considered in this research were selected as representative bridges among the SSLO bridges in Texas, and the findings and guidance are based on the results for the particular geometries considered. General applicability of the findings should be considered in a case by case basis.

9.1.1 Recommendations for Steel Multi-Girder Bridges

Based on the research findings for the selected SSLO steel multi-girder bridges, the following recommendations have been developed.

1. It is recommended to use the most accurate material property information available for capacity calculations during the load rating process. Material properties can be determined using suitable NDE techniques based on standard test procedures and through standard laboratory testing of extracted samples to obtain more accurate material data. Information regarding the reinforcing steel grade may also be determined from mill test certificates, if available, so the corresponding yield strength of steel for design may be used for load rating.
2. Steel multi-girder bridges with a roadway width under 24 ft, experiencing a low ADTT, and with low likelihood of two design trucks passing each other on the bridge at the same time could be analyzed as a one-lane bridge, using one-lane LLDFs, if TxDOT deems appropriate. A bridge meeting these criteria can be re-striped as a one-lane bridge where this does not impede functionality or safety.
3. Three levels of analysis are proposed to consider partial composite action in the load rating process.
    - A Level I analysis pertains to bridges with the girder top flanges embedded into the deck and without significant cracking at the top flange to deck interface. A Level I analysis can then be performed to assess the potential benefit of composite action that is likely to be present in this case. This will inform the need for additional verification of the composite bridge behavior.
    - A Level II analysis involves the use of a load test to confirm the behavior of a bridge. In this case, it is assumed that the flexural response is not be significantly affected by end restraint. Therefore, a Level II analysis is performed by measuring midspan deflection under a known load and configuration to provide in-situ response data to determine the level of composite action.

- A Level III analysis involves the use of a load test to confirm the behavior of a bridge that may have some amount of end restraint causing a restraining moment at the ends of the girders. Therefore, a Level III analysis is performed by measuring midspan deflection and girder end restraint (such as strain) under a known load and configuration to provide in-situ response data to determine the level of composite action.

4. Two levels of analysis are proposed to consider the effect of end restraint in the load rating process for steel multi-girder bridges having a rating factor close to 1.0.
    - A Level I analysis is performed without conducting a load test for bridges that show signs of possible end restraint. For this analysis upper and lower bound rating factors are first calculated using fully-fixed and simply-supported support conditions. This analysis is used to determine whether some degree of end restraint would be sufficient to increase the rating factor to an acceptable level. If so, further verification of the in-situ conditions should be conducted as part of a Level II analysis.
    - A Level II analysis involves the use of a load test to confirm the bridge behavior. The end moments are measured during the load test and can be considered for refined load rating calculations.
5. The live load distribution factors calculated using the approximate equations for steel multi-girder bridges in the *AASHTO Standard Specifications* (AASHTO 2002) were found to be accurate with a reasonable level of conservatism. Therefore, it is recommended to continue using the AASHTO Standard Specifications LLDFs in load rating calculations. However, refined finite element analysis or load testing can be considered as a more refined analysis for important bridges, or when a bridge is close to having an acceptable rating and a slight reduction in LLDFs may be sufficient to allow the desired load level. When using proper modeling parameters, commercial software allows this to be done in an expedient manner.
6. For continuous steel multi-girder bridges, it is recommended to use a multi-span structural analysis to determine moment demands. The effect of continuity along with the potential for load patterning effects should be considered. When using proper modeling parameters, commercial software allows this to be done in an expedient manner.

9.1.2 Recommendations for Simple-Span Concrete Multi-Girder Bridges

Based on the research findings for the selected SSLO simple-span concrete multi-girder bridges, the following recommendations have been developed.

1. It is recommended to use the most accurate material property information available for capacity calculations during the load rating process. Material properties can be determined using suitable NDE techniques based on standard test procedures and through standard laboratory testing of extracted samples to obtain more accurate material data. Information regarding the reinforcing steel grade may also be determined from mill test certificates, if available, so the corresponding yield strength of steel for design may be used for load rating.
2. Simple- span concrete multi-girder bridges with a roadway width under 24 ft, experiencing a low ADTT, and with low likelihood of two design trucks passing each other on the bridge at the same time could be analyzed as a one-lane bridge, using one-lane LLDFs, if TxDOT deems appropriate. A bridge meeting these criteria can be re-striped as a one-lane bridge where this does not impede functionality or safety.
3. Two levels of analysis are proposed to consider partial end restraint in the load rating process for concrete multi-girder bridges having a rating factor close to 1.0.
    - A Level I analysis is first performed for bridges that show signs of possible end restraint. For this analysis upper and lower bound rating factors are first calculated using fully-fixed and simply-supported support conditions. This analysis is used to determine whether some degree of end restraint would be sufficient to increase the rating factor to an acceptable level. If so, further verification of the in-situ conditions should be conducted as part of a Level II analysis.
    - A Level II analysis involves the use of a load test to confirm the bridge behavior. The end moments are measured during the load test and can be considered for refined load rating calculations.
4. The live load distribution factors calculated using the approximate equations for concrete multi-girder bridges provided in the *AASHTO Standard Specifications* (AASHTO 2002) were found to be accurate with reasonable level of conservatism. Therefore, it is recommended to continue using the AASHTO Standard Specification LLDFs in load rating calculations. However,

refined finite element analysis or load testing can be considered as a more refined analysis for important bridges, or when a bridge is close to having an acceptable rating and a slight reduction in LLDFs may be sufficient to allow the desired load level. When using proper modeling parameters, commercial software allows this to be done in an expedient manner.

9.1.3 Recommendations for Simple-Span Concrete Slab Bridges with Integral Curbs

Based on the research findings for the selected SSLO simple-span concrete slab bridges with integral curbs, the following recommendations have been developed.

1. It is recommended to use the most accurate material property information available for capacity calculations during the load rating process. Material properties can be determined using suitable NDE techniques based on standard test procedures and through standard laboratory testing of extracted samples to obtain more accurate material data. Information regarding the reinforcing steel grade may also be determined from mill test certificates, if available, so the corresponding yield strength of steel for design may be used for load rating.
2. Simple-span concrete slab bridges with integral curbs (FS bridges) with a roadway width under 24 ft, experiencing a low ADTT, and with low likelihood of two design trucks passing each other on the bridge at the same time could be analyzed as a one-lane bridge, using one-lane LLDFs, if TxDOT deems appropriate. A bridge meeting these criteria can be re-striped as a one-lane bridge where this does not impede functionality or safety.
3. The moment distribution to the L-curbs and mid-slab portion of the bridge section calculated using the Illinois Bulletin 346 (Jenson et al. 1943) approach was reviewed based on the field test results for Bridge CS-9.
    - The IB346 procedure was found to be accurate with a reasonable level of conservatism when estimating the live load moment demand for the L-curbs; however, it was unconservative for estimating the live load moment demand for the slab region.
    - The IB346 method may be used to determine the distribution of live load to the L-curb sections that are defined by IB346.
    - The distribution of moment to the mid-slab region should be found using the equivalent width for concrete slab bridges given in the AASHTO LRFD Specifications (AASHTO 2017),

specifically when this approach provides a higher moment estimate in comparison to the IB346 method.

- As an alternative for the one-lane loading case, the equivalent width recommendations for slab bridges with integral edge beams proposed by Amer et al. (1999) may be used. Again, this approach should be compared to the IB346 method and the larger slab moment demand should be used.

4. Two levels of analysis are proposed to consider partial end restraint in the load rating process for simple-span concrete slab bridges with integral curbs having a rating factor close to 1.0.
   - A Level I analysis is first performed for bridges that show signs of possible end restraint. For this analysis upper and lower bound rating factors are first calculated using fully-fixed and simply-supported support conditions. This analysis is used to determine whether some degree of end restraint would be sufficient to increase the rating factor to an acceptable level. If so, further verification of the in-situ conditions should be conducted as part of a Level II analysis.
   - A Level II analysis involves the use of a load test to confirm the bridge behavior. The end moments are measured during the load test and can be considered for refined load rating calculations.

9.2 REFINED LOAD RATING EXAMPLES

Several refined load rating examples have been developed for the four selected bridges considered for more detailed testing and analysis in this research project. The examples show the basic and refined load rating procedures and discuss the relative improvements for each refined load rating recommendation. The following subsections provide a summary and findings from the load rating examples for different bridge types.

9.2.1 Examples for Simple-Span Steel Multi-Girder Bridge (Bridge SM-5)

1. The basic load rating analysis for flexure of the two-lane Bridge SM-5 was carried out using simply supported boundary conditions and considering the girders as non-composite. The

resulting rating factors were 0.49 and 0.81 for inventory rating and operating level rating, respectively.

2. Analyzing the Bridge SM-5 as a one-lane bridge increases the rating factor by about 30 percent compared to the two-lane-loaded basic load rating analysis.
3. The Level II analysis considering partial composite action, which was found to be 88 percent of full composite action, based on the field-measured deflection of an interior girder, increases the rating factors by approximately 100 percent as compared to the basic load rating analysis using the non-composite girder strength.
4. The Level II analysis considering partial end restraint based on the field-measured strains resulted in a low amount of end restraint that is approximated as a small negative moment at the girder ends. Including end restraint does not significantly increase the rating factors, which increased by only about two percent as compared to the basic load rating analysis with simply supported boundary conditions.
5. Because the behavior of Bridge SM-5 is very close to that for simply supported boundary conditions, the combined effect of partial composite action and end restraint increases the rating factor by about the same amount as the case considering only the partial composite action.

9.2.2 Examples for Continuous Steel Multi-Girder Bridge (Bridge SC-12)

1. The basic load rating analysis for flexure of the two-lane Bridge SC-12 was carried out using a three-span continuous beam analysis and assuming non-composite girders. The resulting rating factors were 0.54 and 0.91 for inventory and operating level rating, respectively.
2. The Level II analysis considering partial composite action, which was found to be 66 percent of full composite action, based on the field-measured deflection of an interior girder, increases the rating factors by approximately 10 percent as compared to the basic load rating analysis using the non-composite girder strength.

9.2.3 Examples for Simple-Span Concrete Multi-Girder Bridge (Bridge CM-5)

1. The basic load rating analysis for flexural strength of the two-lane Bridge CM-5 was carried out using simply supported boundary conditions. The resulting rating factors were 0.42 and 0.69 for inventory and operating level rating, respectively.
2. Conducting the load rating analysis by assuming one-lane-loading increases the rating factors by approximately 10 percent.
3. Refined load rating analysis by considering the field measured concrete compressive strength of 7 ksi instead of the design value of 4 ksi increases the rating factors for flexural strength by only 3 percent.
4. Refined load rating analysis that considers the slight end restraint observed during testing increases the rating factors by only approximately 1–2 percent.
5. Refined load rating analysis using more accurate live load bending moment predictions from the calibrated FEM analysis using field measurements, which considers the effect of the updated MOE of the concrete, modeled live load distribution, and updated boundary conditions due to slight end restraint, provides a slight reduction in the rating factors. Although the effect of the updated MOE of the concrete coupled with slight end restraint should slightly reduce the midspan moment, the LLDFs from the refined FEM analysis indicate a higher midspan moment as compared to approximate LLDFs from the *AASHTO Standard Specifications*; thereby, reducing the rating factors in comparison to the basic load rating analysis.

9.2.4 Examples for Simple-Span Concrete Slab Bridge with Integral Curbs (Bridge CS-9)

1. The basic load rating analysis for flexural strength of the two-lane Bridge CS-9 was carried out by using simply supported boundary conditions. The resulting rating factors were 0.45 and 0.92 for inventory and operating level rating, respectively
2. Refined load rating analysis considering the field measured concrete compressive strength of 5.2 ksi instead of the AASHTO MBE value of 2.5 ksi increases the rating factors by approximately 5–7 percent.

3. The Level II analysis considering partial end restraint based on the field-measured strains resulted in a very small amount of end restraint that is approximated as a small negative moment at the girder ends. The end restraint observed during the field tests did not change the midspan moment demands significantly. Therefore, refined load rating analysis considering the effect of end restraint did not change the rating factors determined by the basic load rating analysis.
4. Refined load rating analysis, which considers measured concrete compressive strength in the moment capacity calculations, and live load moment demands from a calibrated FEM model, which considers the effect of the updated MOE of the concrete, modeled live load distribution, and updated boundary conditions due to the small level of end restraint based on field measurements, increased the inventory rating factor by 29 percent and operating rating factor by 14 percent.

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