



Development of a Strategy to Address Load-Posted Bridges through Reduction in Uncertainty in Load Ratings—Volume 1: Basic Load Rating

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16. Abstract According to the 2016 National Bridge Inventory data, the state of Texas houses over 2100 bridges posted at load levels below the legal limit. While load postings are generally a management issue, there can be commerce, traffic, and emergency egress issues. For this reason, removing postings is always of interest. Posted structures come in all shapes and sizes, were 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 were developed to reduce uncertainty in load rating procedures in a safe and appropriate manner to potentially increase or remove the load postings of typical steel and concrete bridges. The first phase of the project focused on the detailed review and synthesis of the relevant literature, analysis of the load-posted bridge inventory in Texas, and conducting basic load rating analysis of selected typical bridge types to identify and examine controlling limit states and areas of opportunities that could lead to an increase or removal of load posting. This Volume 1 research report fully documents the state-of-the-practice and state-of-the-art on load rating of existing bridges, the review and synthesis of the bridge characteristics of load-posted bridges in Texas, and the basic load rating for selected representative bridges to identify the controlling limit states that likely lead to an increased operating load for these bridges.					
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**DEVELOPMENT OF A STRATEGY TO ADDRESS LOAD-POSTED BRIDGES
THROUGH REDUCTION IN UNCERTAINTY IN LOAD RATINGS—VOLUME 1:
BASIC LOAD RATING**

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of FHWA or TxDOT. This report does not constitute a standard, specification, or regulation.

This report is not intended for construction, bidding, or permit purposes. The researcher in charge of the project was Mary Beth D. Hueste. The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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1 INTRODUCTION

1.1 BACKGROUND AND SIGNIFICANCE

An ever-increasing demand on highways for improved mobility and connectivity for delivering more goods and services increases the importance of a reliable, well-maintained transportation infrastructure. Maintaining the functionality and health of the transportation infrastructure depends on the successful management of aging bridge assets. Departments of transportation (DOTs) 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. 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. While load postings are generally a management issue, there can be commerce, traffic, and emergency egress issues. Therefore, removing postings is always of interest. However, posted bridges vary greatly in terms of geometry, size, construction style, age, and environmental conditions; their structural behavior can also differ significantly. 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 the bridge load rating through reduction in uncertainty that can lead to removal of load postings for Texas bridges posted at load levels below the legal limit. 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 American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation* (MBE) (AASHTO 2018) 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 for less conservative load

distribution modeling, 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 the Texas Department of Transportation's (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

This Volume 1 report 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. 2019a) 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.

The Volume 3 report (Hueste et al. 2019b) discusses the developed guidelines for refined load rating and provides several examples.

1.4 REPORT OUTLINE

This Volume 1 report consists of eight main chapters that document the findings of Tasks 2–4.

Chapter 1 presents the background and significance, research objectives and scope of the project, the research plan (including specific tasks), and outlines the Volume 1 report.

Chapter 2 summarizes a review of the state-of-the-practice and state-of-the-art on load rating of existing bridges. The state-of-the-practice describes historical and current approaches to bridge load rating for the state of Texas, along with national and select international practices. In addition, a literature review of previous related research published in journals, conferences, and agency reports—both domestic and international—was conducted, and key findings are documented. The compiled information was used to identify possible issues and areas of improvement with respect to load rating.

Chapter 3 focuses on a review of the population of load-posted bridges in Texas to determine the characteristics that are most typical among these bridges. In particular, details for sub-standard for load only (SSLO) bridges are provided because these bridges are the primary candidates to benefit from increases in load rating since they are more likely to remain in service for a longer period of time. Key characteristics were reviewed and synthesized to provide an overall understanding of load-posted bridge parameters. These parameters include kind of material and/or design (concrete, steel, wood, etc.), type of design and/or construction (multi-girder, slab, tee beam, box girder, etc.), year built, maximum span length, total bridge length, operating rating, and ratio of operating rating to maximum legal load. As a result of this investigation, SSLO bridges that are load posted based on the AASHTO MBE (AASHTO 2018) and having typical parameters were identified from the following four bridge categories: (1) steel multi-girder (SM), (2) concrete slab (CS), (3) concrete multi-girder (CM), and (4) continuous steel multi-girder (SC) bridges. The selected subset of bridges includes 25 SM bridges, 23 CS bridges, 14 CM bridges, and 16 SC bridges based on simple random selection to obtain a representative subset within each bridge type and further selection of several additional on-system bridges. The specific bridges identified are listed in Appendix A, and their mapped locations are also shown in Appendix A.

Chapter 4 summarizes the current load rating procedures that are provided in the AASHTO MBE (AASHTO 2018). This chapter first discusses some minor updates in the most recent edition (3rd) of the MBE that affect the load ratings. The three main load rating procedures—(1) Load and Resistance Factor Rating (LRFR), (2) Allowable Stress Rating (ASR), and (3) Load Factor Rating (LFR)—are discussed in detail. Current TxDOT load rating practices for on- and off-system bridges are also included.

Chapter 5 focuses on the basic load rating analysis for the selected steel bridges and evaluates the areas of opportunities to increase or remove the load postings. Basic load rating

analysis was conducted for the selected subset of 25 simple-span steel multi-girder bridges and 16 continuous steel multi-girder bridges by following the procedures specified in the AASHTO MBE (AASHTO 2018) to understand and identify the controlling limit states that lead to a reduced operating load for these bridges. Based on the review and synthesis of SSLO bridges and the basic load ratings of selected typical bridges, several areas of opportunities were identified.

Chapter 6 focuses on the basic load rating analysis for the selected concrete bridges and evaluates the areas of opportunities to increase or remove the load postings. This chapter provides the analysis results for the 23 simple-span concrete slab bridges and 14 simple-span concrete multi-girder bridges and evaluates the possible areas of opportunities to increase the rating based on the controlling limit states.

Chapter 7 presents the details of four selected typical bridges for further refined analysis in Task 4, including one bridge from each of the four prevalent bridge types. The selection was done based on the synthesis and review of the inventory and basic load rating analysis.

Chapter 8 provides a summary of findings from each of the tasks reported in this Volume 1 report.

2 LITERATURE REVIEW

2.1 INTRODUCTION

Prior to conducting the analysis and testing associated with this project, a review of background information related to bridge load rating and testing was conducted. The American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI)-AASHTO ad-hoc group (2009) on bridge inspection, rating, rehabilitation, and replacement noted that load rating a bridge is a critical process. The load rating must be performed to a level of accuracy required for intended purposes, such as load posting, repairs, and so forth, and load ratings have to be reliable, uniform, and consistent. Additionally, documenting load ratings for both the bridge and the critical elements provides more value for effective bridge management.

This chapter summarizes a review of the state-of-the-practice and state-of-the-art on load rating of existing bridges. The state-of-the-practice describes historical and current approaches to bridge load rating for the state of Texas, along with national and select international practices. The history of bridge load rating review is focused on the initiation of bridge load rating, the significant contents and procedures in load rating sections of early bridge evaluation manuals, and how these early manuals developed into the current bridge load rating manual. The review of the state-of-the-practice describes the requirements of the current load rating manual, load rating practices that state DOTs currently use, and load rating studies for which states have implemented the results. The state-of-the-art literature review summarizes related research published in journals, conferences, and agency reports, and key findings relevant to this project are documented.

2.2 HISTORY OF BRIDGE LOAD RATING

2.2.1 Early Manuals

In 1967, the Silver Bridge collapsed into the Ohio River in Point Pleasant, West Virginia, resulting in the deaths of 46 people. Prior to this bridge collapse, inspection of bridges and load rating of bridges were rare. However, the collapse led the US Congress to add a section requiring a national bridge inspection standard to the Federal Highway Act of 1968. The new National Bridge Inspection Standards (NBIS) required an increased effort and diligence regarding bridge

inspections. These events led to the creation of three manuals to ensure uniformity in state bridge inspection procedures (Lichtenstein 1993; Ryan et al. 2012).

Released in 1970, one of these manuals was the American Association of State Highway Officials (AASHO) *Manual for Maintenance Inspection of Bridges* (AASHO 1970). This manual was one of the first to provide overarching guidelines for bridge load rating. It was based on the concept of working stresses, and these stresses caused by loading were not allowed to exceed a specified maximum allowable stress, which depended on the level of rating that was being performed. The upper stress level rating was called operating rating, in which the allowable stress was usually not allowed to exceed 75 percent of the yield stress of the material. The operating rating was defined as the absolute maximum stress level the bridge was allowed to experience. The lower stress level rating was called inventory rating, in which the allowable stress was not allowed to exceed 55 percent of the yield stress or the allowable design stress used in the original design of the structure. The inventory rating was defined as the load that can travel on the bridge safely for an indefinite period of time. Bridges were evaluated using either the AASHO H design loading, or one of three conventional vehicle types, called the Type 3 Unit, the Type 3S2 Unit, and the Type 3-3 Unit. These three trucks are very similar to the current AASHTO *LRFD Bridge Design Specifications* (AASHTO 2017) legal loads, with minor changes. They are slightly less heavy than the AASHTO legal loads, and different amounts of load are distributed to each axle.

AASHO became AASHTO in 1973, and over the next 13 years released three more editions of the *Manual for Maintenance Inspection of Bridges* that were similar in length and layout to the 1970 version but had some significant changes. The 1974 *Manual for Maintenance Inspection of Bridges* (AASHTO 1974) added the HS design loading to the list of loads required for bridge evaluation. It also introduced rating by Load Factor Design (LFD) methods for use on steel girder bridges. Load Factor Rating applied multiples of load or load effects to the bridge. These factors were different for operating and inventory ratings, with the inventory rating having an increased live load factor and applying a larger effect to the bridge. The capacity of the bridge was then determined based on the yield strength of steel or ultimate strength of concrete by using different equations depending on the type of structure and element being examined. A rating factor (RF) equation was then developed for each type of bridge, using some variation of capacity divided by the live load effect. However, although the Load Factor method of rating bridges was introduced,

it was only included in the appendix, and the main focus of the 1974 manual was still load rating using working and allowable stresses.

In the 1978 *Manual for Maintenance Inspection of Bridges* (AASHTO 1978), the Working Stress and Load Factor methods for rating were more equally discussed. The Load Factor method was moved out of the appendix of the manual and into the main load rating section. Use of the Load Factor method for rating concrete bridges was also briefly introduced, with the tension steel reinforcing assumed to yield. Because both methods were now included in the main body of the manual, the definitions of operating and inventory levels were changed to only be related to the load levels that the bridge was subjected to, with no reference to stresses. Also, while the H and HS loadings were still the design loads, the three legal loads for evaluation were slightly increased and redistributed among axles to obtain the legal load configurations that are still used currently. The load rating section of the 1983 *Manual for Maintenance Inspection of Bridges* remained essentially the same as the load rating section of the 1978 version, continuing to provide guidelines for both the working stress and load factor methods of rating (AASHTO 1983; TxDOT 2013).

2.2.2 Introduction of Load and Resistance Factor Rating Methods

In 1987, the National Highway Cooperative Research Program (NCHRP) released two reports that helped develop and introduce a new method for rating bridges called Load and Resistance Factor Rating. NCHRP Report 292 introduced the use of probabilistic and structural reliability methods to develop load factors used in rating equations for reinforced concrete bridges (Imbsen et al. 1987). NCHRP Report 301 expanded on this method to develop load factors that would apply to all types of bridges and proposed a new rating equation that incorporates these factors and Load and Resistance Factor Design (LRFD) principles (Moses and Verma 1987). In 1993, a comprehensive specifications guide for LRFD that included calibrated load factors was accepted by AASHTO (Kulicki 1998).

The associated LRFR method is first mentioned in the load rating section of an AASHTO evaluation manual in the 1994 *AASHTO Manual for Condition Evaluation of Bridges* (MCE) (AASHTO 1994b). The 1994 MCE was the result of a complete rewrite of the AASHTO evaluation manual and included much more information on load rating and more detail than in earlier manuals. However, the LRFR method is only mentioned briefly in the MCE, which noted that the LRFR method can be used to load rate bridges and that the process was described in detail

in a document called the *AASHTO Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges* (AASHTO 1994c). Otherwise, the 1994 MCE focuses on the Working Stress and Load Factor methods, which were given updated titles of Allowable Stress and Load Factor methods. Besides the addition of a simplified, all-encompassing RF equation to use for both methods, the content is effectively the same as previous manuals, with more detail. The design truck loading was also changed to the HS20 truck in the 1994 MCE to conform to the *AASHTO LRFD Bridge Design Specifications*.

NCHRP Project C12-46 (Lichtenstein Consulting Engineers Inc. 2001) was tasked with developing a new AASHTO manual for load rating that incorporated LRFR methods; it was completed in 2001. This project led to the publication of the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* (AASHTO 2003). This manual had an entire section on LRFR, introducing it as the main load rating method and including a general RF equation and procedures to find the load factors. The ASR and LFR methods were still included, but as an appendix to the LRFR section titled “Alternate Load Rating.”

In 2008, AASHTO released the first edition of MBE. The MBE combined the material in the 1994 MCE and the 2003 manual to create an all-encompassing evaluation manual for bridges that superseded all previous manuals. The MBE had an entire section dedicated to load rating made up of three parts: one focused on the LRFR method, one on the ASR and LFR methods, and one provided examples of each (AASHTO 2008). The MBE became the required reference for load rating bridges in the United States.

2.3 CURRENT NATIONAL LOAD RATING PROCEDURES

In 2011, AASHTO released the second edition of the MBE (2011). This manual, along with revisions made in 2016, contains the current standard procedure to load rate bridges in the United States. Section 6 of the MBE describes different load rating methods and has two parts: Part A covers the LRFR method, and Part B covers the ASR and LFR methods. The Federal Highway Administration (FHWA) requires that all bridges constructed after October 1, 2007, be designed using LRFD. This requirement is part of a large push by the industry to move toward the use of LRFD to design bridges and therefore use LRFR as the accompanying load rating method (CTC & Associates 2009). This effort also requires bridges designed using Allowable Stress Design (ASD) to be re-rated using either LFR or LRFR. Bridges designed using LFD may still be rated

using LFR, although load rating using the LRFR method is preferred by the FHWA in all cases (Lwin 2006).

2.3.1 Load and Resistance Factor Rating

The general equation to obtain the bridge RF using the LRFR method is:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \quad (2.1)$$

where:

RF	=	Rating Factor
C	=	Capacity
DC	=	Dead load effect due to structural components and attachments
DW	=	Dead load effect due to wearing surface and utilities
P	=	Permanent loads other than dead loads
LL	=	Live load effect
IM	=	Dynamic load allowance
γ_{DC}	=	LRFD load factor for structural components and attachments
γ_{DW}	=	LRFD load factor for wearing surfaces and utilities
γ_P	=	LRFD load factor for permanent loads other than dead loads = 1.0
γ_{LL}	=	Evaluation live load factor

For the strength limit states, the capacity is determined as the nominal member resistance multiplied by an LRFD resistance factor ϕ , a system factor ϕ_s , and a condition factor ϕ_c . The product of the condition and system factors is required to be greater than 0.85. An RF is determined for each component of the bridge, and the member that has the lowest RF governs. The RF of the controlling member is then said to be the RF for the entire structure. The limit state that is chosen for determination of the RF gives specific values for γ_{DC} , γ_{DW} , and γ_{LL} . The primary limit state used for the LRFR method is Strength I; however, the limit state that is evaluated is dependent on the rating procedure used since some limit states only apply to specific rating procedures and not to others. LRFR includes three different rating procedures for bridges: (1) design load rating, (2) legal load rating, and (3) permit load rating.

2.3.1.1 Design Load Rating

Design load rating is the first assessment conducted in the LRFR bridge rating procedure and uses the HL-93 load model, which consists of an HS20 truck plus lane load, as defined in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017). The design load rating is split into two separate rating levels: (1) inventory rating, and (2) operating rating. The inventory rating level applies a higher live load factor (γ_{LL}) than the operating rating level, which gives an inventory RF that is lower than the operating RF. The inventory RF represents the multiple of HL-93 loads that may be applied to the bridge, such that the bridge can be in service for an indefinite period of time. Similarly, the operating RF represents the multiple of HL-93 loads that is the absolute maximum load that can safely travel on the bridge at any given time. If a bridge has an RF greater than 1.0 at both the inventory and operating Levels, it passes the design load rating and does not need to be posted. If a bridge passes at the operating level, but not at the inventory Level, the owner will likely need to post the bridge for its safe load. If a bridge does not pass at either the inventory or operating levels for the design load rating, it will be posted for loads lower than the HL-93 load and therefore must be evaluated for a legal load rating.

2.3.1.2 Legal Load Rating

Many states have legal loads, which are certain axle configurations and weights for trucks specific to their state, that are different than the HL-93 load model. These legal loads are applied to the structure and evaluated to determine the legal load rating. The *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017) also provide load configurations that are legal throughout the country, called Type 3, Type 3S2, and Type 3-3 trucks, and NRL, SU4, SU5, SU6, and SU7 specialized hauling vehicles. These legal loads produce smaller effects than the HL-93 loading and are therefore evaluated if the bridge does not pass the design load rating. The LRFR equation (Equation 2.1) is still used to determine the RF of the bridge for the group of legal loads. Truck configurations that produce an RF greater than 1.0 are safe to travel on the bridge, while configurations that produce an RF less than 1.0 must be posted. One of the other major differences between the design load rating and legal load rating is the value of the LRFR live load factor, γ_{LL} , which is 1.75 for inventory level and 1.35 for operating level design load ratings. However, for legal load ratings, this value is found through one of two tables that the AASHTO MBE provides

based on the truck configuration being evaluated and the Average Daily Truck Traffic (ADTT) on the bridge (Table 2.1).

Table 2.1. Generalized Live Load Factors for Routine Commercial Traffic and Specialized Hauling Vehicles (AASHTO 2011)

Traffic Volume (One Direction)	Load Factor for Type 3, Type 3S2, Type 3-3, and Lane Loads	Load Factor for NRL, SU4, SU5, SU6, and SU7 Vehicles
Unknown	1.80	1.60
ADTT \geq 5000	1.80	1.60
ADTT = 1000	1.65	1.40
ADTT \leq 100	1.40	1.15

Note: The MBE allows interpolation between ADTT values.

2.3.1.3 Permit Load Rating

Finally, if a bridge passes both the design load rating and the legal load rating, it can be evaluated using the permit load rating procedures. This procedure is used to determine the ability of the bridge to carry a certain load that is over the defined AASHTO or state legal loads. In this case, the bridge is evaluated for a specific load and axle configuration based on a permit request, not a particular family of trucks. Permits are issued as routine permits, which are valid for an unlimited number of trips over a specified period of time, or special permits, which are valid for only a single trip or limited number of trips. As before, if the bridge has an RF greater than 1.0 when evaluated for the specific permit request, it is rated as safe for that vehicle. However, the LRFR live load factor for permit load rating is also different from the design or legal load ratings. The live load factor is still dependent on the ADTT for the bridge, but the factors tend to be higher, which will decrease the RF of the structure (see Table 2.2).

2.3.1.4 LRFR Load Factors

The LRFR dead and live load factors for the design load rating were developed as part of NCHRP Project 12-33 (Kulicki 1998). This project was tasked with developing a design and analysis specification that used reliability indices to derive load and resistance factors. The load factors were selected to obtain reliability indices that were close to a target reliability. This target reliability is indirectly related to the probability of failure of the structure.

Table 2.2. Permit Live Load Factors Currently in the AASHTO MBE (AASHTO 2011)

Permit Type	Frequency	Loading Conditions	Distribution Factor	ADTT (one direction)	Load Factor by Permit Weight	
					Up to 100 kips	≥ 150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	> 5000	1.80	11.30
				= 1000	1.60	11.20
				< 100	1.40	11.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	> 5000	1.50	
				= 1000	1.40	
				< 100	1.35	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	> 5000	1.85	
				= 1000	1.75	
< 100				1.55		

Note: The MBE allows interpolation between ADTT values.

The live load factors for legal load rating (Table 2.1) and permit load rating (Table 2.2) were developed as part of NCHRP project 12-46 (Moses 2001). This project used essentially the same approach involving reliability indices to determine the necessary live load factors for the legal and permit trucks to keep the target reliability the same.

NCHRP project 12-78 (Mlynarski et al. 2011) followed up on these studies and examined the live load factors that were developed and are in current use in the MBE. In this project, researchers analyzed and compared LRFR and LFR ratings of 1500 bridges representing an array of ages, material types, and superstructure types. As part of NCHRP Report 700, released in 2011 and detailing the results of the project, new live load factors were proposed to replace the current ones in the AASHTO MBE (AASHTO 2011). Mlynarski et al. (2011) found that although a reliability index of 2.5 was used to calculate most of the live load factors that are currently in the MBE, based on their bridge database, those live load factors actually correspond to a reliability index of 3.5. The researchers recalculated the live load factors using their bridge database and a reliability index of 2.5 because they believed that the current MBE live load factors “would result in more bridges not passing the rating under LRFR and [were] thought to be too restrictive.” This process resulted in the lower live load factors that are shown in Table 2.3 and Table 2.4, compared to those shown in Table 2.1 and Table 2.2. The lower live load factors increased the RFs found using LRFR for the selected database of bridges. While significant research using reliability

analysis has been done to determine appropriate live load factors, the MBE also provides a procedure in its commentary for determining site-specific live load factors. However, this procedure is geared toward increasing live load factors due to heavier loads in use than those loads that were considered in the calibration studies. Some states have increased their live load factors using this procedure, while one state has actually decreased their live load factors. This process will be discussed later in this chapter.

Table 2.3. Live Load Factors for Routine Commercial Traffic and Specialized Hauling Vehicles Proposed in NCHRP Report 700 (Mlynarski et al. 2011)

Traffic Volume (One Direction)	Load Factor for Type 3, Type 3S2, Type 3-3, and Lane Loads	Load Factor for NRL, SU4, SU5, SU6, and SU7 Vehicles
Unknown	1.80 1.45	1.60 1.45
ADTT ≥ 5000	1.80 1.45	1.60 1.45
ADTT = 1000	1.65 1.30	1.40 1.30
ADTT ≤ 100	1.40 1.20	1.15 1.15

Note: strikethroughs indicate the live load factors provided in the referenced AASHTO MBE (AASHTO 2011).

Table 2.4. Live Load Factors for Permit Vehicles Proposed in NCHRP Report 700 (Mlynarski et al. 2011)

Permit Type	Frequency	Loading Conditions	Distribution Factor	ADTT (one direction)	Load Factor by Permit Weight	
					Up to 100 kips	1 ≥ 150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	> 5000	1.80 1.45	11.30
				= 1000	1.60 1.25	11.20
				< 100	1.40 1.05	11.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	> 5000	1.50 1.25	
				= 1000	1.40 1.15	
				< 100	1.35 1.10	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	> 5000	1.85 1.60	
				= 1000	1.75 1.50	
< 100				1.55 1.45		

Note: strikethroughs indicate the live load factors provided in the referenced AASHTO MBE (AASHTO 2011).

2.3.1.5 Summary

In summary, Part A of the current AASHTO MBE (AASHTO 2011) covers the newest method to load rate bridges, LRFR, which is preferred by the FHWA (Mertz 2015). Within LRFR, bridges are evaluated using possibly three separate procedures. See Figure 2.1 for a load rating procedure flowchart provided in the AASHTO MBE (AASHTO 2011).

- Design load rating is the first-level evaluation and is broken into inventory level rating (the multiple of HL-93 loads that can travel on the bridge while keeping the structure in sustainable condition for an indefinite period of time) and operating level rating (the multiple of HL-93 loads that is the absolute maximum weight that can travel on the structure). If a bridge has an RF greater than 1.0 for both inventory level and operating level rating, it has a passing design load rating and is safe for all AASHTO and state legal truck loads and configurations.
- Legal load rating is the second-level evaluation that takes place if a bridge does not pass the design load rating. In the legal load rating procedure, the bridge is evaluated for all AASHTO and state legal loads, again passing if the RF is greater than 1.0.
- Permit load rating is a third-level evaluation completed when a bridge passes the design load rating and a request is made for a truck heavier than the designated HL-93 load to travel on it. The specific truck weight and configuration is used, and as before, an RF greater than 1.0 indicates that the bridge passes and the vehicle can cross.

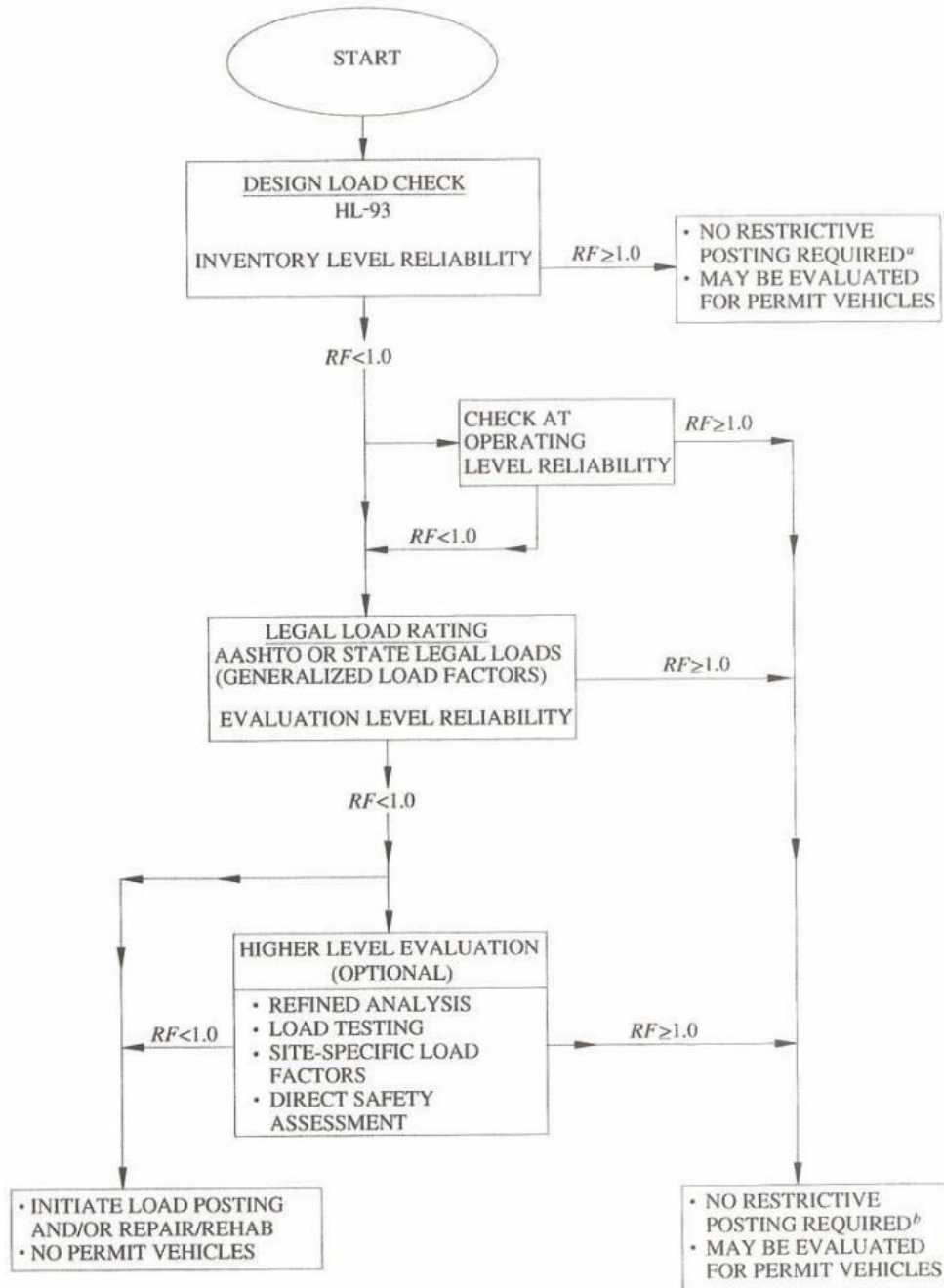


Figure 2.1. MBE LRFR Procedure Flow Chart (AASHTO 2011)

When posting a bridge, the gross weight in tons allowed for specific vehicles is posted on the actual sign. The MBE gives an equation to determine the safe posting load for a bridge if the RF is between 1.0 and 0.3. If the RF is less than 0.3, the bridge is required to be closed. The safe posting load for a structure is given in Equation (2.2).

$$\text{Safe Posting Load} = \frac{W}{0.7}(RF - 0.3) \quad (2.2)$$

where:

RF = Legal load rating factor

W = Weight of rating vehicle

2.3.2 Allowable Stress Rating (ASR) and Load Factor Rating (LFR)

Part B of the MBE covers the older methods to load rate bridges: ASR and LFR. Both methods use the same general equation to obtain an RF, shown in Equation (2.3).

$$RF = \frac{C - A_1D}{A_2L(1 + I)} \quad (2.3)$$

where:

RF = Rating Factor for the live load-carrying capacity

C = Capacity of the member

D = Dead load effect on the member

L = Live load effect on the member

I = Impact factor to be used with the live load effect

A_1 = Factor for dead loads

A_2 = Factor for live load

The ASR and LFR do not have the different load rating procedures used for LRFR. The bridge is only evaluated under the design truck load or one of the legal loadings. The A_1 and A_2 factors, which are similar to the dead and live load factors in LRFR, do not change. The design load, and therefore the first loading scenario to be evaluated for ASR and LFR, is the HS-20 truck configuration, as opposed to the HL-93 loading used by LRFR. If the bridge passes (has an RF greater than 1.0) for the HS-20 loading, it also passes for all of the AASHTO and state legal loads.

For the ASR method, the dead and live load factors, A_1 and A_2 , are both always equal to 1.0. The capacity of the member changes depending on whether the bridge is being evaluated at inventory or operating level reliability. The MBE has tables and subsections dedicated to finding the capacity for different materials and situations using the ASR method. As the name *allowable*

stress rating suggests, a limit on the maximum stress that a member is allowed to experience is used to determine the capacity. This limit is usually some portion of the yield or ultimate strength of the member and is lower for inventory level ratings than operating level ratings and therefore produces a smaller inventory RF.

For the LFR method, the dead load factor, A_1 , is 1.3, and the live load factor, A_2 , is 2.17 for inventory level ratings and 1.3 for operating level ratings. This produces higher RFs for operating level ratings. Also, unlike the ASR method, the member or component capacity is the same for both inventory level and operating level ratings. It is calculated using the procedure that is presented in the *AASHTO Standard Specifications* (AASHTO 2002) and uses relationships corresponding to the component and material being evaluated.

2.3.3 Comparison of Bridge Load Rating Methods

Although having three different rating methods allows engineers to, in some cases, choose which rating method they prefer, there are some issues. When a bridge was designed using LFD or ASD, the FHWA gives the engineer the option to rate it using LFR or LRFR. This option causes uncertainty and a lack of uniformity among states as to which load rating method produces more accurate ratings. Although LRFR is more recently developed and uses a more rigorous approach, many states are hesitant to switch to it is because of the level of comfort they have in using LFR and the amount of resources they have already invested into it (Bowman and Chou 2014). An Indiana DOT survey and an NCHRP survey, both conducted in 2014, found that more states prefer to use the LFR method over the LRFR method to rate their bridges (Bowman and Chou 2014; Hearn 2014). Another reason that state DOTs seem to be unsure of which method to use is because they do not know the effect switching to LRFR will have on the ratings of their bridge populations. Multiple reports have examined how using a different rating method can significantly affect the RF of the same bridge. NCHRP Report 700 (Mlynarski et al. 2011), *A Comparison of AASHTO Bridge Load Rating Methods*, examines this issue and compares the results of load rating 1500 bridges using both the LRFR and LFR methods. The bridge types analyzed include simple-span steel girder, simple-span prestressed I girder, simple-span prestressed box, simple-span reinforced concrete T-beam, simple-span reinforced concrete slab, continuous steel I-girder, continuous prestressed I-girder, and continuous reinforced concrete slab. When analyzed for flexure at the design load inventory level, it was found that almost every bridge exhibited an LFR

RF higher than its LRFR RF. This result was also the case when the bridges were analyzed using the AASHTO legal loads and numerous state-specific legal loads. For most bridges, the LRFR RF was 60–80 percent of the LFR RF. The same trend was seen when the bridges were analyzed for shear. In the majority of bridges, the LFR RF was higher than the LRFR RF.

The authors determined two main reasons that the LFR method produced higher RFs than the LRFR method. First, the factored live loads were typically higher for the LRFR method than they were for the LFR method. Second, in the AASHTO Virtis computer program that was used, the LRFR method checked for more possible controlling capacity scenarios than was done for the LFR method. This process included checking shear in concrete beams and slabs, shear friction resistance between concrete girders and cast-in-place concrete decks, bearing stiffener capacity in steel girder bridges, and the effect of shear on the force in longitudinal reinforcement in concrete beams near the girder ends. Many of these checks that were performed by the LRFR method, but not by the LFR method, controlled the RF of the bridge.

In Task 122 of NCHRP Project 20-07 (Mertz 2005), *Load Rating by Load and Resistance Factor Evaluation Method*, 74 example bridges were obtained from New York and Wyoming State Department of Transportation databases. These bridges were load rated using both the LRFR and LFR methods under design load rating procedures, and the findings were examined and compared. For most bridge types examined, the LRFR RF was greater than the LFR RF for inventory level ratings. However, for operating level ratings, the LRFR RF was less than the LFR RF for most bridge types examined. It was also found that LFR operating level ratings were usually approximately 67 percent higher than LFR inventory level ratings, but LRFR operating level ratings were only about 30 percent higher than LRFR inventory level ratings. A comparison of NCHRP Report 700 and NCHRP Project 20-07 Task 122 indicates that the results contradict each other. Report 700 found LFR RFs to be greater than LRFR RFs at the design load inventory level, sometimes quite significantly. Task 122 found the opposite, that LRFR RFs were slightly greater than LFR RFs at the design load inventory level. One possible explanation is that Task 122 used a smaller sample size of bridges (74 bridges from two states), and those bridges tended to perform better using LRFR than bridges nationwide. There may also have been a difference in the LRFR checks included in the two studies. These two studies show that load ratings can be quite different depending on which method is used and the bridge being analyzed.

Task 122 also explores the differences in reliability of LRFR ratings and LFR ratings at the inventory level. One million Monte Carlo simulations were run for 26 of the 74 bridges, and the failure rate for each of those 26 structures was determined. From this experiment, a reliability index was determined for each bridge using each rating method. This reliability index was plotted versus the inventory level RF of each bridge. This plot is shown in Figure 2.2 and reveals a much greater correlation between the reliability index and inventory level rating for bridges rated using LRFR than for those rated using LFR. This result shows why there has been a big push to switch to using LRFR methods to calculate load ratings in recent years and why it is the FHWA’s preferred method of load rating (Mertz 2015; Mertz 2005).

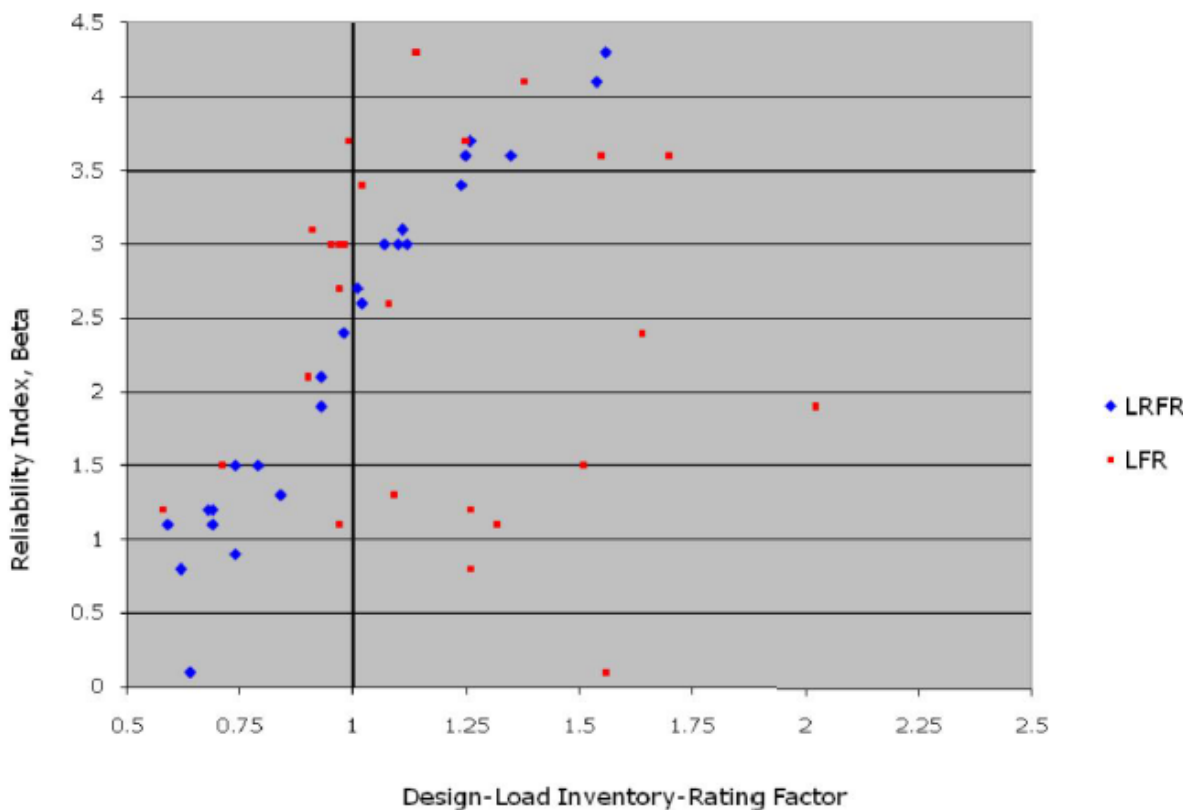


Figure 2.2. Reliability Index vs. Inventory RF for a Database of New York State and Wyoming Bridges (Mertz 2005)

2.3.4 Nondestructive Testing for Bridge Load Rating

The AASHTO MBE (AASHTO 2011) also has a section called Nondestructive Load Testing. Part of this section describes a procedure for load rating a bridge through load testing, using either diagnostic test results or proof test results. A diagnostic test uses loads that are high enough to

verify and adjust an analytical model of the bridge. These loads are usually lower than a proof test, in which specific loads are applied to the bridge to determine if the bridge has the capacity to carry these loads without being damaged. Load rating using diagnostic test results is a fairly straightforward procedure. The RF based on test results is the RF found using LRFR methods multiplied by an adjustment factor, K . This adjustment factor is affected by other factors that take into account the calculated member strains based on the test vehicle, the member strains observed during the load test, and the ability of the test results to be extrapolated to performance at higher loads. Proof test results can only be used to find the operating level ratings. The RF at operating level, RF_O , obtained from proof test results is given as:

$$RF_O = \frac{OP}{L_R(1 + IM)} \quad (2.4)$$

where:

- OP = Operating level capacity
- L_R = Comparable unfactored live load due to the rating vehicle
- IM = Dynamic load allowance

The operating level capacity, OP , is found as:

$$OP = \frac{k_O L_p}{X_{pA}} \quad (2.5)$$

where:

- k_O = 1.0 if the proof load test reached the target load, and 0.88 if the proof load test was terminated because of distress in the bridge
- L_p = Actual maximum proof live load applied to the bridge
- X_{pA} = Target live load factor found using the procedure in Section 8.8.3.3.2 of the MBE (AASHTO 2011)

Although AASHTO gives these procedures to calculate an RF from a load test and load tests are a more refined way to obtain a load rating, load tests are expensive and can be inefficient to run. Load tests can require a significant amount of time and resources to conduct and only give the RF for one particular structure. For this reason, many states rely on the LRFR, ASR, and LFR methods to load rate bridges and only load test in special circumstances (Hearn 2014).

2.3.5 **Material Properties**

A large portion of the AASHTO MBE also focuses on material strengths for bridges when the quality of materials used is not known. This situation could arise because the original plans for the bridge either cannot be found or did not specify required material strengths. Because material strengths often play an important role in determining the capacity of the bridge, the MBE gives common values corresponding to time periods of construction.

2.3.5.1 LRFR Material Properties

Table 2.5, Table 2.6, Table 2.7, and Table 2.8 contain material strength properties to be used in LRFR when the in-situ strengths are not known. These tables represent some of the material strength information given by the AASHTO MBE (AASHTO 2011). More material strengths for other components are given in Section A of the AASHTO MBE.

Table 2.5. Minimum Compressive Strength of Concrete by Year of Construction (AASHTO 2011)

Year of Construction	Compressive Strength, f'_c, ksi
Prior to 1959	2.5
1959 and later	3.0

Table 2.6. Yield Strength of Reinforcing Steel (AASHTO 2011)

Type of Reinforcing Steel	Yield Strength, f_y, ksi
Unknown steel constructed prior to 1954	33
Structural grade	36
Billet or intermediate grade, Grade 40, and unknown steel constructed during or after 1954	40
Rail or hard grade, Grade 50	50
Grade 60	60

Table 2.7. Tensile Strength of Prestressing Steel (AASHTO 2011)

Year of Construction	Tensile Strength, f_{pu}, ksi
Prior to 1963	232
1963 and Later	250

Table 2.8. Minimum Mechanical Properties of Structural Steel by Year of Construction (AASHTO 2011)

Year of Construction	Minimum Yield Point or Minimum Yield Strength, F_y, ksi	Minimum Tensile Strength, F_u, ksi
Prior to 1905	26	52
1905 to 1936	30	60
1936 to 1963	33	66
After 1963	36	66

2.3.5.2 LFR and ASR Material Properties

Table 2.9 and Table 2.10 contain material strength properties to be used in LFR and ASR when the in-situ strengths are not known. These tables represent some of the material strength information given by the AASHTO MBE. More material strengths for other components are given in Section B of the AASHTO MBE.

Table 2.9. Allowable Unit Stresses for Concrete (AASHTO 2011)

Year Built	Compressive Strength, f'_c, psi
Prior to 1959	2500
1959 and later	3000

Table 2.10. Allowable Unit Stresses (psi) for Reinforcing Steel (AASHTO 2011)

Type of Reinforcing Steel	Inventory Rating	Operating Rating	Yield
Unknown steel constructed prior to 1954	18,000	25,000	33,000
Structural grade	20,000	27,000	36,000
Billet or intermediate grade, Grade 40, and unknown steel constructed during or after 1954	20,000	28,000	40,000
Rail or hard grade, Grade 50	20,000	32,500	50,000
Grade 60	24,000	36,000	60,000

For cases in which the yield strength or allowable stress of other steel members is not specified, the LFR/ASR section of the MBE directs the user to the “date built” column of MBE Tables 6B.6.2.1-1 and 6B.6.2.1-4 for yield strengths, and to Table 10.56A in the *AASHTO Standard Specifications* (AASHTO 2002) for the maximum strengths of welds, bolts, and rivets. Section 6B.6.3.1 of the MBE also allows for coupon testing to be used to determine the yield strength of a bridge if it is unknown. The yield strength used to determine capacity is taken as the mean test value minus 1.65 standard deviations.

These material strength properties will be examined and discussed in more depth later in this report since some of them may be able to be refined through laboratory testing of samples obtained from bridges in current use whose load ratings are being evaluated and from available mill certificate information from bridges of interest.

2.3.6 Other Relevant Load Rating Practices

2.3.6.1 National Bridge Inventory and Weight Limits

In addition to the MBE, the FHWA provides further policy for load rating bridges. The FHWA requires states to load rate all of their public bridges longer than 20 ft and report the results (United States Government 2004). This information, along with information about dozens of other characteristics about each bridge, is stored in the NBI, which makes it relatively easy to obtain selected bridge information without having to conduct a site visit or rating calculations.

Title 23 of the US Code (Government 1958) provides weight limits for vehicles traveling on the Interstate Highway System, and the FHWA summarizes these limits in a guidance pamphlet entitled *Bridge Formula Weights*, which was most recently revised in May 2015. This guide contains a formula to determine the allowable overall gross weight on any group of two or more consecutive axles that is allowed on an Interstate Highway System bridge. This formula is based on the distance between the outer axles in the group and the number of axles in the group. From this formula, a convenient table was produced to show the maximum allowable weights for different numbers of axles and spacing. Even though the table may provide higher allowable weights, the FHWA sets the maximum gross weight allowed for a vehicle to an upper limit of 80,000 lbs, unless the formula gives a lower number (FHWA 2015). Title 23 of the US Code also sets a limit of 20,000 lbs per axle and 34,000 lbs per tandem axle (Government 1958).

2.3.6.2 *LRFD Live Load Distribution Factors*

Live load distribution factors (LLDFs) are another item of interest when discussing load rating of bridges. Moment LLDFs are focused on in this report because the moment capacity is usually the controlling feature in the capacity of the bridge, as opposed to the shear. Determination of shear LLDFs follows a procedure similar to the determination of moment LLDFs, equations for which can be found in the *AASHTO LRFD Bridge Design Specifications (AASHTO LRFD Specifications)* (AASHTO 2017). Moment LLDFs are applied to the live load effect that a member experiences and therefore influence the RF calculated for that member. Currently, for moment on interior beams, LRFD moment LLDFs are calculated through a series of procedures and equations outlined in Section 4 of the *AASHTO LRFD Specifications* (AASHTO 2017). In almost all cases, depending on the type and cross-section of the superstructure, different equations involving the girder spacing, length of span, and transverse stiffness are given to determine moment LLDFs for interior or exterior girders.

For moment on exterior beams, in most cases, the lever rule is used to determine the LRFD moment distribution factor. The lever rule begins by assuming that the deck of the bridge is hinged at the interior girder closest to the exterior girder for which the moment distribution factor is being analyzed. The truck is then placed with the closest wheel 2 ft from the barrier or curb. Finally, the reaction on the exterior girder from this loading is to determine the exterior girder moment LLDF (Barth 2015).

These equations and procedures to determine the LRFD LLDFs were first developed and recommended to AASHTO through NCHRP Project 12-26, completed in 1990 (Zokaie et al. 1991). Data from bridge inventories were compiled, and average bridges that represented a variety of different bridge types were developed. The average bridges had wheel loads applied to them, and their geometric parameters were varied to see how the moment and shear LLDFs changes with those variations. Formulas for the LLDFs were then developed by analyzing each average bridge at three different levels. Bridges were analyzed using simplified analytical methods, graphical analysis methods, and the finite element method (FEM). From these analyses, simplified formulas for the LLDFs were developed. These simplified formulas were compared to the analytical methods and FEM analysis, and the formulas that gave the most accurate results were adopted as the pertinent formulas. With minor adjustments, these formulas are the ones that are included in the *AASHTO LRFD Bridge Design Specifications* (Mertz 2007).

When using the lever rule, the moment LLDF for each girder is multiplied by a multiple presence factor and the overall live load effect to determine the design live load effect for an individual girder. When using one of the simplified equations, the multiple presence factor is included, and therefore the resulting moment LLDF only needs to be multiplied by the overall live load effect. When calculating the RF for an individual member, the LLDF will affect the amount of live load moment or shear that the member must resist. The AASHTO MBE (AASHTO 2011) does not specifically mention under which scenarios to use LLDFs and if those factors are modified for load rating. However, in Section 6A.3 it states that “methods of structural analysis suitable for the evaluation of bridges shall be as described in Section 4 of the *AASHTO LRFD Bridge Design Specifications*.” From this statement, it can be inferred that the live load distribution factors that are described in the *AASHTO LRFD Bridge Design Specifications* therefore also apply to the LRFR method of load rating. Figure 2.3 shows the moment LLDFs for interior beams given in the 2017 edition of the *AASHTO LRFD Bridge Design Specifications*.

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Wood Deck on Wood or Steel Beams	a, l	See Table 4.6.2.2a-1	
Concrete Deck on Wood Beams	l	One Design Lane Loaded: $S/12.0$ Two or More Design Lanes Loaded: $S/10.0$	$S \leq 6.0$
Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$3.5 \leq S \leq 16.0$ $4.5 \leq t_s \leq 12.0$ $20 \leq L \leq 240$ $N_b \geq 4$ $10,000 \leq K_g \leq 7,000,000$ $N_b = 3$
Cast-in-Place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75 + \frac{S}{3.6}\right) \left(\frac{1}{L}\right)^{0.35} \left(\frac{1}{N_c}\right)^{0.45}$ Two or More Design Lanes Loaded: $\left(\frac{13}{N_c}\right)^{0.3} \left(\frac{S}{5.8}\right) \left(\frac{1}{L}\right)^{0.25}$	$7.0 \leq S \leq 13.0$ $60 \leq L \leq 240$ $N_c \geq 3$ If $N_c > 8$ use $N_c = 8$
Concrete Deck on Concrete Spread Box Beams	b, c	One Design Lane Loaded: $\left(\frac{S}{3.0}\right)^{0.35} \left(\frac{Sd}{12.0L^2}\right)^{0.25}$ Two or More Design Lanes Loaded: $\left(\frac{S}{6.3}\right)^{0.6} \left(\frac{Sd}{12.0L^2}\right)^{0.125}$ Use Lever Rule	$6.0 \leq S \leq 18.0$ $20 \leq L \leq 140$ $18 \leq d \leq 65$ $N_b \geq 3$ $S > 18.0$
Concrete Beams used in Multibeam Decks	f, g	One Design Lane Loaded: $k \left(\frac{b}{33.3L}\right)^{0.5} \left(\frac{I}{J}\right)^{0.25}$ where: $k = 2.5(N_b)^{-0.2} \geq 1.5$ Two or More Design Lanes Loaded: $k \left(\frac{b}{305}\right)^{0.6} \left(\frac{b}{12.0L}\right)^{0.2} \left(\frac{I}{J}\right)^{0.06}$	$35 \leq b \leq 60$ $20 \leq L \leq 120$ $5 \leq N_b \leq 20$
Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
	h, also i, j if connected only enough to prevent relative vertical displacement at the interface	Regardless of Number of Loaded Lanes: S/D where: $C = K(W/L) \leq K$ $D = 11.5 - N_L + 1.4N_L(1 - 0.2C)^2$ when $C \leq 5$ $D = 11.5 - N_L$ when $C > 5$ $K = \sqrt{\frac{(1+\mu)I}{J}}$ for preliminary design, the following values of K may be used: Beam Type K Nonvoided rectangular beams 0.7 Rectangular beams with circular voids: 0.8 Box section beams 1.0 Channel beams 2.2 T-beam 2.0 Double T-beam 2.0	$\text{Skew} \leq 45^\circ$ $N_L \leq 6$
Open Steel Grid Deck on Steel Beams	a	One Design Lane Loaded: $S/7.5$ If $t_g < 4.0$ $S/10.0$ If $t_g \geq 4.0$ Two or More Design Lanes Loaded: $S/8.0$ If $t_g < 4.0$ $S/10.0$ If $t_g \geq 4.0$	$S \leq 6.0$ $S \leq 10.5$
Concrete Deck on Multiple Steel Box Girders	b, c	Regardless of Number of Loaded Lanes: $0.05 + 0.85 \frac{N_L}{N_b} + \frac{0.425}{N_L}$	$0.5 \leq \frac{N_L}{N_b} \leq 1.5$

Figure 2.3. Moment Live Load Distribution Factors per Lane for Interior Beams (AASHTO 2017)

2.3.6.3 *LFR and ASR Live Load Distribution Factors*

When determining capacity using the LFD method, the LLDFs are calculated using simplified equations that were first introduced by Newmark in 1938 (Newmark 1938) and slightly updated as research became available to increase their accuracy. The factors are given in the form of S/D , where S is the spacing of the girders and D is a factor that changes depending on the type of bridge being analyzed. As can be seen, this formula does not take into account deck stiffness, span length, or skew, which are considered for the LRFD LLDFs. This omission can make the LFD distribution factors less accurate and too conservative for bridges with characteristics outside of certain ranges (Hueste et al. 2015). Adjusting the LFD distribution factors to make them more accurate is something that could be explored in an effort to improve load ratings. The LFD distribution factors, known as S/D distribution factors, are given in Figure 2.4.

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or more Traffic Lanes
Concrete: On steel I-Beam stringers ¹ and prestressed concrete girders	S/7.0 If S exceeds 10' use footnote f.	S/5.5 If S exceeds 14' use footnote f.
On concrete T-Beams	S/6.5 If S exceeds 6' use footnote f.	S/6.0 If S exceeds 10' use footnote f.
On timber stringers	S/6.0 If S exceeds 6' use footnote f.	S/5.0 If S exceeds 10' use footnote f.
Concrete box girders ²	S/8.0 If S exceeds 12' use footnote f.	S/7.0 If S exceeds 16' use footnote f.
On steel box girders On prestressed concrete spread box Beams	See Article 10.39.2. See Article 3.28.	

¹In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam.

3.28 DISTRIBUTION OF LOADS FOR BENDING MOMENT IN SPREAD BOX GIRDERS*

3.28.1 Interior Beams

The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel load (both front and rear) determined by the following equation:

$$D.F. = \frac{2N_L}{N_B} + k \frac{S}{L} \quad (3-33)$$

where,

- N_L = number of design traffic lanes (Article 3.6);
- N_B = number of beams (4 ≤ N_B ≤ 10);
- S = beam spacing in feet (6.57 ≤ S ≤ 11.00);
- L = span length in feet;
- k = 0.07 W - N_L (0.10N_L - 0.26) - 0.20N_B - 0.12; (3-34)
- W = numeric value of the roadway width between curbs expressed in feet (32 ≤ W ≤ 66).

3.28.2 Exterior Beams

The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than 2N_L/N_B.

Figure 2.4. LFD Distribution Factors (AASHTO 1996)

2.4 ADDITIONAL LOAD RATING PRACTICES

2.4.1 TxDOT Load Rating Procedures

2.4.1.1 General

TxDOT currently does not have a manual dedicated solely to bridge load rating. However, there is a Load Ratings section in the *TxDOT Bridge Inspection Manual*, the most recent version of which was published in 2013 (TxDOT 2013). In general, TxDOT follows the procedures set out in the AASHTO MBE (AASHTO 2011). Any bridge designed using the LRFD method is load rated using LRFR. Otherwise, the *TxDOT Bridge Inspection Manual* states the LFR method is to be used for all on-system bridges except for timber bridges, which are load rated using ASR.

TxDOT allows the inventory level rating to be “initially estimated to be at least equal to the design loading if no damage or deterioration exists and the original design was made using an HS or HL-93 load pattern” (TxDOT 2013). However, it is not specified for how long that initial estimate is allowed to be used as the RF for the bridge, if at all. TxDOT also does not take into consideration temporary repairs when determining inventory or operating level ratings.

One main area in which TxDOT does differ from the MBE is in determining ratings for concrete bridges with unknown reinforcing details. The MBE allows these types of bridges to not be posted “if it has been carrying normal traffic for an appreciable period and shows no distress” (AASHTO 2011). However, under these same conditions, TxDOT requires the bridge to be posted at the operating level for an HS20 loading scenario (TxDOT 2013).

The *TxDOT Bridge Inspection Manual* also explains a load rating scenario for one of its unique bridge types, cast-in-place concrete slab bridges with integral curbs—also referred as FS (Farm Service Road) Bridges in the TxDOT bridge drawings. The manual goes on to say that the AASHTO bridge design specifications do not differentiate between single-lane and multiple-lane distribution factors for FS bridges. Therefore, when load rating these bridges, sometimes engineers adjust the LLDFs based on the number of lanes loaded.

Other than these details and a brief introduction about which methods to use, the *TxDOT Bridge Inspection Manual* does not specify many situations in which TxDOT differs from the MBE in loading rating procedures. Therefore, TxDOT essentially follows what is laid out in the MBE along with these additional guidelines.

2.4.1.2 THD Supplement

Texas bridges built between 1949 and 1961 were designed using TxDOT’s *THD Supplement No. 1* (TxDOT 1946) in addition to the *AASHTO Standard Specifications for Highway Bridges* (denoted as *AASHTO Standard Specifications*) (TxDOT 2013). When load rating Texas bridges built in this time period, the changes this supplement made to bridge design should be considered. The *THD Supplement No. 1* made five significant changes to Texas bridge design between 1949 and 1961, as follows:

- The supplement allowed the omission of curbs on the bridge as long as the fence or barrier was continued throughout the structure. This feature could potentially change the number or width of loaded lanes on the bridge.
- The *AASHTO Standard Specifications* (AASHTO 1944) required an overload increase of 100 percent in one lane if the bridge was designed for less than an H-20 load. The allowable stress was then increased to 150 percent of the normal allowable stress. The *THD Supplement* required this same overload to be applied to truss counter members when determining the necessary size of the counter member.

- In an effort to produce maximum positive and negative moments, the *AASHO Standard Specifications* (AASHO 1944) also required an additional concentrated load in the next span of a continuous structure when H-10, H-15, or H-20 design loads were being used. The *THD Supplement* limited the distance from the concentrated loads in the lane load to this additional concentrated load to 30 ft, thereby reducing the maximum negative moment for some continuous spans.
- The *THD Supplement* also required the actual loaded length indicated by the influence line of the member being considered as the length used in determination of the negative moment impact factor value. This usually increased the impact value used in design.
- Finally, the *THD Supplement* put limitations on the axle loads that were to be considered when designing bridge floors (TxDOT 2013).

While these changes may not directly affect the load rating of a bridge, they should be kept in mind when conducting load ratings on a Texas bridge built between 1949 and 1961.

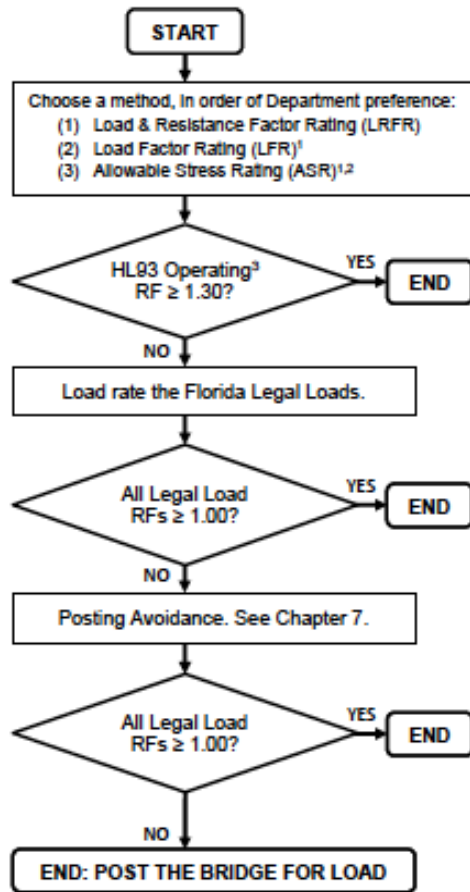
2.4.2 Other Load Rating Practices in the United States

In 2014, NCHRP released Synthesis 453, *State Bridge Load Posting Processes and Practices* (Hearn 2014). A survey was sent to all fifty states asking a variety of questions about their load rating practices. Out of the 50 states that the survey was sent to, 43 responded and the results were compiled and reported. Out of the 43 responses, 29 states reported that they use LRFR, 39 use LFR, 27 use ASR, 34 use more than one load rating method, and 18 reported using all three load rating methods in practice. It can be seen that although the LRFR method is encouraged by the FHWA, many states still heavily rely on the older methods of load rating. Many of the states that were surveyed also use other methods to load rate besides the basic analytical methods.

2.4.2.1 Refined Analysis and Improvement of Ratings

Twenty-four states reported using refined analysis methods, such as FEM analysis, in their load rating practices. Of those, the NCHRP Synthesis 453 report states that 18 reported using refined methods to avoid posting, 14 used them for analyzing complex bridges, and six used refined methods for both of those reasons (Hearn 2014). Other states such as Iowa, Oregon, and Pennsylvania use refined methods or FEM to determine new live load distribution factors to use in analytical methods (Iowa DOT 2013; Oregon DOT 2015; Pennsylvania DOT 2010). West

Virginia uses refined analysis methods to compute conversion factors between refined and analytical ratings when new bridges are designed. These conversion factors may then be used to refine future analytical rating calculations (Hearn 2014). The Oregon DOT states in its *LRFR Manual* that updated material properties may be used if samples from the bridge are obtained and tested. It also allows the LLDFs to be obtained by placing the vehicles in the actual striped lanes of the bridge if it does not initially pass using analytical methods (Oregon DOT 2015). Both the Oregon and Florida DOTs allow the initial RF calculated using AASHTO’s analytical methods to be rounded up by 5 percent (Florida DOT 2017; Oregon DOT 2015). Florida also allows the AASHTO provided dead load factor of 1.25 to be reduced to 1.15 “when dimensions are thoroughly and accurately field verified” (Florida DOT 2017). These practices are part of the Posting Avoidance step of FDOT’s load rating procedures, which is given in its *Load Rating Manual* as a flowchart, shown in Figure 2.5.



1. LFR and ASR are not permitted among spans exceeding 200 feet.
2. ASR is not permitted for bridges on the National Highway System.
3. At existing bridges, if $RF_{HL93 \text{ Operating}} < 1.30$, or if LFR/ASR, assess the Legal Loads.

Figure 2.5. Florida DOT Load Rating Flowchart (Florida DOT 2017)

2.4.2.2 Field Evaluation and Engineering Judgement

Out of the 43 states that responded to the NCHRP Synthesis 453 survey, 27 reported allowing field evaluation or engineering judgement to be used in load rating bridges. Indiana, Iowa, Pennsylvania, and Wisconsin are among those that allow engineering judgement to be used to rate a reinforced concrete bridge if the reinforcing details are unknown (INDOT 2016; IOWADOT 2013; PennDOT 2010; WisDOT 2017). The *FDOT Bridge Load Rating Manual* states that if a bridge with unknown reinforcing details shows no signs of distress, it can be assigned a load rating. However, if it does show signs of distress, the bridge must be load tested (Florida DOT 2017). Although numerous state DOTs allow load rating through engineering judgement, some specifically do not. The Massachusetts DOT states in its *LRFD Bridge Manual* (MassDOT 2013)

that using engineering judgement alone is not allowed for load rating. It requires that field measurements, nondestructive testing, and material testing be performed for bridges with unknown structural details.

2.4.2.3 Load Testing

A total of 19 out of 43 states in the original NCHRP Synthesis 453 reported using load tests to determine load ratings. The *Iowa DOT Bridge Rating Manual* allows load testing for rating if “deterioration is difficult to quantify, conventional analysis methods are difficult to apply to a unique structural configuration, or there is a public need to allow larger vehicles to cross a bridge than the conventional analysis will allow” (Iowa DOT 2013). A variety of other state DOT load rating manuals specifically allow for load testing to be used on a case-by-case basis, with most following the load testing processes set out in the AASHTO MBE (MDOT 2009; MnDOT 2009; NYSDOT 2005; ODOT 2015; PennDOT 2010; RIDOT 2017; UDOT 2014; WisDOT 2017).

2.4.3 International Load Rating Practices

2.4.3.1 United Kingdom

In 2010, the newly introduced Eurocode Standards created a uniform code for the design of bridges within the European Union. However, the Eurocodes specifically do not address the assessment of existing structures (Hendy et al. 2011). This oversight leaves developing load rating procedures up to the specific countries. For this reason, the United Kingdom (UK) has developed its own load rating practices. UK load rating procedures differ somewhat from the AASHTO procedures; however, the end goal of ensuring that capacity exceeds demand is the same. These procedures are laid out in their *Design Manual for Roads and Bridges, Volume 3, Section 4, Part 3, BD 21/01* (UK Highways Agency 2001).

For bridges between the lengths of 2–50 meters, the structure is evaluated for three different loading scenarios. The first scenario is the type HA design loading, in which each notional lane is loaded with both a uniformly distributed load (UDL) and a knife edge load (KEL). The UDL, W , in kN per meter length and a lane width of 3.65 meters, is found through the following equation:

$$W = 336 \left(\frac{1}{L} \right)^{0.67} \quad (2.6)$$

where L is the length of the bridge in meters. The KEL is simply 120 kN uniformly distributed across the lane width. The HA design loading is also divided by an adjustment factor that depends on the length of the bridge and decreases the HA loading. This step is done to remove the effect of a lateral bunching factor that was used to derive the HA loading.

The second scenario is a single-axle load in each lane of the bridge and the third scenario is a single-wheel load in each lane of the bridge. Each of these scenarios can be evaluated at different assessment live loadings of 40 metric tons, 26 metric tons, 18 metric tons, 7.5 metric tons, and 3 metric tons, along with two groups of fire engines. Each of these assessment live loadings represent a group of vehicles with a gross weight corresponding to its name. Depending on the assessment live loading being used and the volume of traffic and quality of roadway, the weights of the single-axle and single-wheel loads are changed. Likewise, depending on these same situations, a reduction factor, K , is applied to the HA design loading. This reduction factor is applied to reduce some of the overt conservativeness that was used when the HA loading was derived. If the bridge does not pass the evaluation for the 40 metric ton assessment live loading level, it is assigned a live load capacity factor, C , which is found by dividing the available live load capacity by the live load capacity required for the adjusted HA loading. The bridge is then posted for the highest assessment live loading level that has a K value less than C . Essentially, similar to the AASHTO LFR method, the UK applies a factored load to the bridge and ensures that the capacity of the bridge is greater than the factored loads.

The British Standard Codes also allow for the use of FEM analysis in the design of bridges, leading to the use of models in analysis and capacity determination since they have already been developed for many bridges in the design phase (Hida et al. 2010). In addition, the UK has a section in its *Design Manual for Roads and Bridges* entitled “Load Testing for Bridge Assessment” (UK Highways Agency 1994). This section provides guidelines for using load tests to evaluate existing bridges. However, it does not provide any specific equations to obtain a capacity or RF through load tests like the AASHTO MBE (AASHTO 2011). Instead, it gives general guidelines, leaving much of the decisions up to engineering judgement. Load tests are divided into two categories: supplementary load tests and proving load tests.

Supplementary tests are used in conjunction with analytical calculations to change factors used in those calculations so that they are more representative of the actual bridge. Possible features that could be changed are end fixities, transverse distribution of load, and composite action. Actual strains and deflections are observed during the load test, and then analytical calculations are performed again using the identified characteristics of the bridge in an effort to obtain increased capacity.

Proving tests, on the other hand, are not combined with analytical calculations. The bridge is loaded until the full desired assessment live load is reached or until its stress-strain behavior becomes nonlinear. It is then known and proven that the bridge can carry the load that was applied during the test. The document “BA 54/94” (UK Highways Agency 1994) does mention that most of the time a static load test alone will not represent the real situation because static load tests do not use moving loads and do not include the conservativeness that the analytical calculations include. For this reason, when conducting supplementary load tests, which are most common, no safe basis exists for extrapolating the results to determine the ultimate capacity of the bridge, which means that caution must be used when using load tests to increase bridge ratings.

2.4.3.2 Ontario, Canada

The Ontario Highway Bridge Design Code (OHBDC), for which development first began in 1975, was one of the first North American codes to use target reliabilities in bridge load rating (Csagoly and Dorton 1978). The design truck and live load factors were developed based on actual truck loading data and the target reliability indices obtained in Ontario in 1973. It was the basis for the development of the current Canadian bridge load rating code. Unlike in the United States, Canadian bridges are not required to be inspected and rated at periodic intervals. Bridges are rated using a design live load truck or the presence of multiple vehicles in a lane, which are similar to the AASHTO legal loads. These loads are checked at ultimate, service, and fatigue limit states. As in the AASHTO MBE, the strength limit state is the only limit state required to be checked for the load rating of bridges.

There are some significant differences in the OHBDC versus the AASHTO MBE when evaluating a structure. For example, if a bridge is inspected at least once every five years, the live load acting on that structure may be reduced by up to 10 percent. The live load factor is allowed to be reduced for redundant, multiple lane bridges that exhibit certain characteristics. The

distributed lane load can be reduced based on the measured traffic volume on the bridge and the type of road class the structure is on. Also, the dead load factor is allowed to be reduced if the dimensions and unit weights affecting the dead load are precisely measured or known. All of these practices work toward avoiding posting or refining posting limits. The OHBDC does have some similarities to the MBE. It allows a concrete bridge to not be posted if it has been carrying normal traffic without any signs of distress and allows load tests to be used to assist in the evaluation of bridges. However, unlike the MBE, it does not provide information about types of load tests and load test procedures that can be performed or about how to calculate a rating factor from the results of a load test. In essence, the OHBDC is a reliability-based load rating code with procedures similar to what is presented in the AASHTO MBE. This similarity should not come across as a surprise since the OHBDC strongly influenced the development of the MBE. Although the two are quite similar, the OHBDC does provide some possibilities to improve load ratings based on the actual bridge characteristics and loading scenarios, something the MBE does not do (Csagoly and Dorton 1978; Ellingwood et al. 2009a).

2.5 RESEARCH STUDIES

2.5.1 Research by State DOTs to Improve Load Rating Practices

2.5.1.1 Georgia

In 2006, the Georgia Institute of Technology (Georgia Tech) was tasked by the Georgia Department of Transportation (GDOT) to develop *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia* by analyzing and load testing four bridges (Ellingwood et al. 2009a, b, c). The bridges selected for testing were a straight reinforced concrete T-beam bridge, a skewed reinforced concrete T-beam bridge, a prestressed concrete I-beam bridge, and a non-composite slab on steel I-girder bridge. All of the bridges were simply supported. These bridges were selected because regular and skewed T-beam bridges and steel girder bridges represent a significant population of the bridges in Georgia that are load rated. The prestressed concrete bridge was included because over half of the prestressed concrete bridges in the state that were posted were less than 30 years old and GDOT wanted Georgia Tech to determine why this was the case. A static, diagnostic load test was conducted on each of the four bridges

using one, two, three, and four Type 3 trucks, with individual truck weights ranging from 50 kips to almost 70 kips. Dial gauges and potentiometers were used to measure deflections under loading. These field-measured deflections were compared to deflections obtained using three-dimensional finite element models of each bridge. The finite element models were constructed using the computer program ABAQUS.

During the load tests, the two concrete T-beam bridges and the steel girder bridge were all loaded significantly above their design or posted limits, and all four bridges remained within their elastic limit. Therefore, the bridges all had significantly greater reserve capacity than the design or posted loads. The researchers also found that in most cases the FEM analysis predicted similar and slightly higher deflections than the field tests produced. In all cases, the measured and computed deflections were similar, and the maximum live load deflections all remained less than 50 percent of the $L/800$ limit that the *AASHTO LRFD Specifications* (AASHTO 2017) provides. As a result of this similarity, the FEM models were used to explore certain characteristics about the bridges that affected their load-carrying capacity. The moment LLDFs for interior girders were calculated using the formulas provided in the *AASHTO LRFD Specifications* (AASHTO 2017) and were determined through the FEM analysis developed for each bridge. For each bridge, the moment LLDFs for interior girders were lower when found using the FEM. This finding decreased the live load moment effect for those girders, thus increasing the RF for them, which increases the live load moment effect on the exterior girders. However, it does not increase their exterior girder LLDF above the AASHTO LRFD LLDF. Therefore, when the FEM LLDFs are used, the controlling moment that affects the RF of the bridge is reduced.

The researchers also conducted what they called a “static pushdown test” using the FEM model of each bridge. In this test, two HS20 trucks were placed side-by-side in a location that maximized the moment effect on the girders. The loads of the trucks, without application of load factors, were systematically increased until an element of the structural system yielded in the model. This occurrence allowed the researchers to effectively determine an operating level rating from the model because the lowest moment effect that caused yielding was used as the live load effect in the RF equation. Through this method, it was found that each bridge had an operating capacity substantially higher than the HS-20 truck load. The operating RFs for an HS-20 truck were greater than 4.0 for all four bridges and greater than 5.0 for two of them.

At the conclusion of the research, the research team from Georgia Tech produced *Recommended Guidelines for Condition Assessment, Evaluation, and Rating of Bridges in Georgia*. The main change they suggest for the load rating section of the AASHTO MBE is increasing some material properties based on laboratory tests that were also conducted. The straight T-beam bridge that was field-tested was scheduled to be decommissioned after the field test. Concrete cores were taken from the slab and girders of this bridge and tested under compression. The tests found that the compressive strength of the cores was significantly higher than what was specified when the bridge was constructed. Therefore, some of the concrete compressive strengths that depend on the period of construction were increased and updated. The researchers also provided load rating examples for a straight T-beam bridge, a steel girder bridge, and a prestressed concrete bridge in their recommendations. In each of these examples, the live load distribution factors found using FEM analysis were used to increase the RF for the bridge.

2.5.1.2 Oregon

In a study sponsored by the Oregon Department of Transportation (ODOT) in 2005, researchers used weigh-in-motion (WIM) tests to determine the actual truck weights and configurations that were driving on the roads in Oregon (Pelphrey et al. 2008). They then used these data to develop new live load factors for legal loads and permit loads and replaced the factors given in Table 6A.4.2.2-1 in the AASHTO MBE.

To obtain WIM data, a pair of loop detectors and sensors were installed directly into one lane on the roadway. They were spaced a certain distance apart longitudinal to the roadway. The sensors were also offset in the transverse direction so that one sensor covered one half of the lane, and the other sensor further down covered the other half of the lane. When the sensors were run over by a vehicle, they recorded the force applied as an electrical charge. The magnitude of this electrical charge corresponds to the magnitude of the force that was applied. As a result of being offset both transversely and longitudinally within the lane, the sensors could also measure velocity and spacing of axles. Therefore, the axle configurations and weights of any vehicle that passed over the sensors could be determined (Quinley 2010; Shoukry et al. 2008).

The researchers in the Oregon study used four WIM sites on four major roadways in the state to obtain truck weight data that were specific to the state of Oregon. The maximum loading event used to calibrate new factors assumes a legal or permit truck in one lane of the bridge and a

random truck, called an alongside truck, in the lane next to it. Using the WIM data, the researchers determined the mean and standard deviation of the top 20 percent of rating for vehicle truck weights and alongside vehicle truck weights. Using these data and following the same procedure used in NCHRP Project 12-46 and outlined in NCHRP Report 454 (Moses 2001), new live load factors were calculated as:

$$\gamma_L = 1.8 \frac{W_T}{240} \times \frac{72}{W} \quad (2.7)$$

for legal loads,

$$W_T = R_T + A_T \quad (2.8)$$

$$R_T = W^* + t_{ADTT} \sigma_{type}^* \quad (2.9)$$

$$A_T = W_{along}^* + t_{ADTT} \sigma_{along}^* \quad (2.10)$$

for permit loads, Equation (2.11) replaces Equation (2.9).

$$R_T = P + t_{ADTT} \sigma_{along}^* \quad (2.11)$$

where:

- γ_L = Live load factor for the legal or permit vehicle
- W = Gross weight of the legal or permit vehicle for which the live load factor is being calculated
- W_T = Expected maximum total weight of rating and alongside vehicles
- R_T = Rating truck
- A_T = Alongside truck
- W^* = Mean of the top 20 percent of legal trucks from the group being examined
- σ_{type}^* = Standard deviation of the top 20 percent of trucks from the group being examined
- t_{ADTT} = Fractal value corresponding to the number of side-by-side events (see NCHRP Report 454)

- W_{along}^* = Mean of the top 20 percent of alongside trucks
 σ_{along}^* = Standard deviation of the top 20 percent of alongside trucks
 P = Weight of permit truck

The researchers found significantly more overloads in the original data used to determine the AASHTO LRFD live load factors than in the Oregon state-specific WIM data that were collected, which led them to suggest to ODOT the reduction of some of the live load factors that depend on ADTT (Pelphrey et al. 2008). Reducing the live load factor reduces the live load effect on the bridge. Because the live load effect is in the denominator of the LRFR RF equation (Equation 2.1), reducing it will cause an increase in the RF of the bridge. These new live load factors that were developed are currently being used by ODOT in its LRFR procedures (Oregon DOT 2015). ODOT modifies the load factors from the routine commercial traffic (Type 3, Type 3S2, and Type 3-3) side of Table 2.1 to those in Table 2.11 or Table 2.12, with interpolation allowed between ADTTs.

Table 2.11. Oregon Generalized Live Load Factors for Legal Loads on State-Owned Bridges (ODOT 2015)

ADTT (one direction)	Live Load Factor, γ_L
Unknown	1.40
≥ 5000	1.40
$= 1500$	1.35
≤ 500	1.30

Table 2.12. Oregon Generalized Live Load Factors for Legal Loads on Local Agency Bridges (ODOT 2015)

ADTT (one direction)	Live Load Factor, γ_L
Unknown	1.45
≥ 5000	1.45
≤ 1000	1.30

2.5.1.3 Delaware

In 2012, researchers at the University of Delaware were tasked by the Delaware Department of Transportation (DelDOT) to load test a selection of slab bridges and determine their actual effective width, with the goal of developing new effective width formulas to be used in Delaware (Jones and Shenton 2012). The effective width of the slab affects the LLDF that is applied to it, which affects the live load effect on the bridge and therefore the RF.

In this study, a diagnostic load test was conducted on six slab bridges in the state of Delaware. The bridges varied from 8 ft to almost 20 ft in span length, with a 10 to 18 in. slab thickness and 26–47 ft width, and all were 80 years old or older. Truck axle weights and spacings were measured, and the trucks were driven slowly across each bridge at different transverse positions along the structure. In some cases, multiple trucks were driven side-by-side across the bridge. While the trucks were driven across the bridge, strain transducers placed on the underside of the slab—at midspan and spaced every 2 ft in the transverse direction—recorded the bridge response.

Longitudinal strain versus transverse transducer location plots were developed for each bridge from the data collected from the field tests. The area under the curve of these plots was used to convert the plot to one that had a constant strain with the same area under the graph. From this new plot, the measured effective width was found to be half the width of the constant strain graph. After conducting calculations to remove the multi-presence factor from the AASHTO LRFD equivalent width equations, the normalized AASHTO LRFD effective width was plotted against the aspect ratio—the length divided by width—of the bridge. On the same plot, the new field-measured, normalized effective widths were graphed against their aspect ratio. The equations of the new curves were converted back into the same form that is presented in AASHTO and in Table 2.13 are compared to the current AASHTO LRFD equivalent width equations.

Table 2.13. Delaware Equivalent Width Equations for Slab Bridges Compared to AASHTO LRFD Bridge Design Specifications Equations

	Effective width (in.) per lane for one lane loaded	Effective width (in.) per lane for multiple lanes loaded
AASHTO LRFD	$E = 10.0 + 5.0\sqrt{L_1 W_1}$	$E = 84.0 + 1.44\sqrt{L_1 W_1} \leq \frac{12.0W_1}{N_L}$
Delaware study	$E = 10.0 + 5.8\sqrt{L_1 W_1}$	$E = 84.0 + 2.06\sqrt{L_1 W_1} \leq \frac{12.0W_1}{N_L}$

Note:

- E = Equivalent or effective width (in.).
- L_1 = Modified span length taken to be equal to the lesser of the actual span length or 60.0 ft.
- W_1 = Modified edge-to-edge width of the bridge taken to be equal to the lesser of the actual width or 60.0 ft for multilane loading, or 30.0 ft for single-lane loading.
- W = Physical edge-to-edge width of the bridge (ft).
- N_L = Number of design lanes.

As is shown by multiple DOTs in their design manuals and examples, the inverse of the effective width in feet equals the LLDF for a 1 ft longitudinal strip of a slab bridge (IDOT 2012;

VDOT 2011; WisDOT 2017). Increasing the effective width, like the Delaware study has suggested, will decrease the LLDF for the loaded strip of a slab bridge. The increase will decrease the live load effect on that strip, thereby increasing the RF for that strip and the controlling RF for the bridge. DelDOT has included these new effective width formulas in its rating software and as a result has removed or improved posting restrictions on over fifty bridges in the state of Delaware (Arndt et al. 2017; Jones and Shenton 2012).

2.5.1.4 Texas

In 2010, Wood et al. (2010) developed the *Culvert Rating Guide* for TxDOT. The guide outlines the load rating procedure as per current AASHTO specifications with four levels of increasing complexity of demand modeling:

- Level 1 corresponds to culvert-specific frame analysis programs such as CULV-5.
- Level 2 incorporates two-dimensional general frame analysis programs with spring subgrade support, like Rapid Interactive Structural Analysis-2D (RISA-2D).
- Level 3 includes two-dimensional finite element soil-structure interaction programs with linear elastic finite elements, such as RISA-2D.
- Level 4 relies on higher order generalized programs, including non-linear two-dimensional models and three-dimensional models.

Load rating using the first three levels is outlined in the *Culvert Rating Guide*. In order to validate the *Culvert Rating Guide*, 100 culvert designs representing the TxDOT culvert inventory were load rated using the first three levels of analysis. It was noted that Level 2 analysis resulted in slightly higher load ratings than Level 1 analysis. However, if the soil is sufficiently stiff, Level 3 analysis could result in a higher load rating. Through parametric analysis, it was found that Level 3 analysis was relatively insensitive to the Poisson's ratio of the fill and lateral earth pressure. The highly sensitive parameters for Level 3 analysis were determined to be the modulus of elasticity for the soil and depth of fill. Field instrumentation and load tests were conducted on three in-service culverts and measured results were compared with predicted values. Although the analytical results were conservative, the procedures outlined in the *Culvert Rating Guide* help reduce over-conservatism in the load rating process.

Following the research done in 2010, Lawson et al. (2013) developed CULVLR: Culvert Load Rating, Version 1.0.0. This is a Windows-based software package that automates the load

rating process for reinforced concrete box culverts. Reinforced box culvert parameters, typical of TxDOT design standards, are provided as input in CULVLR. The capacity of each critical section of the culvert is determined using these inputs. The user can choose between Level 1 (CULV5) and Level 3 structural analysis to determine the design loads. Rating factors are calculated by CULVLR based on the calculated capacities and demands.

The influence of factors such as cover soil depth in a structural frame model, production simplified modeling sophistication, and live load attenuation method on the load rating of reinforced concrete box culverts was investigated by Wood (2015). A population of TxDOT cast-in-place reinforced concrete box culverts were load rated using the AASHTO MBE (AASHTO 2013) policy guidance and a direct-stiffness demand model for a range of cover soil depths. Cover soil depth above the culvert was found to be a critical parameter for load rating and design of reinforced concrete box culverts. Live load test data from three instrumented, reinforced concrete box culverts under four cover soil depths were used to evaluate two production-simplified culvert load rating demand models. The production-simplified soil structure interaction model was found to be more exact than the AASHTO-recommended production-simplified structural frame model. In comparison to the traditional top-slab-calibrated live load attenuation method, precision of the live load estimate was approximately doubled by using a depth-calibrated, out-of-plane, live load attenuation model. These improvements to the load rating calculations helped reduce the discrepancy between field inspection observations and load rating calculations of culverts.

2.5.2 Field Testing of Bridges

One of the common practices for bridge load rating is field testing. Vehicles with known weight are driven over the bridge at various configurations, and the behavior of the bridge is studied. The load rating of the bridge is determined based on the measured response of the bridge.

Experimental load testing on a bridge is of two types: diagnostic and proof testing. The bridge is subjected to a load closer to its rated capacity in a diagnostic test. The corresponding response is used to develop a numerical model to help estimate the maximum allowable load. Conversely, in a proof test, the bridge is subjected to incremental loads until the targeted load or a limit state is reached. Thus, the operating capacity of the bridge is determined (Chajes et al. 1997).

Kaliber et al. (1997) conducted an extensive experimental testing program of reinforced concrete bridges that were either deemed deficient or load rated based on the *AASHTO Manual for*

Maintenance Inspection of Bridges (AASHTO 1983). The experimental tests revealed that the majority of the bridges tested had much greater load carrying capacity than predicted by traditional analytical models. This test showed that the traditional analytical models were overly conservative, leading to certain bridges being load rated and others being deemed unfit for use.

2.5.2.1 *Monitoring*

The structural integrity of a bridge and the characteristics of any damage to it can be examined through various tools of structural health monitoring. Several studies are highlighted below.

Kashif Ur Rehman et al. (2016) noted the importance of frequent condition assessment and health monitoring of bridges because undetected damage to bridges could significantly affect their structural integrity and service capacity over time. There are two broad classifications of structural health monitoring: (1) global, and (2) local. In global health monitoring, the focus is on the detection of damage within the structure, while in local health monitoring, the location, extent, and severity of damage is determined. Nondestructive testing and evaluation provide a means for effective examination and monitoring of aging structures. These methods help identify damage, thereby averting unpredictable failure of structures. Various audiovisual methods, stress wave methods, electromagnetic methods, and deterministic methods for nondestructive testing and evaluation of bridges and their applications and limitations were outlined by Kashif Ur Rehman et al. (2016). Although visual inspection is extensively used for monitoring concrete structures due to its prompt and inexpensive nature, it fails to identify defects at an interior level. Therefore, the authors recommend using electromagnetic methods and coring for accurate monitoring of concrete bridges.

Washer (1998) discussed the use of nondestructive testing for load rating of highway bridges. A bridge is instrumented with accelerometers, strain gauges, and deflection gauges that are used to record the structural response to an applied load. This response is then analyzed to determine the capacity of the bridge. However, Washer (1998) noted that bridge instrumentation can be a time-consuming and disruptive procedure that could be eliminated by remote measurement systems. Remote noncontact deflection measurements can be precisely obtained using a frequency modulated laser. Scanning from a single point over large sections is achieved by fitting the laser with servo-controlled motors, thus requiring no targets for measurements. This is the design principle of the laser radar system. A wireless bridge monitoring system using a

network of microwave transponders and a controller has also been developed by the nondestructive evaluation (NDE) research program of FHWA. Data such as strain, rotation, deflection, and vibration are collected by the transponder and downloaded to the local transponder, thereby eliminating the long wire runs from the instruments to the data acquisition system and any associated noise (Washer 1998).

Santini-Bell et al. (2013) conducted a case study highlighting the advantages of a bridge rating procedure incorporating three-dimensional structural modeling, structural health monitoring, and nondestructive testing during the design phase. During fabrication, they installed strain gauges on the girders of Powder Mill Pond Bridge in Massachusetts, which is a three-span, continuous composite steel-girder bridge with a reinforced concrete deck. Prior to opening the bridge to traffic, it was subjected to pseudo-static loading, and the results were used to calibrate a baseline structural model. The three-dimensional structural model for bridge management and experimental load rating was developed in SAP2000. It consisted of frame elements and shell elements representing the steel girders and concrete deck, respectively. The rating factor was calculated using the AASHTO LRFR method and was lower than that obtained from the three-dimensional finite element model. Two scenarios for girder section loss were simulated in the model, one for an external girder and one for an internal girder. Girder RFs for the two scenarios were first calculated assuming section loss over the entire length and then assuming section loss over the damaged area only. In the AASHTO LRFR procedure, the girder RFs were calculated assuming section loss over the entire girder length. The resulting RFs were again observed to be lower than those obtained from the cases modeled. Thus, the authors called for adjustments in the overall process of bridge design, inspection, and rating to incorporate instrumentation and modeling to accurately rate bridges.

The theoretical model for lifetime prediction of a structure usually involves a load model, a system transfer model, and a damage model (Peil et al. 2005). Because one model serves as an input to the next model, the systematic and random errors flow together, and the reliabilities are multiplied, causing the lifetime prediction to be unreliable. By monitoring random strains at critical points of the structure and some laboratory tests, the uncertainties in the load, transfer, and damage model may be minimized, thus improving the reliability of the results. To avoid the damage model, a generated time history is used as an input to a test rig that is digitally controlled, in which a sample of the actual construction detail is tested. The artificial time history contains the overall

statistics of the load process, which includes the past and future loading. If the specimen survives the test, then the actual detail in the structure will survive, as well. However, the hot spots in the structure must be determined beforehand using a finite element analysis.

2.5.2.2 *Nondestructive Testing*

Nondestructive testing, such as computer vision methods, remote sensors, and artificial neural networks (ANN) can assist in analyzing the structural condition and load carrying capacity of bridges. Several related studies are summarized below.

Catbas et al. (2012) proposed a novel approach for structural health monitoring of bridges using computer vision techniques. Together with operating traffic image and traditional sensor data, this methodology can be extended to determine load rating of bridges. After consultation with bridge engineers, the Sunrise Bridge in Fort Lauderdale, Florida, was selected for monitoring due to its typical condition, material, and geometric characteristics. This 1989 movable bridge has double bascule leaves carrying three lanes of traffic. Strain gauges, accelerometers, and tiltmeters were installed across the bridge to monitor its response to operating traffic. A video camera was installed at the bridge entrance to capture images of the vehicles during operation. These image data were matched with sensor data to record the bridge response for a given type of vehicle and to extract the unit influence line (UIL). Two UILs were developed, one corresponding to a fully loaded bus and another to an empty bus. These UILs could then be used to estimate the upper and lower bounds for the load rating of a bridge. An FEM model of the bridge, using shell elements, frames, rigid links, and solid elements, was developed in SAP2000. The results from the model were used to validate the load rating from operational traffic and UILS, both of which showed excellent correlation.

Alipour et al. (2017) proposed another new methodology incorporating machine learning for safety screening and load rating of bridges. The national concrete slab bridge dataset from the NBI was used for training two classification algorithms: decision trees and random forests. The fundamental relationship between various bridge properties and their load-posting status was used to train the models. Validation on an independent dataset and comparison with existing schemes were carried out. The results were also compared with DOT practice as a baseline. Load postings of bridges with missing plans from the dataset were determined using the models. Good correlation between these results and the judgment-based bridge load postings was observed. The aim of the

proposed procedure is to provide a data-driven management technique, thereby improving resource allocation.

Ji and Zhang (2012) created a method to measure displacements of civil structures using a digital camera to record planar targets placed on vibrating structures. The computational connection between real space and the image plane was determined using computer vision. Point reconstruction rules were used to quantify the structural displacement at the location of the target. The displacement measurement method consists of three steps: camera calibration, object tracking, and point reconstruction. A fully calibrated camera is needed to detect all intrinsic and extrinsic camera parameter geometric data. The intrinsic parameters specify the optical and geometric attributes of the camera. Extrinsic parameters are used to transform the pixel coordinate system of the image into a prespecified object coordinate system. In this study, three different experiments were carried out to validate the suggested technique. In the first experiment, a fixed planar pattern on a shake table was measured. In the second, the same pattern was placed at the end of a cantilever beam to measure the displacement during free vibration. In the last experiment, the dynamic displacement of a girder bridge was measured (Ji and Zhang, 2012).

Fukuda et al. (2010) developed a vision-based method for measuring displacements. In this method, the dynamic response of large-scale structures such as bridges and buildings is monitored in real time. The system consisted of a digital camcorder, a telescopic lens, a computer with image-processing software, and a target plate tagged with a geometric pattern. The panel is placed at a specific location to detect displacement of the structure, and the target is captured on the video camera from a distant reference point. Alternatively, a geometric pattern placed on the structure may be used instead of the panel. The displacement of the structure was computed using an image-processing software calibrated with premeasured parameters. The image-processing algorithm of this software identifies the geometric pattern on the target panel captured by the digital video camera. In the last module, the effective displacement from the discerned geometric pattern is calculated (Fukuda et al. 2010).

Khuc and Catbas (2017) created a completely contactless structural health monitoring system for structural vibration and displacement monitoring. To realize completely contactless monitoring, the need of physical targets was eliminated by using a special type of virtual mark named “imaging key-points.” In the vision system, a low-cost camcorder, a laser distance measurement device, and other equipment such as a checkerboard and a tripod are used. In

comparison to conventional vision-based displacement measurement systems where physical targets are used as a marker, imaging key points were used as virtual markers to take advantage of their repeatable characteristics. The motion of a local area can be received by tracking the picture key points of the area captured by this system. These points describe textures of a definite location in pictures, such as maxima of brightness, object corners, or color. Because of a constant zooming factor, the extracted key points are expected to be invariable with regard to translation, rotation, and variation in illumination.

The study of Yoon et al. (2016) depicts a target-free, vision-based system for displacement measurement and structural system identification. In comparison to Fukuda et al. (2010), this method can detect displacements without any target. This method uses different consumer-grade cameras and different algorithms to measure the displacement of an object. The method consists of three process modules: camera calibration, vision-based displacement measurement, and system identification. The vision-based displacement measurement module consists of different steps that describe different processes, like the selection of the region of interest box, the feature detection, the feature tracking, and the outlier detection. During these processes, several methods and algorithms are used. The result of this module is the displacement of an object. The displacement can be quantified in pixels by tracing the center of the region of interest. Apart from the displacement measurement, this method uses system identification in the next step to analyze the dynamic characteristics of the object. This technique serves to calculate a mathematical model that represents the physical structure. To validate the system, the displacement of a six-story building model was measured in a laboratory. The displacement results from this study showed more accurate deliverables in the lower frequency section of the frequency response function by using the vision-based system rather than accelerometer-based measurements. Otherwise, there was noise apparent in the higher frequency sector (caused by temporal aliasing) when using the vision-based system with the lower frame rate camera.

Chajes et al. (1997) conducted a nondestructive experimental load test on a three-span, load-posted, slab-on-steel girder bridge. The 1940 bridge was designed as a non-composite structure with three simply supported spans. The bridge has undergone numerous repairs over the years and was load posted in 1994 using the Bridge Rating and Analysis of Structural Systems (BRASS) program (AASHTO 1987). The test results indicated composite action between the girders and deck and substantial restraint at the bearing supports. A two-dimensional FEM model

with plate elements and one-dimensional beam elements representing the deck and girders, respectively, was developed. The composite section properties and bearing restraints observed in the test data were used in the model. The results obtained from the model indicate that the bridge has a higher load-carrying capacity and that the load posting may not be required.

Aguilar et al. (2015) evaluated the load rating of a prestressed concrete double T-beam bridge without plans by field testing. This investigation involved a 31 ft single-span prestressed concrete bridge with nine double T-beams. The procedure for the bridge load rating involved estimating eccentricity and the number of prestressing strands, detecting the primary steel reinforcement, performing a diagnostic load test to measure beam strains under truck loads, and, finally, performing a proof test with dump trucks to obtain the largest midspan moment. The authors concluded that if no excessive signs of damage or distress are found during an in-depth inspection, the diagnostic test may be excluded, provided that there is confidence that the behavior of the bridge can be predicted. Furthermore, load testing may be avoided altogether if high-quality images of the prestressing strands can be obtained from a rebar scanner and the bridge is in good condition. In this case, the bridge may be modeled and rated analytically on the basis of the bridge properties determined from the design specifications and the Magnel diagrams.

2.5.2.3 *Destructive Testing*

During destructive testing, a bridge is loaded to its ultimate capacity and its response is extensively studied. Comparison of the experimental capacity with the analytical prediction of capacity can help identify areas of conservatism in the current design and load rating practices. However, full-scale destructive testing of bridges is not a common practice because the bridge is damaged by the applied loading. The magnitude of load required to conduct a destructive test typically necessitates the design of an intricate and costly loading system. The availability of decommissioned bridges to conduct such tests is limited, thus rendering destructive testing an atypical approach. Additionally, the conclusions drawn from the destructive tests cannot be generalized due to the specific characteristics of the examined bridges.

The outcome of the majority of destructive tests is that bridges have a higher capacity than that prescribed by the existing design and rating practices. Jorgenson and Larson (1976) demonstrated this increase in ultimate capacity for a three-span continuous reinforced concrete bridge through destructive testing. The material strengths of steel and concrete components of the

bridge were also determined to be higher than design minimum values through laboratory testing of steel coupons and concrete cores. A three-span reinforced concrete slab bridge was similarly tested to its ultimate capacity by Miller et al. (1992), and the factor of safety was found to be seven in comparison to the design loads. Aktan et al. (1994) proved a 134 percent increase in the safety factors calculated by the procedures outlined in the *AASHTO LRFD Specifications* (AASHTO 1994a) by load testing two steel bridges to failure.

Bechtel et al. (2011) tested a 1/5-scale steel I-girder bridge of the Delaware River and Bay Authority Bridge 7R to its ultimate capacity. Similitude between the model and specimen bridge was ensured through the Buckingham π -theorem. The Buckingham π -theorem states that if an independent function is made up of physical variables, it can be transformed into another function made up of dimensionless π terms. The scaled AASHTO HS20 truck load was applied along the centerline of one of the interior girders and the strain response recorded. It was concluded from the destructive test of the scale model that the diaphragm and deck conditions affect the bridge load distribution characteristics, and at deck failure the steel girders had substantial reserve capacity. The potentially significant effect of construction-induced stresses is emphasized in the experimental results, and knowledge of their possible effects is important. The experimental results were compared with analysis using an FEM model developed in ABAQUS, and the results were in good agreement.

Miller et al. (1994) conducted a destructive test on a decommissioned reinforced concrete slab bridge. The bridge had a skew angle of 30 degrees, two lanes, and severely deteriorated shoulders. It was incrementally loaded to a maximum load of 720 kips (3200 kN) by means of a hydraulic loading system designed to simulate the HS20-44 truck loading. During the various stages of the incremental loading process, behavioral changes were observed in the bridge response that may be attributed to the boundary conditions. Shear failure initiated in the deteriorated shoulders over the piers and the crack propagated through the slab. At the maximum load of 720 kips, which is equivalent to 10 HS20 trucks, the slab failed in punching shear. This failure load was found to be significantly lower than that predicted by the AASHTO guide specifications (AASHTO 1989), suggesting that the shoulder deterioration affected the failure mode. Bridge design load (demand) and capacity were computed via three methods: effective strip model, linear finite element analysis, and nonlinear finite element analysis. Core samples from the bridge were tested to determine the material properties for concrete and steel, and these values were used in the

analytical models. It was found that the effective strip model gave highly conservative estimates of bridge capacity. The flexural demand obtained from linear finite element analysis was 45 percent lower than that determined from the effective strip model because of its capability to account for load distribution mechanisms in the slab, such as slab action and effect of skew. Nonlinear finite element analysis yielded a demand approximately similar to that obtained from the linear finite element analysis due to bridge failure prior to the onset of nonlinear behavior. However, due to the consideration of factors such as slab action, membrane action, cracking, effect of skew, boundary effects, and actual material responses, nonlinear finite element analysis is expected to provide improved results.

Lantsoght et al. (2016) field-tested a reinforced concrete slab bridge to failure, then laboratory-tested three beams sawn from the bridge. This research was carried out because several existing reinforced concrete slab bridges in The Netherlands were found to be unsatisfactory under shear assessment. The example bridge has five spans supported over four cross-beams cast integrally onto the piers and a skew angle of 18 degrees. During the field test, two spans were loaded with the tandem load outlined in the *Eurocode 1: Actions on Structures* by Comité Européen de Normalisation (EN 1991-2 2011). Although flexural distress was observed in the first span, failure due to flexure was not achieved. This result may be due to the integral nature of the bridge, whereby the moment capacity of the span is higher through the effect of support moments rather than through assumed hinge supports. In the second span, flexural failure was achieved for increased applied load. It was found that the EN 1991-2 (2011) estimated the flexural capacity of the bridge satisfactorily; however, the shear capacity of the bridge was higher than that predicted by the EN 1991-2 (2011).

Azizinamini et al. (1994) investigated the performance of concrete slab bridges that were designed for H15 truck loading. A concrete slab bridge that was decommissioned was experimentally tested using HS20 trucks. The tests showed that the behavior of the bridge was linear-elastic even when loaded with four HS20 trucks in each lane. The field-testing results showed that there was significant reserve strength in the bridge that was not accounted for by the AASHTO Bridge Analysis and Rating System (AASHTO 1991). The authors also conducted a detailed analytical study using FEM analyses and yield line analyses incorporating a moment curvature approach together with actual material properties. Based on the experimental and analytical investigations conducted, it was concluded that the concrete slab bridge possessed

significant reserve capacity and that the yield line analysis was able to accurately predict the strength of these types of bridges. The differences in the results from the FEM analysis and those from the traditional analysis was attributed to the actual material properties being higher than the design values specified and participation of non-structural elements, such as curbs, in the load-carrying capacity.

2.5.3 Investigation of Material Properties

Standard values for the material properties of the bridge components have been adopted in the AASHTO MBE (AASHTO 2011), as outlined in Section 1.3.5 of this report. However, while intended to be conservative, these values may not accurately represent the true material properties of the structure. Estimated material strengths may result in less accurate load ratings of bridge structures. The true material properties can be obtained through laboratory testing when further refinement to the load rating process is desired.

2.5.3.1 Nondestructive Testing

Pucinotti (2015) outlined the results of numerous destructive and nondestructive tests performed on the historic National Museum of Reggio Calabria in Italy for concrete strength assessment. Laboratory tests were carried out on cores extracted from selected structural elements to determine the compressive strength of concrete. These results were compared to the estimated concrete compressive strength based on nondestructive testing. The three nondestructive methods highlighted in this paper are the rebound method, the ultrasonic pulse velocity (UPV) method, and the combined Sonic-Rebound (SonReb) method. A correlation curve was developed for each indirect method and the results calibrated with laboratory test results of concrete cores. Satisfactory results were obtained from the UPV method when compared with the results from the destructive tests. However, the author observed that the accuracy of the assessment of the in-situ compressive strength increased when combined methods are engaged.

Although material testing of the samples gives more accurate strength values, recent research has shown that strength properties of concrete and steel can accurately be inferred from nondestructive test data (Huang et al. 2011; Schindler 2005). Huang et al. (2011) explored using the UPV and the rebound number (RN) for predicting the concrete compressive strength of existing structures. While the RN gave information about the concrete strength at the surface of the

concrete, the UPV test reflected interior concrete properties. The authors developed a probabilistic model to predict the compressive strength of concrete using information from rebound hammer and UPV tests. The water-cement ratio and the age of the concrete were taken into account in their formulation; results showed the concrete compressive strength modeled well when compared to other regression models.

Breysse (2012) reviewed past models to estimate the concrete strength based on NDE methods. The quality and efficiency of the strength estimation depended on the sensitivity of concrete strength to the NDT measurement, the variation of the measured NDT values in relation to the range of variation of concrete strength being investigated, and the magnitude of error in the measurement. Several empirical models to establish a relation between NDT measurements and concrete strengths have also been proposed in the past. However, it was observed that the model error was very small when compared to the measurement error. It was also shown that combining NDT measurement techniques whose quality of measurement are of the same order could produce effective concrete strength measurements. Analysis of datasets from laboratory tests show that combining rebound hammer and UPV tests provided a good quality measurement of the concrete strength.

2.5.3.2 Destructive Testing

In 2006 and 2009, at the Lulea University of Technology in Sweden, Larsson (2006) examined and tested samples from steel bridges. Samples from Swedish bridges built between 1901 and 1940 had previously been tested, and the results were compiled into a database. It was found that steel from bridges built between 1901 and 1919 showed a 34 percent increase in yield strength and a 29 percent increase in ultimate strength when compared to the Swedish code requirement for the time period. However, steel from bridges built between 1919 and 1940 showed an 8 percent increase in yield strength, but a 4 percent decrease in ultimate strength from the code values. Larsson (2009) also conducted tensile tests on rivets from riveted bridges built around the same time period. He found that, in all cases, the rivets exhibited higher yield and ultimate strengths than required by the Swedish code. In some cases, this increase was as high as 40 percent for the yield strength and 30 percent for the ultimate strength of the rivets. Although some of Larsson's testing is inconclusive, especially for steel bridges built between 1919 and 1940, his study suggests

that historical steel yield and tensile strengths are typically higher than specified values, with the magnitude of these increases varying depending on the period of construction.

Wolf and Poulin (2015) released a report examining the methods used to obtain actual steel strengths in historic buildings. As part of the report, they conducted tensile testing on steel coupons from two renovation projects in Manhattan, New York: a 1920 residential building and a 1922 bank building. They found that the steel tensile strength was 50 percent higher than the current code design value for the residential building and 35 percent higher for the bank.

Guidelines for obtaining concrete samples have been outlined in ASTM C 823 (2017) and discussed in the ACI Committee 437 report *Strength Evaluation of Existing Concrete Buildings* (Subcommittee I 1967). The committee recommended that the cores should be tested in moisture conditions representative of the in-situ concrete. It has been found that the core compressive strengths are lower for cores obtained from the upper portions of slabs and beams than from lower portions of the same. Due to the nonuniform nature of concrete, the committee recommended obtaining cores from different locations within the structure for laboratory testing, and the average value should be taken as the compressive strength. The committee recommended the use of information from mill tests for reinforcing steel only if the engineer and the building official approve. In case of disagreement or in the absence of mill test results, important guidelines for extraction and testing of reinforcing bars were outlined by the committee. The size of the sample and type of yield strength test should follow ASTM A370 (2017). Locations where the reinforcement is subjected to minimum stress are ideal for procurement of samples. However, to avoid deterioration in strength of the structural member, no two samples should be obtained from the same location.

Ozyildirim and Carino (2006) outlined the procedure for testing of drilled cores and sawed beams from existing structures. Sawed beams are tested in flexure to determine the modulus of rupture while compression or splitting tension tests are carried out on drilled cores. Although cutting of concrete specimens may be costly, drilling or sawing may affect strength test results due to the possibility of damage to a specimen. Reduction in strength may also be observed due to excess voids in samples obtained from areas of poor consolidation. Furthermore, the strength test results are affected by the location and orientation of the sample in the structure. A five percent reduction in concrete strength has been shown by Bloem and Gaynor (1965) for cores taken from the top as opposed to those taken from the bottom of an 8 in. slab. Another factor that may affect

the measured compressive strength of a concrete core is the amount of stress within the structure. Cores from highly stressed regions have lower strength compared to those from under-stressed regions. This may be due to the micro-cracks present in regions of high stress. Lower concrete strengths have also been observed in cores drilled horizontally compared to those drilled vertically (Mindess et al. 2003; Neville 1996; Suprenant 1985). The weak interfacial regions aligned in the loading direction of horizontally drilled cores, as shown in Figure 2.6(b), result in lowered strength values. These interfacial regions are caused by the accumulation of water under coarse aggregates, otherwise known as bleeding.

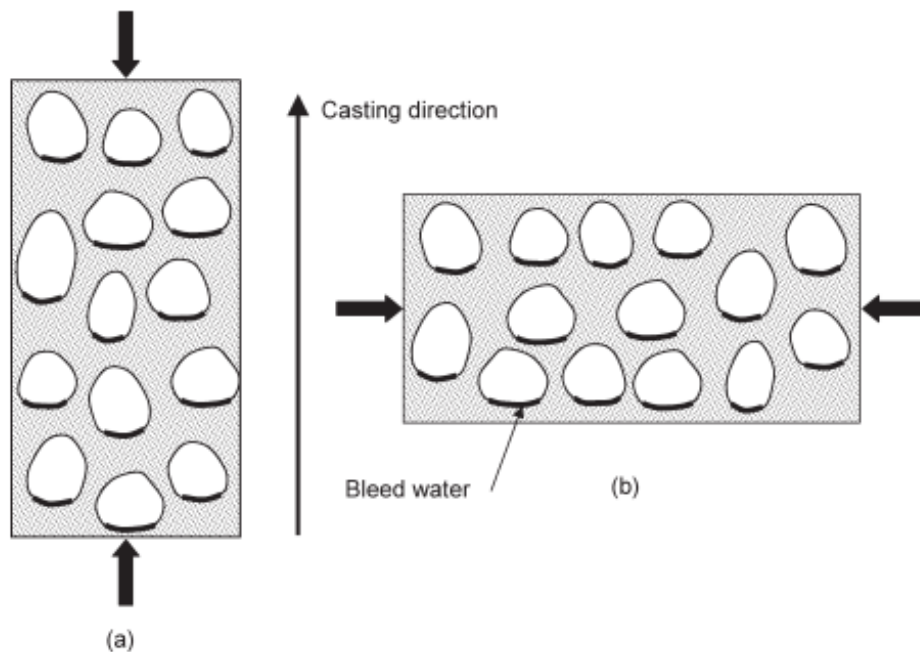


Figure 2.6. Planes of Weakness Due to Bleeding: (a) Axis of Specimen Vertical; (b) Axis of Specimen Horizontal (Ozyildirim and Carino 2006)

Neville (2011) explained the process of in-place concrete core testing and the criteria governing the required characteristics of core samples, such as core diameter, length-diameter ratio, and moisture condition. Bartlett and MacGregor (1994; 1995) recommended a process for assessing the equivalent strength of in-place concrete using core tests. After selecting the member or region to be cored, core samples are obtained following the standard procedure outlined in ASTM C42/C42M-16 (2016). The core strengths, f_{core} , should be converted to the corresponding in-place strengths, f_{cis} , using the following equation:

$$f_{cis} = f_{core}(F_{l/d}F_{dia}F_r)(F_{mc}F_d) \quad (2.12)$$

where:

$F_{l/d}$ = Correction for length/diameter ratio as given in ASTM C42/C42M-16 (2016)

F_{dis} = Correction for diameter of core
= 1.06 for 2 in. cores, 1.00 for 4 in. cores, and 0.98 for 6 in. cores

F_r = Correction for the presence of reinforcing bars
= 1.00 for no bars, 1.08 for one bar, and 1.13 for two bars

F_{mc} = Accounts for the effect of moisture condition of the core at the time of the core test
= 1.09 if the core was soaked before testing, and 0.96 if the core was air-dried at the time of the test

F_d = Accounts for damage in the surface of the core due to drilling
= 1.06 if the core is damaged

As specified by Bartlett and MacGregor (1995), outliers in the set of f_{cis} should be identified. Finally, the equivalent specified strength, f'_{ceq} , may be computed from f_{cis} by using the following equation:

$$f'_{ceq} = k_2 \left[\bar{f}_{cis} - 1.282 \sqrt{\frac{(k_1 s_{cis})^2}{n} + \bar{f}_{cis}^2 (V_{l/d}^2 + V_{dia}^2 + V_r^2 + V_{mc}^2 + V_d^2)} \right] \quad (2.13)$$

where:

\bar{f}_{cis} = Mean f_{cis} after removal of outliers

s_{cis} = Sample standard deviation after removal of outliers

k_1 = A factor dependent on the number of core tests, after removal of outliers, equal to 2.40 for 2 tests, 1.47 for 3 tests, 1.20 for 5 tests, 1.10 for 8 tests, 1.05 for 16 tests, and 1.03 for 25 tests

k_2 = A factor dependent on the number of batches of concrete in the member or structure being evaluated, equal to 0.90 and 0.85, respectively, for a cast-in-place member or structure that contains one batch or many batches, and equal to 0.90 for a precast member or structure

n = Number of cores after removal of outliers

- $V_{l/d}$ = Coefficient of variation due to length/diameter correction, equal to 0.025 for $l/d = 1$, 0.006 for $l/d = 1.5$, and zero for $l/d = 2$
- V_{dia} = Coefficient of variation due to diameter correction, equal to 0.12 for 2 in. diameter cores, zero for 4 in. cores, and 0.02 for 6 in. cores
- V_r = Coefficient of variation due to presence of reinforcing bars in the core, equal to zero if none of the cores contained bars, and to 0.003 if more than a third contained bars
- V_{mc} = Coefficient of variation due to due to correction for moisture condition of core at time of testing, equal to 0.025
- V_d = Coefficient of variation due to damage to core during drilling, equal to 0.025

2.5.4 Modeling and Analysis

Finite element models, developed using various commercial software packages, provide tools for studying bridge behavior and response to different types of loading, for bridge load rating, and for undertaking parametric studies. Several studies are described below.

Hueste et al. (2015) investigated the practicability of spreading out precast slab beams using 4 in. thick precast concrete panel (PCP) spanning in between as stay-in-place forms topped with a minimum 4 in. thick cast-in-place reinforced concrete deck. Design guidelines for this type of spread slab beam bridge system were developed. The feasibility of the spread slab beam bridge was confirmed by testing a full-scale specimen of the bridge at the Texas A&M University Riverside Campus. The bridge was instrumented with load cells, spring potentiometers, and accelerometers, and tested under service loads. Similarly, a spread slab beam bridge on US 69 in Grayson County, Texas, was instrumented and field-tested. The measured bridge response was used to verify two computational models, grillage method and FEM, to examine the load sharing behavior for a range of bridge geometries. The grillage method is the most basic computational modeling approach, in which the deck and girders are assumed to be a mesh of beam elements in orthogonal directions. The grillage model of the two bridges were developed in SAP2000 using longitudinal frame elements for the slab beams and transverse beams to distribute the applied deck loads. Three-dimensional FEM models of the two bridges using solid brick elements were developed using two commercial software programs, ABAQUS (Dassault Systemes 2013) and CSiBridge (Computers and Structures 2015). The FEM models provided more accurate deflections

and LLDFs. Using the developed modeling approach and parameters, parametric studies were conducted, and shear and moment LLDFs for the design of spread slab beam bridges were developed.

Yousif and Hindi (2007) compared the AASHTO LRFD LLDFs with those obtained from a FEM model that was developed for simple-span slab-on-girder concrete bridges. A thorough investigation of the LLDFs prescribed by *AASHTO LRFD Specifications* (AASHTO 2004) in regard to span length, slab thickness, girder spacing, and longitudinal stiffness was conducted. Finite element models for a wide range of bridges were developed in SAP2000. Several models were investigated to determine an accurate representation of the superstructure. The selected model idealizes the bridge as a two-dimensional system (Figure 2.7). The model uses four-node quadrilateral shell elements with six degrees of freedom (DOF) at each node to represent the bridge deck, and two-node space frame elements with six DOF at each node to represent the girders and end diaphragm beams. Hinges and rollers were used to model the simply supported span of the bridge. The models were subjected to vehicular load and standard lane load as prescribed by *AASHTO LRFD Specifications* (AASHTO 2004). Each bridge was loaded in one-lane, two-lane, and three-lane loading scenarios. It was concluded that the AASHTO LRFD LLDFs are conservative compared to the analysis results from the finite element models.

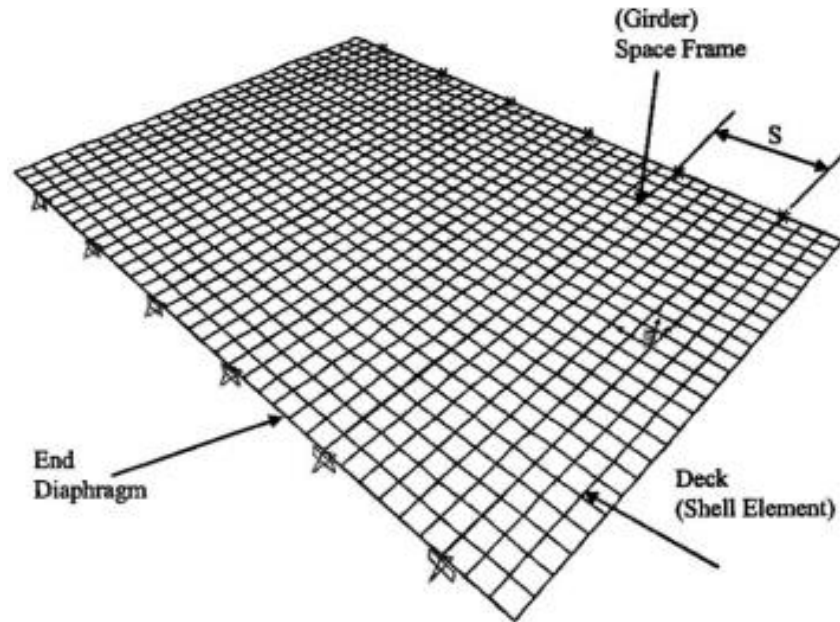


Figure 2.7. Finite-Element Model Using SAP2000 (Yousif and Hindi 2007)

Suksawang and Nassif (2007) carried out a parametric study to determine the effect of bridge parameters on the LLDFs. Five steel I-girder bridges were modeled and analyzed using ABAQUS (Dassault Systemes 2013) software. A comparison between two different idealizations (shell-shell and shell-beam) resulted in the selection of a shell-beam model (Figure 2.8) due to reduced computation time. In this model, the concrete deck is represented by four-node shell elements with six DOF while two-node beam elements with six DOF represent the steel girders. A rebar element was used to model the reinforcing bars in the concrete deck. All material properties were based on laboratory testing results of identical samples. The connectivity between the shell and beam elements was modeled using multiple-point constraint (MPC), and the end supports were modeled using roller-pin boundary conditions. The bridge response obtained from the analysis of the FEM models was compared with field-test results of the five bridges. It was found that the LLDFs were sensitive to girder spacing and stiffness. After a thorough comparison of the various LLDF equations and verification against field-test results, simpler yet accurate equations for LLDFs were proposed. These equations are of the traditional *S*-over form and independent of the girder stiffness.

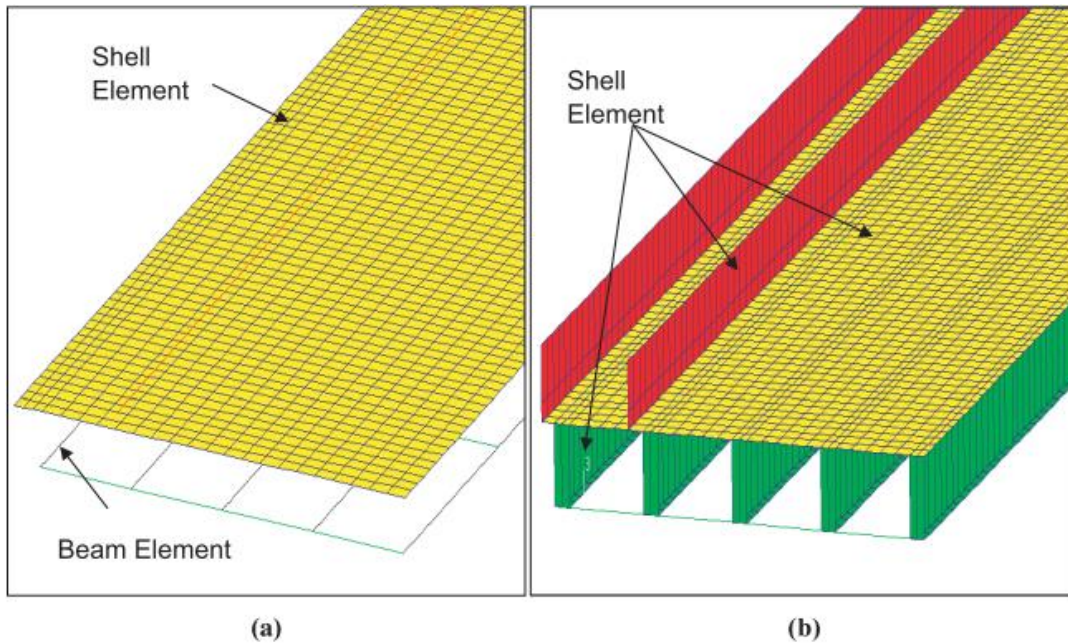


Figure 2.8. Finite Element Models: (a) Shell-Beam Model; (b) Shell-Shell Model (Suksawang and Nassif 2007)

Barth and Wu (2006) studied the ultimate load behavior of slab-on-steel girder bridges using three-dimensional, nonlinear finite element analysis. The bridge model was created in ABAQUS (Dassault Systemes 2013) with four-node shell elements representing the girders, slab, and stiffeners; one-dimensional strain theory elements representing the steel reinforcement in the concrete slab, and two-node Timoshenko beam element representing cross frames. Multiple-point constraint beams modeled the full composite action between the concrete deck and steel girders. The finite element model was validated against ultimate test results from two simple-span steel girder bridges and a four-span continuous steel girder bridge. In the model, concrete behavior was represented by two methods: smeared concrete cracking and concrete damaged plasticity. It was found that the ultimate behavior of the simple-span bridges was efficiently captured by the smeared crack model, while that of the continuous span bridge was captured reasonably well by the concrete damaged plasticity model.

Dauids et al. (2013) developed a finite element model for load rating flat slab bridges. The slab moments predicted from the finite element model were compared to the results from live load testing of an in-service instrumented flat-slab bridge. It was found that the finite element model gave conservative predictions of the slab moments when compared to the live load tests. Fourteen flat slab bridges were load rated with the developed finite element model and the equivalent strip

method—a simplified method of analysis for flat slab bridges prescribed by AASHTO. Despite the conservatism of the developed finite element model, it predicted higher rating factors when compared to the equivalent strip method. This study showed that a significant amount of flat slab bridges with low rating factors according to the AASHTO equivalent strip method may have the potential for increased rating factors.

Sanayei et al. (2016) presented a comparison of three approaches used to load rate a concrete-steel composite bridge based on the LRFR approach. The three approaches included: (1) the conventional design office load rating technique using a simplified line girder analysis, (2) a modified conventional load rating based on input from measurements from diagnostic load tests, and (3) a finite element modeling approach that accounted for the 3D structural system behavior. In the three-span, continuous composite concrete slab on steel girder bridge, that was part of this investigation, it was consistently found that the load rating of the girders based on nondestructive diagnostic tests and the FEM model were greater compared to the load rating based on the conventional method.

2.6 POTENTIAL AREAS OF REFINEMENT TO LOAD RATINGS

Several potential areas of refinement to the load rating process have been identified and are summarized below.

2.6.1 Live Load Distribution Factors

The current LLDFs that are in the *AASHTO LRFD Specifications* (AASHTO 2017) are approximate equations that were developed in an effort to represent bridges with a large range of geometric characteristics using simplified expressions. Multiple research projects have found, through field measurements and FEM analysis of existing bridges, that the AASHTO LLDFs can be inaccurate when compared to the measured LLDFs and LLDFs based on refined models of the bridge. The AASHTO LLDFs tend to over-distribute the moment to interior girders and distribute less moment to the exterior girders (Li and Chen 2011). The AASHTO LRFD LLDFs can range from being accurate to quite inaccurate (Hueste et al. 2015). In a report published in 1996 (Chen and Aswad 1996), researchers at Pennsylvania State—Harrisburg conducted FEM analysis of prestressed I-beams and spread-box beams. Although the research focused on improving span lengths, it also found that the LLDF formulas in the 1994 *AASHTO LRFD Bridge Design*

Specifications, which are the same as in the current specifications, were conservative when compared to distribution factors found using FEM. In some cases, the *AASHTO LRFD Bridge Design Specifications* produced distribution factors that were almost 20 percent higher than those found from FEM analysis.

If the live load moment effect on the controlling girders of a bridge can be reduced, the RF of that bridge will increase as long as the girder is the controlling rating member. Furthermore, according to the *TxDOT Bridge Inspection Manual*, the LFR is used to load rate all on-system bridges that were not designed using LRFD (TxDOT 2013). The requirement by the FHWA to use LRFD was enforced as of 2007 (Lwin 2006). This fact means that most bridges in Texas that are not passing load rating are most likely being rated using LFR. The use of LFR LLDFs could significantly affect RFs. Although determining more accurate LLDFs may not affect all bridges, it could be an important area of investigation.

2.6.2 Composite Action

A deck-girder bridge exhibits a higher capacity when the deck and girders act compositely with one another, as opposed to if they act in a non-composite manner, because under composite action, the girders and deck work together to resist the applied loads, whereas under non-composite action, the girders and deck resist the applied loads separately. Composite action increases the moment of inertia and raises the neutral axis of the bridge cross-section. This combination results in an increased section modulus, which increases the moment capacity of the structure. If slab-on-steel girder bridges originally designed as non-composite structures could be shown to exhibit some degree of composite action, and this behavior could be quantified, their moment capacity could be increased.

2.6.3 Possible Areas of Improvement for Slab Bridges

One cause of low rating factors in slab bridges is related to the steel reinforcing bar cutoffs or development lengths. These lengths are specified in the relevant AASHTO bridge design specifications from the year that the bridge was designed. In NCHRP Synthesis 359, *Bridge Rating Practices and Policies for Overweight Vehicles* (Fu and Fu 2006), a survey was sent to DOT officials in 2006, and 44 US state DOTs and 10 Canadian provincial departments responded. One of the questions in the survey asked how the bar cutoffs are determined in their load rating

procedures. Each department gave essentially one of two different responses. One was that they remove the development length, or some portion of the development length, from the end of each bar, effectively ignoring the steel up until it is fully developed. The other response was that the department has no special procedure for bar cutoffs and ignores them in load rating. According to the survey, TxDOT does not consider bar cutoffs in load rating.

Furthermore, recent TxDOT research by Mander et al. (2011a, 2011b) tested reinforced concrete bridge decks and showed the load capacity was well in excess of the design ultimate strength—typically some 100 to 200 percent higher than the specified ultimate strength. This reserve capacity is attributed to the two-way action of the slab system that is not normally accurately accounted for by design. Accounting for the two-way action of the slab could also be used to improve the RFs of slab bridges when deemed necessary.

2.6.4 Rating Method

As was examined earlier in this report, the use of different load rating methods can produce vastly different RFs for the same bridge. It may be possible that using a load rating method for certain bridge types or for bridges designed using a particular method will produce more accurate or higher RFs.

Mertz (2005) noted in his NCHRP report *Load Rating by Load and Resistance Factor Evaluation Method* that for design load Inventory Ratings, the LRFR method tended to produce higher RFs than the LFR method, which was also the case for continuous slab bridges and for most steel plate girder bridges. Mertz also determined that the LRFR method produced a more pronounced correlation than the LFR method when the reliability indices of a group of bridges were compared to the design load inventory level RFs.

Mlynarski et al. (2011) found, as noted in their NCHRP report *Comparison of AASHTO Bridge Load Rating Methods*, that the LFR method tended to produce higher ratings than the LRFR methods. However, they also determined that certain characteristics, such as bearing capacity and shear capacity of longitudinal reinforcement near the ends of girders, were considered by the LRFR method, while the LFR method did not. In conclusion, using a particular rating method in some scenarios may produce more precise RFs. This will be examined in more depth during Task 3 of this project.

2.6.5 Live Load Factors

Reducing live load factors to obtain certain reliability indices or to better represent the types of trucks that are actually on the road could improve bridge load ratings. As previously stated, NCHRP Project 12-78 found the current live load factors in the AASHTO MBE to be overly conservative for the targeted reliability. Mlynarski et al. (2011) also proposed new live load factors based on their finding that the current AASHTO live load factors represented a reliability index higher than intended. After reducing the live load factors to correctly provide the desired reliability index, they found that the RFs of a database of bridges increased. The live load factors are applied to the live load effect, which is in the denominator of the LRFR RF equation (Equation 2.1). Reducing the denominator will increase the RF. Although this approach is not within the scope of this project, it may be an area to explore in the future.

2.6.6 Material Properties

As shown by multiple reports and studies presented earlier, many historic steel members exhibit higher yield and ultimate strengths than the nominal value used in design. Laboratory testing of coupons from a number of steel bridges and review of mill test data could lead to a pattern of improved steel strengths for older bridges over values that the AASHTO MBE provides. Even testing of fairly young structures could lead to new insights about the actual strength of steel members used in construction. Where these increases are permissible, it would improve the capacity of these structures, which would lead to higher RFs. While concrete bridges deficient in flexure strength may not be affected by increased compressive strength of concrete, both steel and concrete structures could benefit from updating of reinforcing steel and structural steel material properties.

2.6.7 Refined Analysis

Load ratings can be increased for some bridges through refined analysis methods such as FEM. Current analysis methods given in AASHTO's manuals are simplified procedures that are meant to be fairly quick to complete and straightforward to understand. However, this means that they incorporate simplifying assumptions that reduce the accuracy of the results, usually erring on the side of conservatism. FEM does not make the same simplifying assumptions that AASHTO does and is therefore designed to be more representative of the actual, in-situ condition of the bridge.

As a result, the outcomes of loading a finite element model of a bridge should be more accurate in terms of how the bridge will actually react under the same real-life loading. That being said, finite element models take a considerable amount of time to develop and can be inaccurate if incorrect modeling parameters are used. Analyzing every bridge in an inventory would be an onerous task. However, if a number of typical bridges were analyzed using finite element models, a practice could possibly be developed for bridges of one type based on representative models. If these changes increased capacity or decreased the load effect, RFs for a group of bridges could be increased.

3 ANALYSIS OF LOAD-POSTED BRIDGE INVENTORY

According to the NBI (2016) database, the state of Texas has 2111 bridges posted at load levels below the legal limit. Posted bridges come in various shapes and sizes, were built in different eras and environments, and exhibit vastly different structural behaviors. All load-posted bridges in Texas were reviewed in terms of bridge type, age, number of spans, maximum span length, bridge width, and condition ratings. This review was used as the basis to select typical bridges that represent the populations of load-posted bridges found to be most prevalent and most significant with respect to increasing operating loads. The following subsections describe the terminology used for condition ratings and the overall distribution of Texas load-posted bridges by various parameters considered.

3.1 GENERAL

One characteristic of a bridge is its location in regard to a highway system. *On-system* bridges are part of a designated highway system and maintained by TxDOT, while *off-system* bridges are not located on a highway system and are under the jurisdiction of a local government, such as a county or city. The load-posted bridge inventory in Texas was categorized by location as *on* or *off* the state highway system. Another important characteristic that could help in categorizing and understanding load-posted bridges is condition classification. FHWA has two main classifications for the condition rating of bridges: (1) *structurally deficient* (SD), and (2) *functionally obsolete* (FO). FHWA classifies a bridge as SD if it has severe deterioration or extreme reduction in load capacity. An FO bridge does not satisfy the original design criteria in terms of load-carrying capacity, deck geometry, clearance, or approach roadway alignment. TxDOT has a third condition classification: *sub-standard for load only*. Bridges that are not SD or FO but have a load capacity below the legal limit are placed in the SSLO category. In other words, SSLO bridges are load posted but not classified as SD or FO according to FHWA criteria. Load posting and sufficiency status (SD, FO, or not deficient) for all bridges are provided in the NBI. This research project is focused on load-posted bridges; therefore, only the bridges that are classified as load posted under Item 41 (structure open, posted, or closed) of the NBI were extracted from the database. Table 3.1 summarizes the number of load-posted bridges in Texas as a function of condition status and location with respect to the highway system using the 2016 NBI database (NBI 2016).

Table 3.1. Texas Load-Posted Bridges by Condition Status

Condition Classification	On-System	Off-System	Total
Structurally Deficient (SD)	39	473	512
Functionally Obsolete (FO)	58	572	630
Sub-standard for Load Only (SSLO)	78	891	969
Total	175	1936	2111

Past research has shown that standard load rating methods may use conservative material properties, load distribution factors, and load rating methods in some cases. Input from TxDOT indicated that the focus should be on SSLO bridges given that the only issue for these bridges is related to load posting. Removing load postings for some of the 969 SSLO bridges in Texas may be possible given more accurate information on the current bridge condition. Refined analysis and rating methods together with more accurate material properties can better describe the actual reserve capacity of existing bridges, which allows for safe and appropriate decisions to be made with respect to removal of load postings.

3.2 LOAD-POSTED AND SSLO BRIDGES BY KIND OF MATERIAL/DESIGN

According to the 2016 NBI database, there are 2111 Texas bridges that are load posted. Figure 3.1 shows the geographic location of all load-posted bridges in Texas. Most of these structures are in good condition but are rated below the legal load limit. Of these load-posted bridges, 630 are rated FO, and another 512 are listed as SD. A total of 969 bridges are SSLO, which indicates that they are not structurally deficient but require load posting based on current legal load limits provided in the AASHTO MBE (AASHTO 2018). Figure 3.2 shows the geographic locations of the SSLO bridges. The load-posted and SSLO bridge inventory were first evaluated based on two major characteristics: (1) kind of material and/or design, and (2) type of design and/or construction.

Figure 3.3 shows the distribution of all load-posted bridges and all SSLO bridges in Texas by type of material/design using a Pareto histogram. In these Pareto charts, the total number of bridges for each category are shown as descending bar charts, and the cumulative total is presented by the line plot (shown with a red line). Load-posted and SSLO bridges have the same descending order when ranked by population based on type of material. The six bridge types with the highest population (steel, concrete, wood, steel continuous, prestressed, and concrete continuous) constitute approximately 99 percent of all load-posted or SSLO bridges in Texas. Therefore, only

these bridge types were evaluated further in terms of design and/or construction type. Maps showing the geographic locations of the SSLO bridges for each of these six bridge types are provided in Appendix A (Figure A-1 thru Figure A-6).

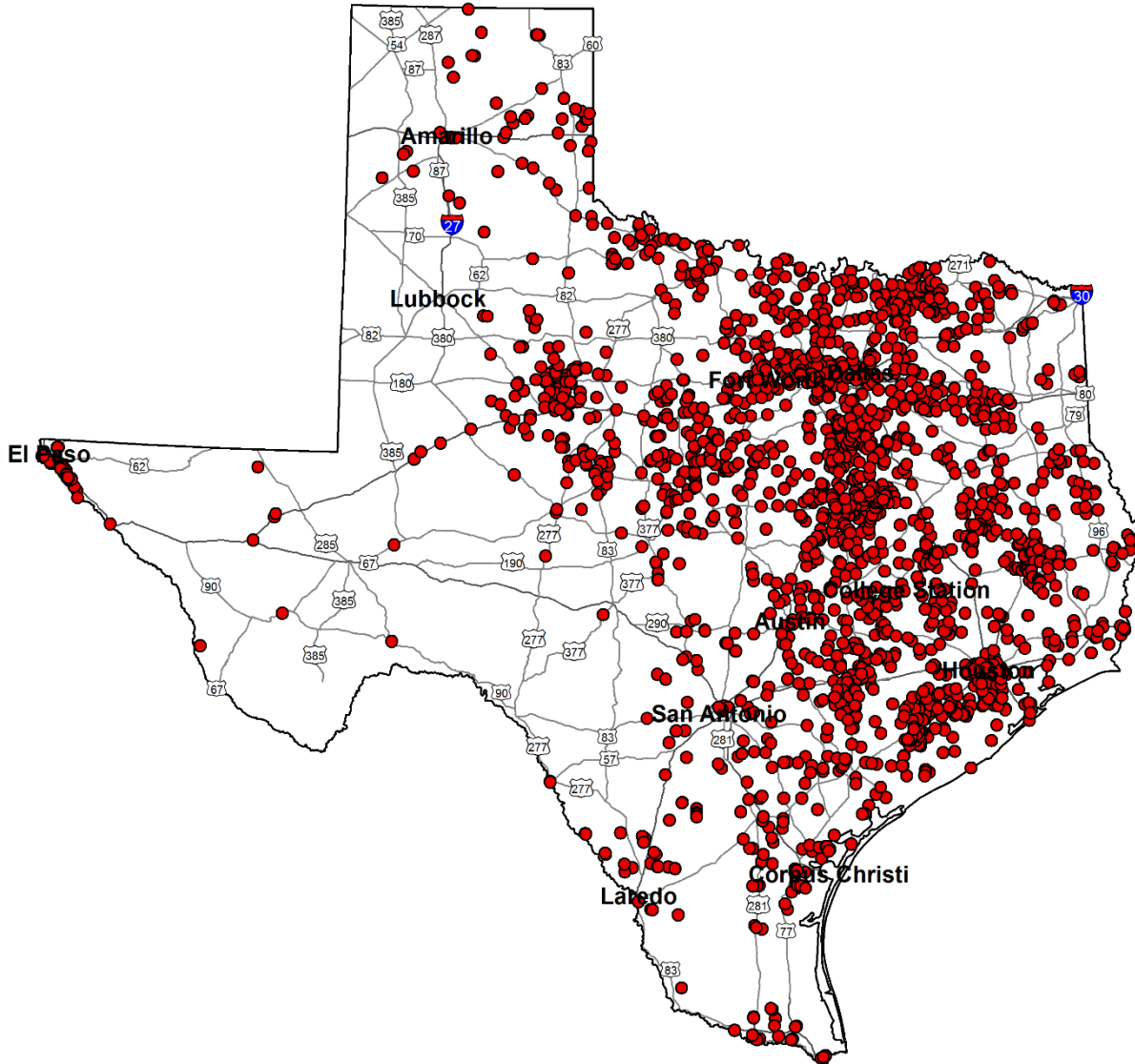


Figure 3.1. Map of All Load-Posted Bridges in Texas (2111 Bridges)

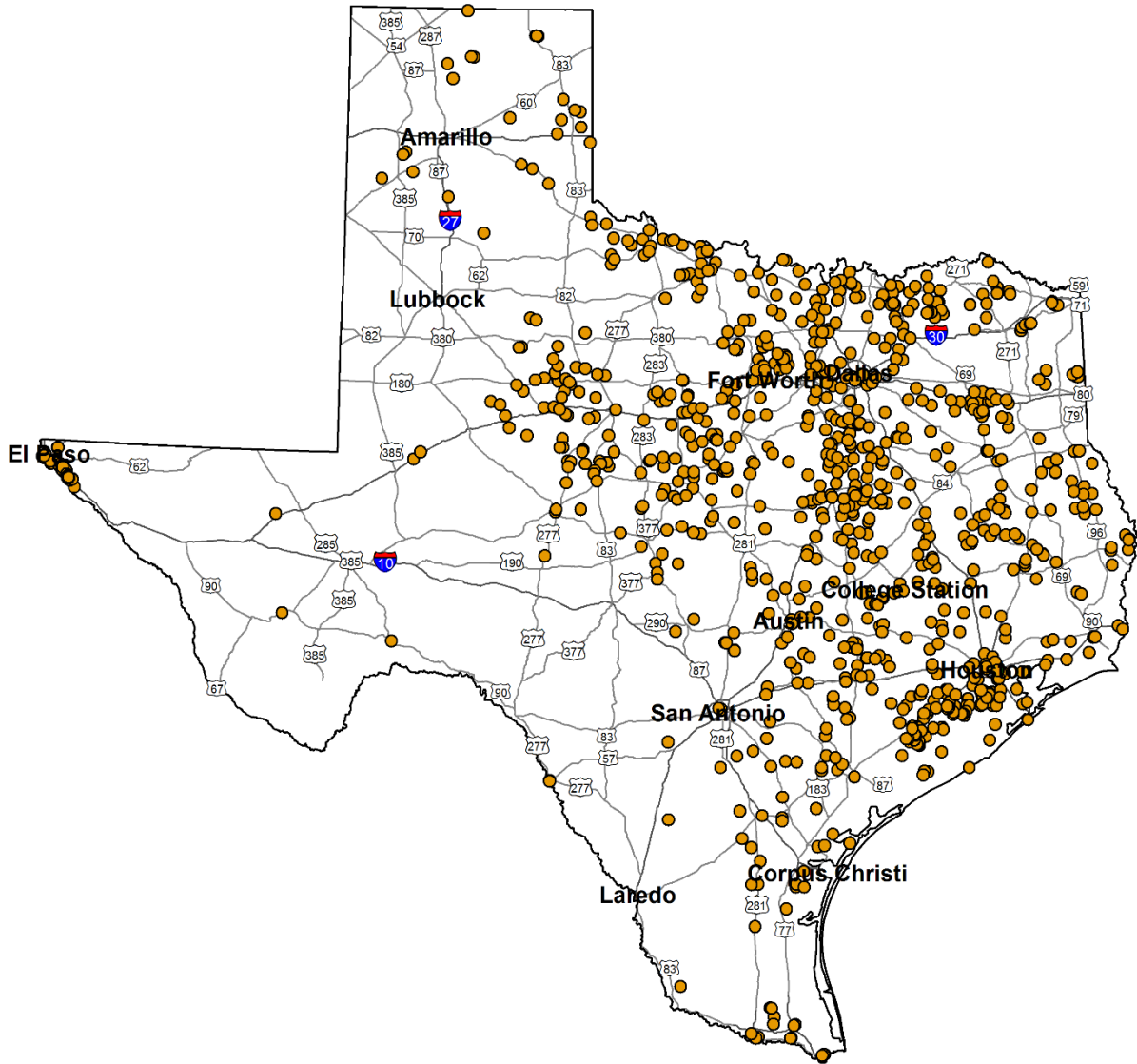
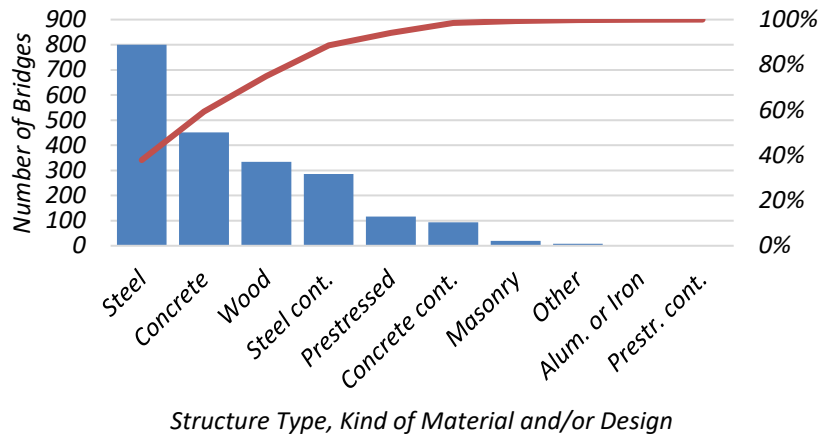
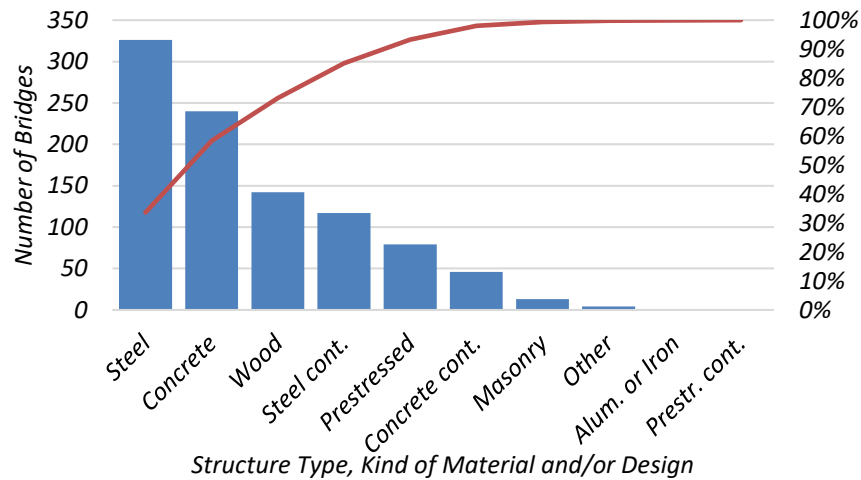


Figure 3.2. Map of All SSLO Bridges in Texas (969 Bridges)



(a) All load-posted bridges



(b) SSLO bridges

Figure 3.3. Distribution of Texas Load-Posted Bridges by Structure Type—Kind of Material and/or Design

3.3 LOAD-POSTED AND SSLO BRIDGES BY TYPE OF DESIGN AND/OR CONSTRUCTION

An initial review of the Texas load-posted bridge inventory revealed that the majority of these bridges are steel, concrete, wood, steel continuous, prestressed, and concrete continuous bridges. Each one of these six bridge categories was further evaluated based on type of design and/or construction. Figure 3.4 shows the distribution of the six bridge types by different

design/construction structure types. These distributions are plotted for both load-posted bridges and SSLO bridges for comparison.

This evaluation indicates that the distribution of load-posted and SSLO bridges follow the same pattern for all six bridge types. For example, Figure 3.4(a) and (b) show the distribution of load-posted and SSLO simple-span steel bridges by type of design and/or construction. Steel bridges are the most populated bridge category by material kind, and 74 percent of load-posted steel bridges and 79 percent of SSLO steel bridges are stringer/multi-girder bridges. Figure 3.4(c) and (d) illustrate that the majority of all simple-span load-posted or SSLO concrete bridges are slab, culvert, or stringer/multi-girder bridges. Approximately 45 percent of all load-posted concrete bridges are slab bridges, and 15 percent are multi-girder bridges. While almost 30 percent of load-posted concrete bridges are culverts, this type is not considered further in this project because culvert bridges were the subject of a recently completed TxDOT project (Wood et al. 2010). Multi-girder wood bridges are the third most populated kind of bridge, comprising 15 percent of all load-posted or SSLO bridges. However, wood bridges have varying characteristics and are not currently being considered for further study as part of this project. Therefore, steel continuous bridges, which comprise 12 percent of all load-posted bridges, were selected for further detailed evaluation. Figure 3.4(g) and (h) show the distribution of load-posted steel continuous bridges by type of design and/or construction; the majority of these bridges are stringer/multi-girder bridges including 85 percent of all load-posted and 93 percent of all SSLO steel continuous bridges. Additional bridge types include prestressed bridges, which constitute only 8 percent (79 bridges), and concrete continuous bridges, which constitute less than 5 percent of all SSLO bridges. Therefore, they are not included in the final selected list of bridges. Figure 3.4(i) and (j) show that 86 percent (68 bridges) of all SSLO prestressed bridges are categorized as “other.” Figure 3.4(k) and (l) illustrate that 42 out of 46 concrete continuous bridges are slab bridges.

In summary, the four most populated bridge types were identified, excluding wood bridges and culverts, for further evaluation in terms of various characteristics that include age, maximum span length, bridge length, deck width, operating rating, and posting evaluation.

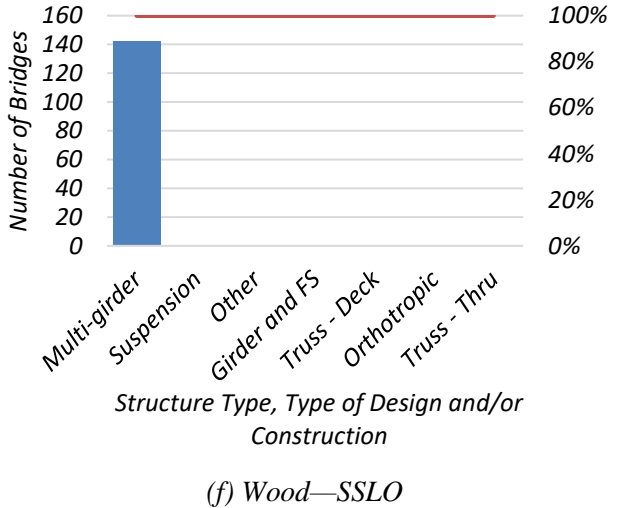
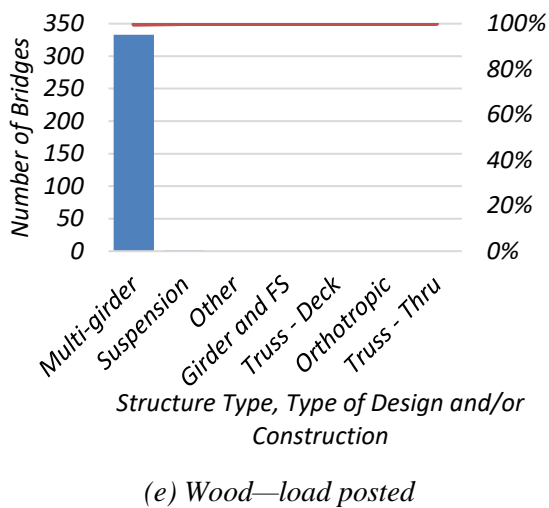
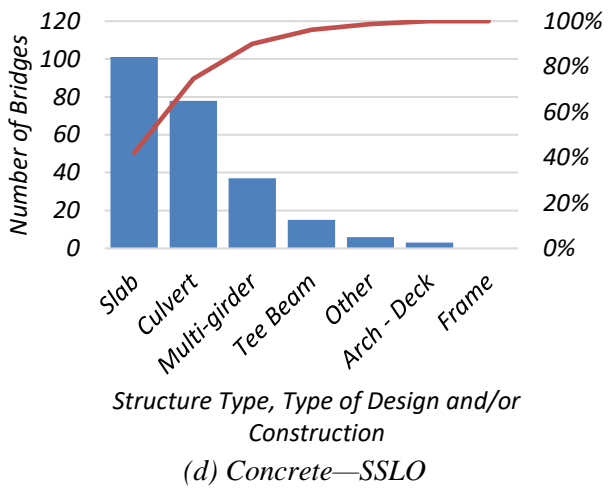
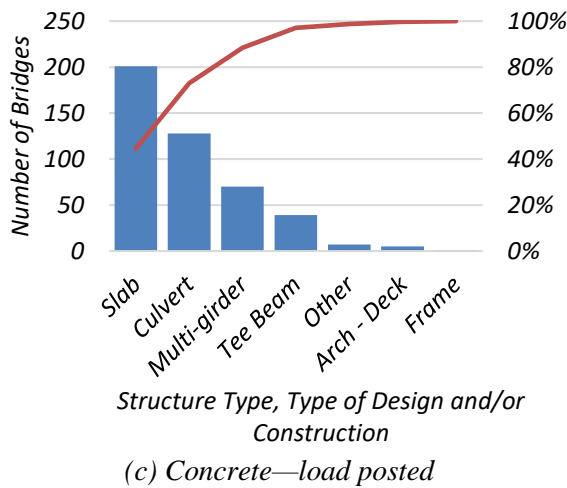
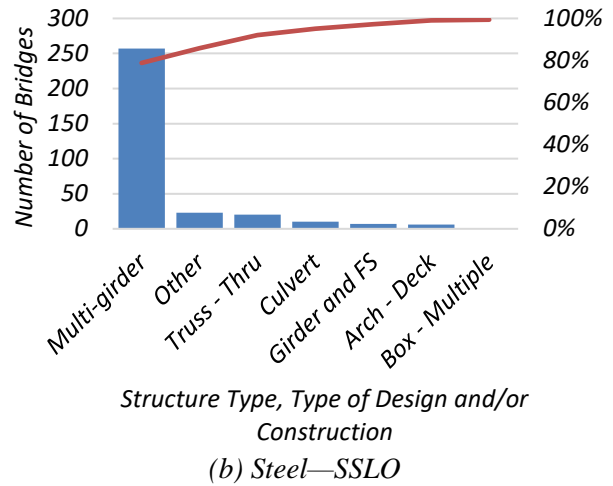
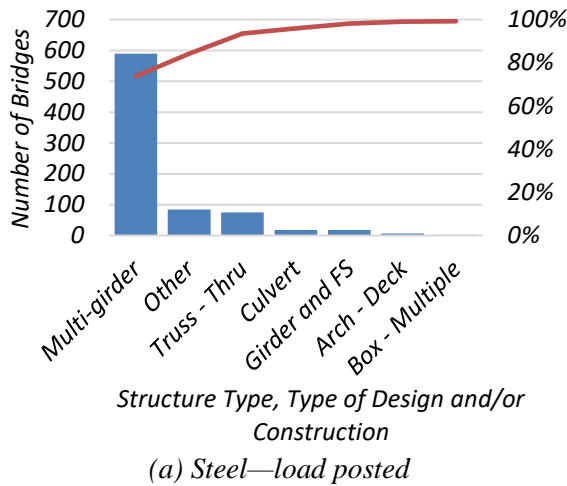
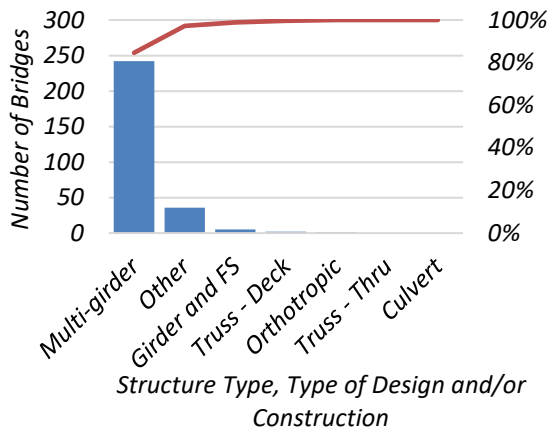
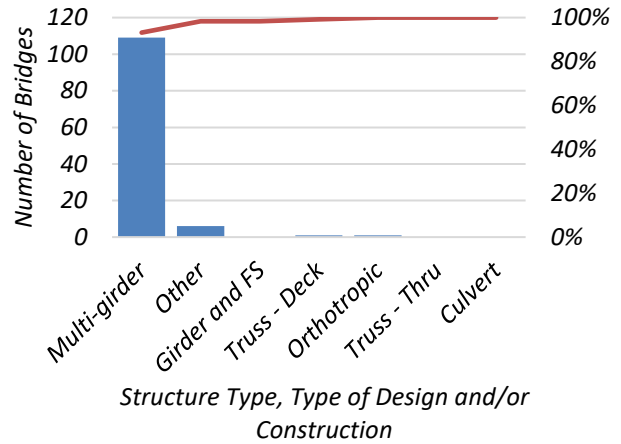


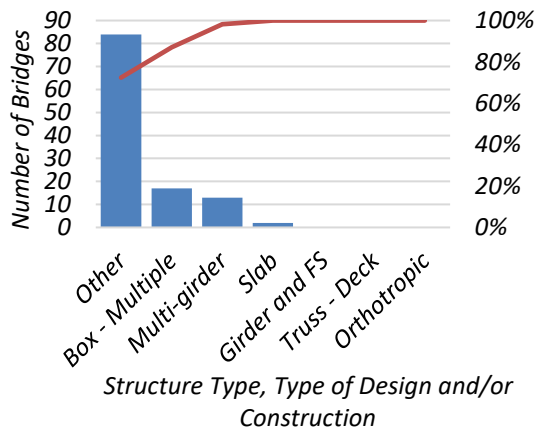
Figure 3.4. Distribution of Texas Load-Posted and SSLO Bridges by Structure Type—Type of Design and/or Construction



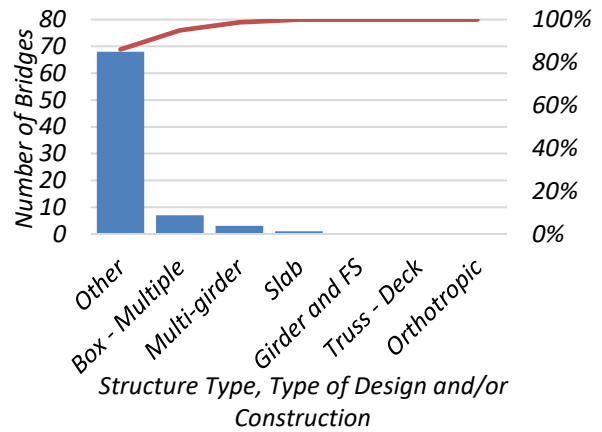
(g) Steel continuous—load posted



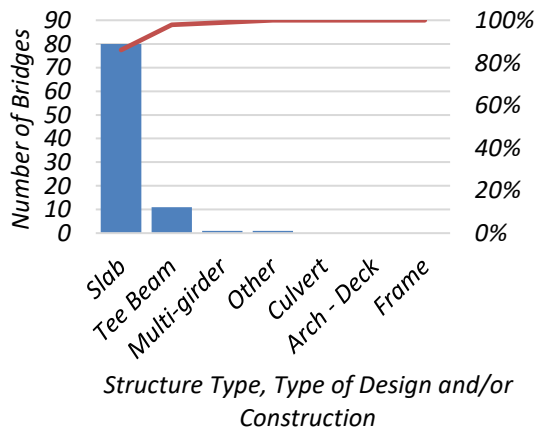
(h) Steel continuous—SSLO



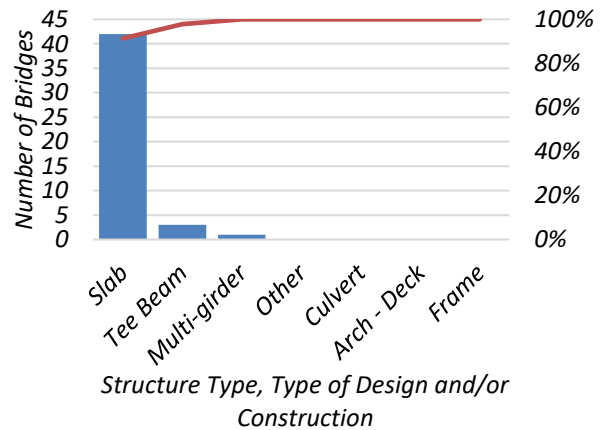
(i) Prestressed—load posted



(j) Prestressed—SSLO



(k) Concrete continuous—load posted



(l) Concrete continuous—SSLO

Figure 3.4. Distribution of Texas Load-Posted and SSLO Bridges by Structure Type—Type of Design and/or Construction (Continued)

3.4 MAJOR CHARACTERISTICS OF SELECTED TYPICAL BRIDGE TYPES

3.4.1 General

As noted in Section 3.3, certain types of bridges were not considered further because they are not likely to be included in this research project based on an initial review of the inventory, TxDOT suggestions, and overlaps with other TxDOT projects. Therefore, wood bridges and culverts are not included for further detailed evaluation in terms of other geometric and condition characteristics. Also, it is more likely that load postings can be removed for bridges that are not structurally deficient or functionally obsolete. Therefore, only SSLO bridges were further evaluated in terms of various detailed characteristics, including age, maximum span length, total length, deck width, operating rating, and posting status.

Table 3.2 lists the number of SSLO bridges in Texas by bridge type and is differentiated by on-system and off-system bridges based on the 2016 NBI database. Steel, concrete, and steel continuous bridges comprise 70 percent of all SSLO bridges in Texas. Stringer/multi-girder bridges make up 80 percent (257 bridges) and 93 percent (109 bridges) of SSLO steel and steel continuous bridges, respectively. Slab bridges and multi-girder bridges comprise 42 percent (101 bridges) and 15 percent (37 bridges) of all SSLO concrete bridges, respectively. These four bridge types (steel multi-girder, concrete slab, concrete multi-girder, and continuous steel multi-girder SSLO bridges) were identified for additional consideration for the basic load rating, refined analysis, and field-testing tasks of the current project. In addition, there are a total of 68 prestressed SSLO bridges that comprise approximately 6 percent of all SSLO bridges and are classified as “other.” Additional information regarding structural design and construction of these bridges are reviewed later in this report.

It is not practical to conduct basic load rating analysis for all the bridges in each of the four bridge types noted above. The initial selection of bridges described in Technical Memorandum 2 was done by constrained sampling that considers average age, span length, deck width, and posting status. The research team further refined this approach because selecting bridges based on average values of primary parameters (age, span length, deck width, and operating rating) may not be ideal to obtain an adequate subset of bridges to provide a representative sampling of all bridges in each bridge type category. Therefore, the selected bridges for each one of these four bridge types were updated to better represent the full distribution of parameters. This approach uses simple random

sampling (SRS) (Pruim 2011; Thompson 2012) in which each bridge of the subset has an equal probability of being chosen. This process allows selection of an unbiased representation of a dataset while keeping a similar distribution for the primary parameters. The following subsections provide a detailed evaluation of key characteristics for the four selected bridge types.

3.4.2 SSLO Simple-Span Steel Multi-Girder Bridges

Among the SSLO bridges, steel multi-girder bridges are the highest in number. There are 14 on-system and 243 off-system SSLO steel multi-girder bridges in Texas. They were built in different eras and have varying geometries. Figure 3.5(a, c, e, g) shows the distribution of steel multi-girder bridges by age, various geometric properties, and condition status for all SSLO steel multi-girder bridges. About 40 percent of steel multi-girder SSLO bridges were built between 1980 and 2000. Their average construction year is 1974, with a standard deviation of 24 years. The majority of SSLO steel multi-girder bridges are short-span bridges having a span length of less than 50 ft. The average span length is 36 ft, with a standard deviation of 12 ft. The majority of the bridges have either one or two lanes. They have an average deck width of 20.4 ft, with a standard deviation of 7.5 ft. Figure 3.5(g) shows that 197 bridges, 77 percent of all steel multi-girder SSLO bridges, have an operating load rating less than the legal load limit.

Although all SSLO bridges are load posted by definition and these bridges are classified as “posted” based on Item 41 (structure open, posted, or closed) of the NBI, there are 60 steel multi-girder SSLO bridges that have an operating rating equal to or above the maximum legal load per Item 70 (Bridge Posting) of the NBI. The reason for this discrepancy is explained in the recording and coding guide (FHWA 1995) as a difference in coding practice between the highway agencies. “Although posting a bridge for load-carrying capacity is required only when the maximum legal load exceeds the operating rating, highway agencies may choose to post at a lower level. This posting practice may appear to produce conflicting coding when Item 41 - Structure Open, Posted or Closed to Traffic is coded to show the bridge as actually posted at the site and Item 70 - Bridge Posting is coded as bridge posting is not required. Since different criteria are used for coding these items, this coding is acceptable and correct when the highway agency elects to post at less than the operating rating.”

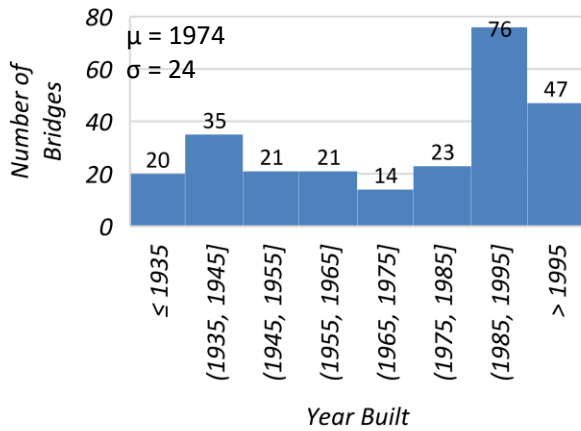
Table 3.2. Texas SSLO Bridges by Bridge Type

Material/Design	Bridge Type	No. of Bridges		Total
		On-system	Off-system	
Steel	Stringer/Multi-beam or Girder	14	243	257
	Girder and Floor beam System	0	7	7
	Box Beam or Girders—Multiple	0	1	1
	Box Beam or Girders—Single or Spread	0	1	1
	Truss-Thru	0	20	20
	Arch-Deck	0	6	6
	Movable-Swing	1	0	1
	Culvert	0	10	10
	Other	1	22	23
Concrete	Slab	42	59	101
	Stringer/Multi-beam or Girder	2	35	37
	Tee Beam	3	12	15
	Arch-Deck	0	3	3
	Culvert	2	76	78
	Other	0	6	6
Wood	Stringer/Multi-beam or Girder	0	142	142
Steel Continuous	Stringer/Multi-beam or Girder	6	103	109
	Orthotropic	0	1	1
	Truss-Deck	0	1	1
	Other	0	6	6
Prestressed	Stringer/Multi-beam or Girder	1	2	3
	Box Beam or Girders—Multiple	1	6	7
	Slab	0	1	1
	Other	0	68	68
Concrete Continuous	Slab	4	38	42
	Stringer/Multi-beam or Girder	0	1	1
	Tee Beam	0	3	3
Masonry	Arch-Deck	0	3	3
	Culvert	1	9	10
Other	Other	0	4	4
Alum. or Iron	Other	0	1	1
Prestressed Cont.	Stringer/Multi-beam or Girder	0	1	1
Total		78	891	969

Figure 3.5(b, d, f, h) shows the distribution of the primary variables for the selected subset of SSLO steel multi-girder bridges in Texas. After primary selection of 30 bridges using SRS, an additional 10 bridges were included in the selected subset based on their proximity to College

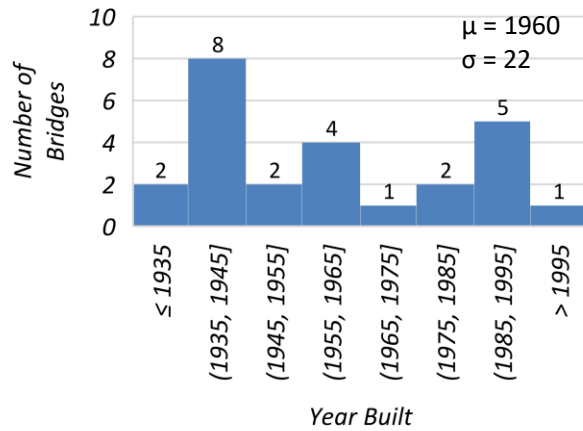
Station, Texas, because it would be more practical and economical to select a bridge in this area for field testing. It was determined later, when reviewing inspection records, that this initial subset of bridges included steel multi-girder bridges having concrete, wood, and steel channel decks, while the project primarily focuses on on-system bridges with concrete decks. Only 18 bridges from the initial selection have reinforced concrete decks. Therefore, an additional seven on-system steel multi-girder bridges with concrete decks have been included in the set of selected bridges. The histograms of the primary variables for the selected subset of steel multi-girder bridges include all 25 selected bridges. The comparison of the histograms indicates that the selected subset is a good representation of the original set of all SSLO steel multi-girder bridges in terms of the distribution of primary variables. One of these 25 bridges is proposed as a typical steel multi-girder bridge to be further studied as part of the refined analysis and field-testing tasks; details of that bridge are provided in Chapter 5.

Full dataset of 257 SSLO steel multi-girder bridges

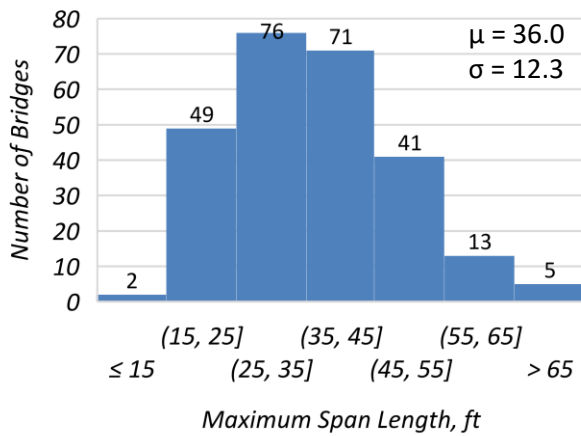


(a) Year built—All 257 bridges

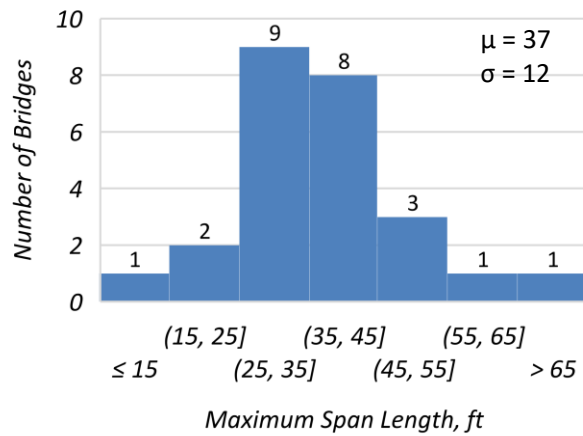
Subset of 25 SSLO steel multi-girder bridges



(b) Year built—Selected 25 bridges



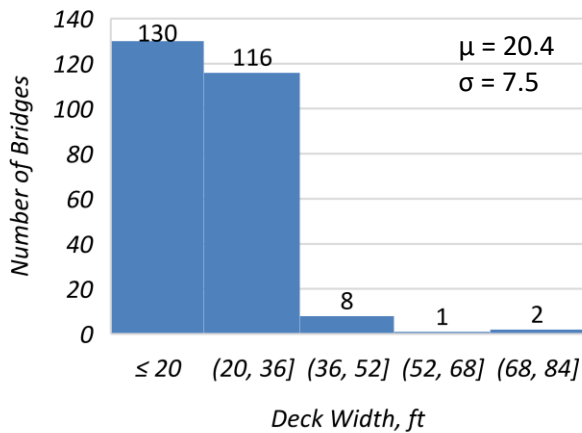
(c) Maximum span length—All 257 bridges



(d) Maximum span length—Selected 25 bridges

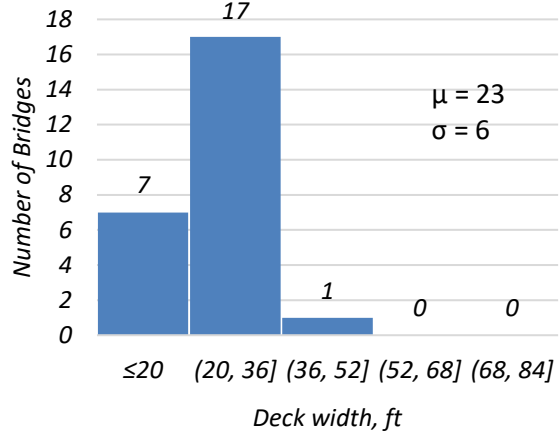
Figure 3.5. Distribution of Bridge Parameters for SSLO Steel Multi-Girder Bridges

Full dataset of 257 SSLO steel multi-girder bridges

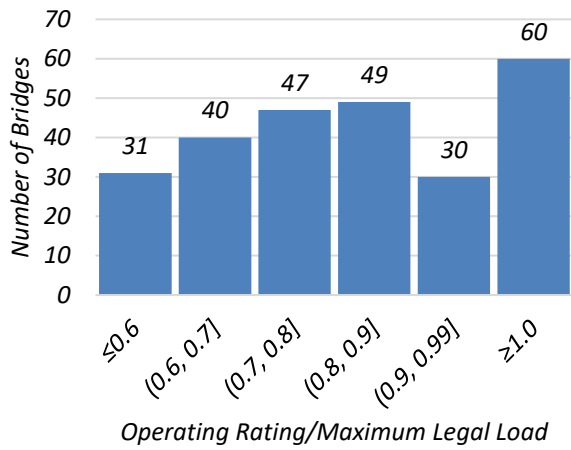


(e) Deck width—All 257 bridges

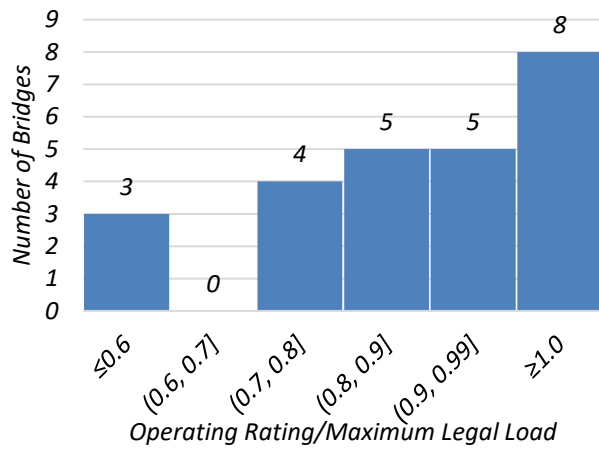
Subset of 25 SSLO steel multi-girder bridges



(f) Deck width—Selected 25 bridges



(g) Posting evaluation—All 257 bridges



(h) Posting evaluation—Selected 25 bridges

Figure 3.5. Distribution of Bridge Parameters for SSLO Steel Multi-Girder Bridges (Continued)

Figure 3.6 shows the geographic locations of all 257 SSLO steel multi-girder bridges (yellow placemark) and the selected 25 SSLO steel multi-girder bridges (orange placemark) in Texas. Table 3.3 lists these selected SSLO steel multi-girder bridges, along with their primary geometric properties, condition and operating ratings, average daily traffic (ADT), and route information.

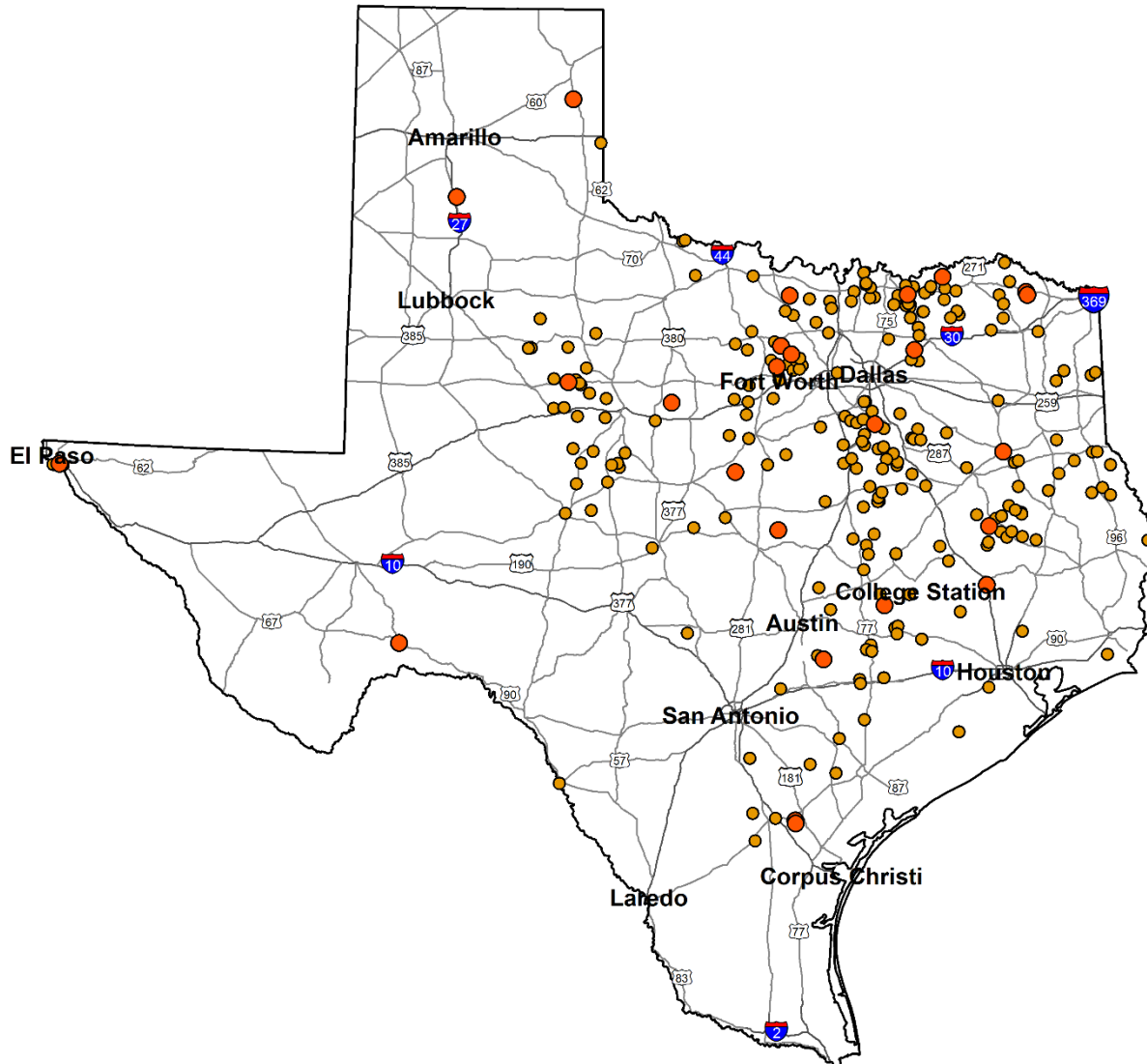


Figure 3.6. Location Map of SSLO Steel Multi-Girder Bridges

*Note: Yellow Placemark = Full dataset of all SSLO steel multi-girder bridges (257 bridges)
 Orange Placemark = Selected subset of SSLO steel multi-girder bridges (25 bridges)*

Table 3.3. Selected SSLO Steel Multi-Girder Bridges

ID	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
						Deck	Super-structure	Sub-structure		
SM-1	3	1931	290	39	25	6	6	7	0.55	0
SM-2	3	1931	290	39	25	6	6	6	0.55	0
SM-3	3	1938	450	30	25	6	6	6	0.80	4
SM-4	3	1938	450	30	25	6	6	7	0.55	0
SM-5	3	1938	300	41	24	7	6	7	0.77	2
SM-6	4	1940	50	24	22	6	6	7	0.99	5

SM-7	3	1940	340	30	23	6	5	5	0.94	4
SM-8	3	1940	170	69	27	6	6	6	0.72	2
SM-9	4	1941	50	40	16	7	6	6	0.99	5
SM-10	4	1945	100	29	22	5	5	6	1.18	5
SM-11	2	1947	1230	40	46	6	6	5	1.21	5
SM-12	4	1955	247	30	22	6	6	5	0.85	3
SM-13	5	1958	590	14	26	5	5	6	0.99	5
SM-14	4	1960	80	38	16	7	6	7	0.74	2
SM-15	3	1961	1150	43	26	7	6	6	0.83	3
SM-16	5	1965	730	36	30	7	6	7	1.21	5
SM-17	4	1970	70	30	16	7	5	5	0.88	3
SM-18	4	1977	100	33	15	6	5	7	0.94	4
SM-19	4	1982	20	21	14	7	6	5	0.72	2
SM-20	4	1986	330	54	24	6	6	5	0.91	4
SM-21	4	1990	550	53	25	8	7	7	0.99	5
SM-22	4	1990	250	55	24	7	6	6	0.85	3
SM-23	4	1992	60	48	17	8	6	6	0.88	3
SM-24	4	1995	50	29	14	8	5	5	0.96	4
SM-25	4	2000	10	28	21	7	6	5	0.99	5

Note: ADT = Average daily traffic.

Route Prefix (NBI Item 5B)

- 1 = Interstate highway
- 2 = US numbered highway
- 3 = State highway
- 4 = County highway
- 5 = City street
- 6 = Federal lands road
- 7 = State lands road
- 8 = Other

Condition Ratings (NBI item 58–62)

- 9 = Excellent
- 8 = Very good
- 7 = Good
- 6 = Satisfactory
- 5 = Fair
- 4 = Poor
- 3 = Series
- 2 = Critical
- 1 = Imminent Failure
- 0 = Failed

Posting evaluation (NBI Item 70)

- 5 = $\Omega \geq 1.0$
 - 4 = $0.99 \geq \Omega > 0.9$
 - 3 = $0.9 \geq \Omega > 0.8$
 - 2 = $0.8 \geq \Omega > 0.7$
 - 1 = $0.7 \geq \Omega > 0.6$
 - 0 = $0.6 \geq \Omega$
- Ω = operating rating/maximum legal load

3.4.3 SSLO Continuous Steel Multi-Girder Bridges

There are total of 117 SSLO continuous steel bridges, which constitute 12 percent of all SSLO bridges in Texas. Among these bridges, the majority (109 bridges) are multi-girder bridges, including six on-system and 103 off-system bridges. Figure 3.7(a, c, e, g) shows the distribution of various bridge characteristics for SSLO continuous steel multi-girder bridges in Texas. The majority of these bridges, 82 percent, have short spans between 10 and 30 ft. Figure 3.7(e) indicates that approximately 53 percent of these bridges have one lane with a total deck width less than 20 ft. Although 30 percent of all SSLO continuous steel multi-girder bridges were built between 1930

and 1950, the age distribution of the remaining bridges is almost uniform until 2010. The average construction year is 1962, with a standard deviation of 26 years.

Figure 3.7(b, d, f, h) shows the distribution of the primary variables for the selected subset (16 bridges) of continuous steel multi-girder bridges in Texas. The initial random sampling was conducted for 20 bridges, and then five more bridges were included in the selected subset based on their proximity to College Station. After reviewing inspection records, it was found that only 12 bridges in this initial selection have a concrete deck. Therefore, an additional four on-system continuous steel multi-girder bridges with concrete decks were selected. The comparison of the histograms indicates that the selected subset is a reasonable representation of the original set of continuous steel multi-girder bridges in terms of the distribution of primary variables.

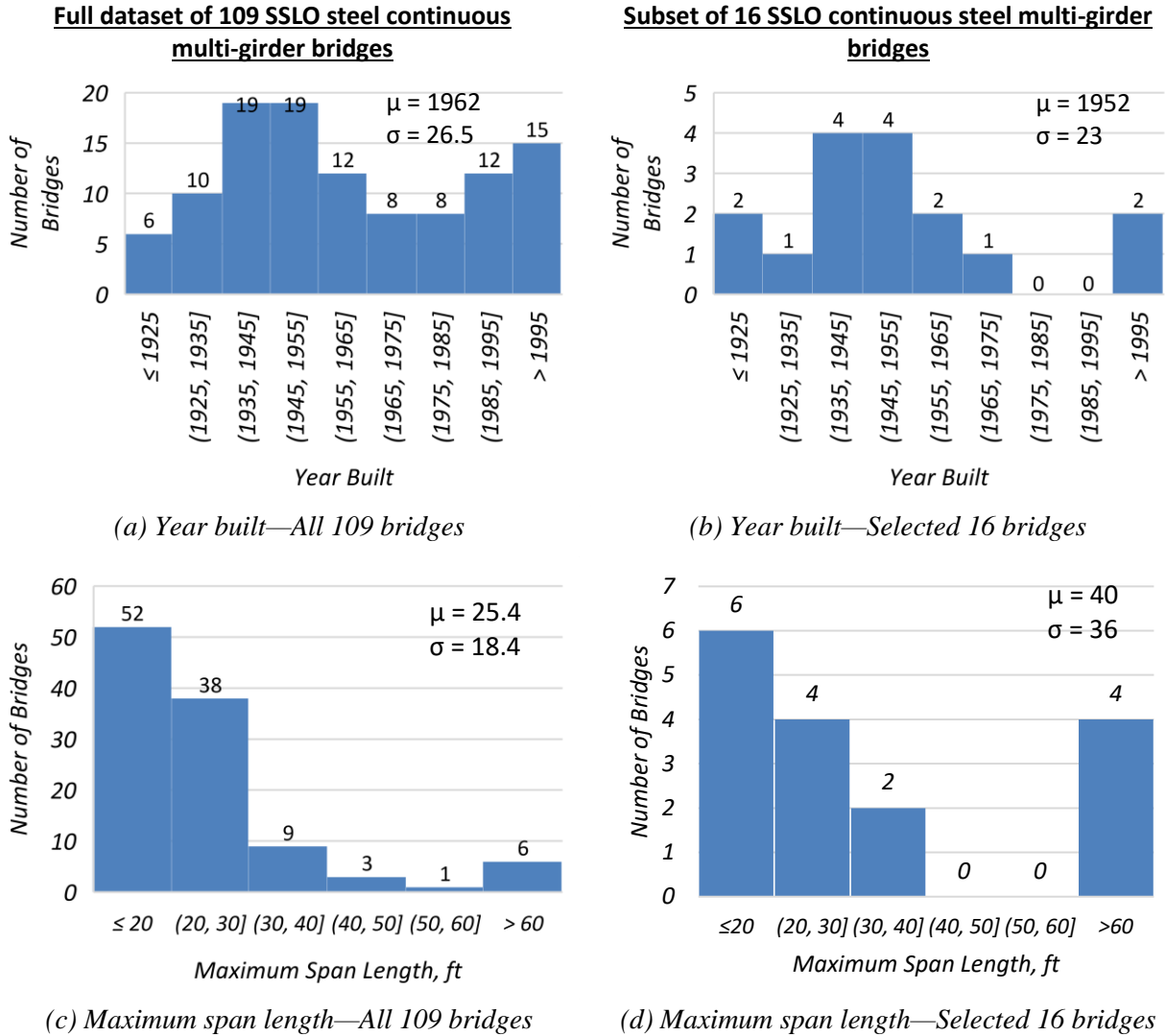


Figure 3.7. Distribution of Bridge Parameters for SSLO Continuous Steel Multi-Girder Bridges

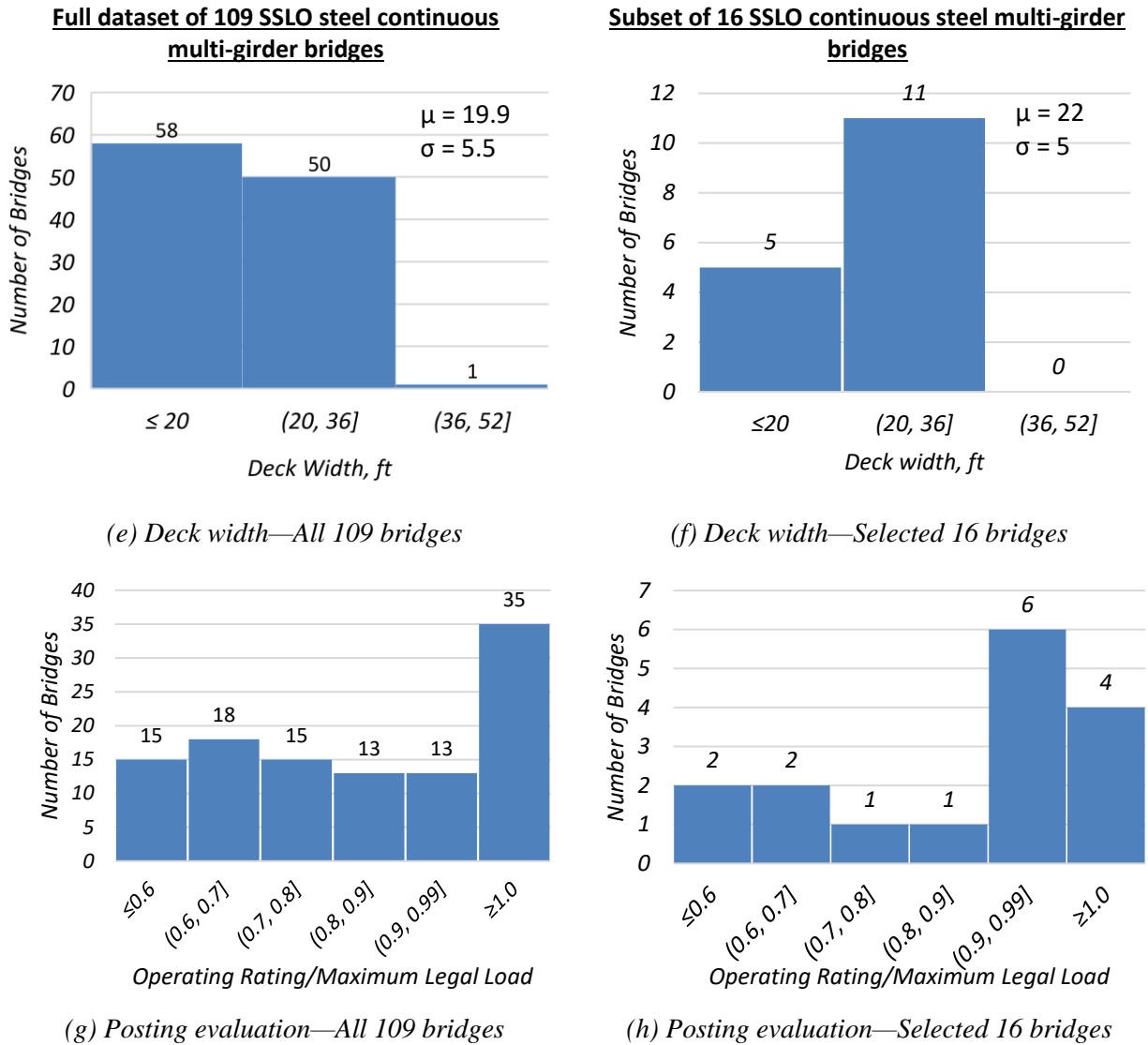


Figure 3.7. Distribution of Bridge Parameters for SSLO Continuous Steel Multi-Girder Bridges (Continued)

Figure 3.8 shows the geographic locations of all 109 SSLO continuous steel multi-girder bridges (yellow placemark) and the selected 16 SSLO steel continuous multi-girder bridges (orange placemark) in Texas. Table 3.4 lists selected SSLO continuous steel multi-girder bridges, along

with their primary geometric properties, condition and operating ratings, ADT, and route information.

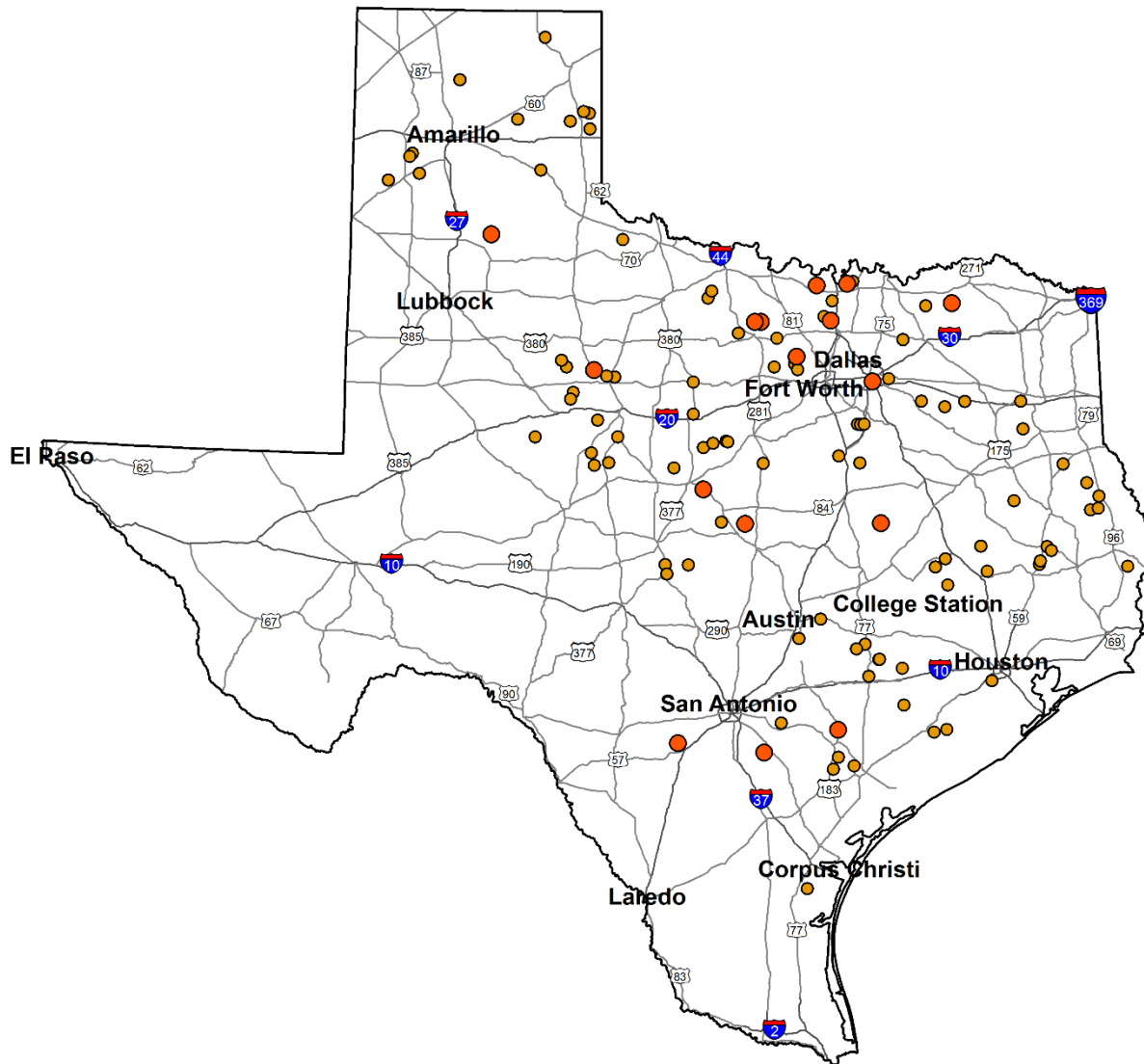


Figure 3.8. Location Map of SSLO Continuous Steel Multi-Girder Bridges

Note: Yellow Placemark = Full dataset of all SSLO steel continuous multi-girder bridges (109 bridges)
 Orange Placemark = Selected subset of SSLO steel continuous multi-girder bridges (16 bridges)

Table 3.4. Selected SSLO Continuous Steel Multi-Girder Bridges

ID	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
						Deck	Super-structure	Sub-structure		
SC-1	4	1910	40	19	19	6	5	6	0.55	0
SC-2	4	1925	10	26	14	5	6	5	0.72	2
SC-3	4	1935	50	18	18	5	6	7	1.05	5

SC-4	4	1937	50	18	18	5	6	6	1.02	4
SC-5	4	1940	50	21	23	5	6	6	0.88	3
SC-6	4	1942	350	23	24	5	6	5	1.07	5
SC-7	4	1943	180	12	24	7	6	5	0.99	5
SC-8	3	1949	230	70	26	7	6	5	0.91	4
SC-9	4	1950	60	11	21	6	6	6	0.91	4
SC-10	3	1950	7300	152	34	6	5	6	0.94	5
SC-11	4	1955	60	19	21	6	5	6	0.61	1
SC-12	3	1959	260	75	26	6	7	7	0.88	4
SC-13	3	1965	1160	75	28	6	6	6	0.96	4
SC-14	4	1970	80	36	14	6	6	5	0.61	1
SC-15	4	1997	60	24	24	7	7	6	0.91	4
SC-16	4	1999	50	39	20	7	6	6	0.55	0

ADT = Average daily traffic.

<u>Route Prefix (NBI Item 5B)</u>	<u>Condition Ratings (NBI item 58-62)</u>	<u>Posting evaluation (NBI Item 70)</u>
1 = Interstate highway	9 = Excellent	5 = $\Omega \geq 1.0$
2 = US numbered highway	8 = Very good	4 = $0.99 \geq \Omega > 0.9$
3 = State highway	7 = Good	3 = $0.9 \geq \Omega > 0.8$
4 = County highway	6 = Satisfactory	2 = $0.8 \geq \Omega > 0.7$
5 = City street	5 = Fair	1 = $0.7 \geq \Omega > 0.6$
6 = Federal lands road	4 = Poor	0 = $0.6 \geq \Omega$
7 = State lands road	3 = Series	Ω = operating rating/maximum legal load
8 = Other	2 = Critical	
	1 = Imminent Failure	
	0 = Failed	

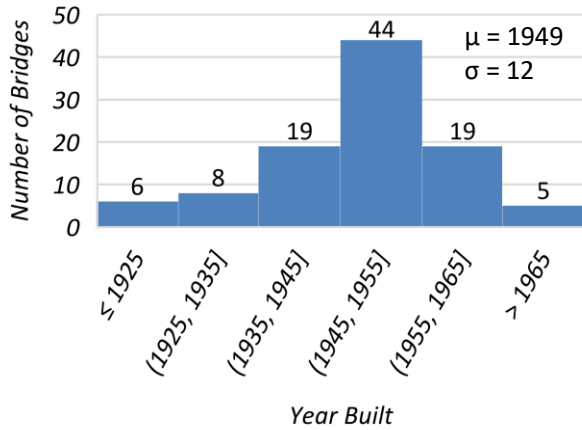
3.4.4 SSLO Simple-Span Concrete Slab Bridges

Concrete bridges constitute 24 percent of all SSLO bridges. Among these, there are 42 on-system and 59 off-system simple-span concrete slab bridges in Texas. Figure 3.9(a, c, e, g) shows the distribution of these bridges by age, geometric parameters, and condition ratings. More than 60 percent of the slab bridges were built between 1939 and 1959. They have an average maximum span length of 22.3 ft with a standard deviation of 3.9 ft. The majority of the simple-span concrete slab bridges, 81 percent, have a span length between 20 and 25 ft. Most concrete slab bridges, 75 percent, are relatively narrow bridges having a width smaller than 36 ft. Although all 101 concrete slab SSLO bridges were posted following TxDOT practices, 70 percent of them have load postings above the maximum legal load, as shown in Figure 3.9(g).

Figure 3.9(b, d, f, h) shows the distribution of the primary variables for the selected subset of 23 SSLO concrete slab bridges in Texas. The initial random sampling was done for 18 bridges, and then 5 more on-system bridges were included in the selected subset. Only one of the selected

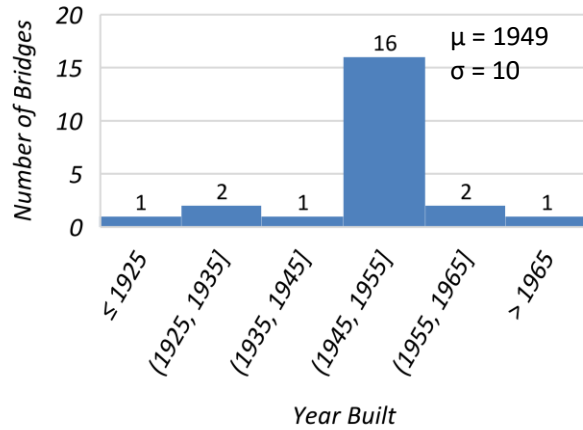
bridges is a one-lane bridge; the rest have two lanes. The comparison of the histograms indicates that the selected subset is a reasonable representation of the original set of concrete slab bridges.

Full dataset of 101 SSLO concrete slab bridges

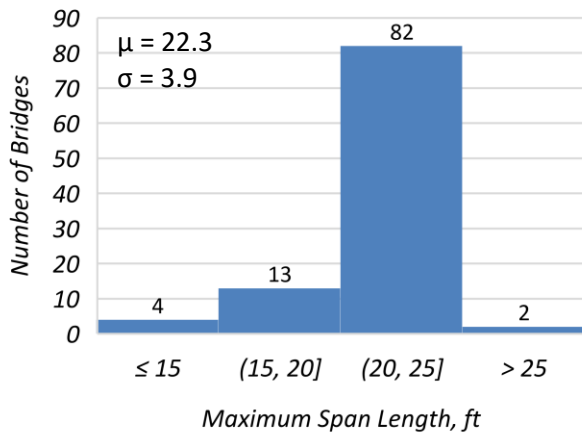


(a) Year built—All 101 bridges

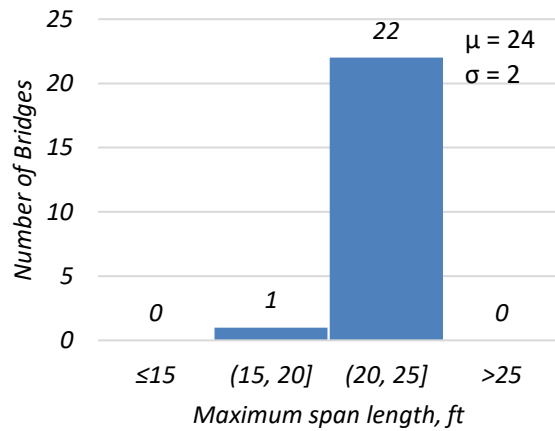
Subset of 23 SSLO concrete slab bridges



(b) Year built—Selected 23 Bridges



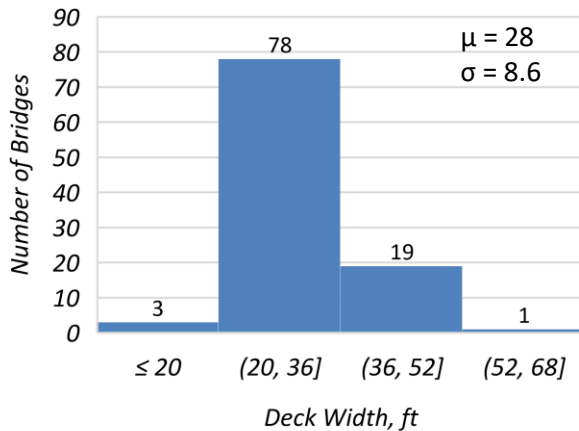
(c) Maximum span length—All 101 bridges



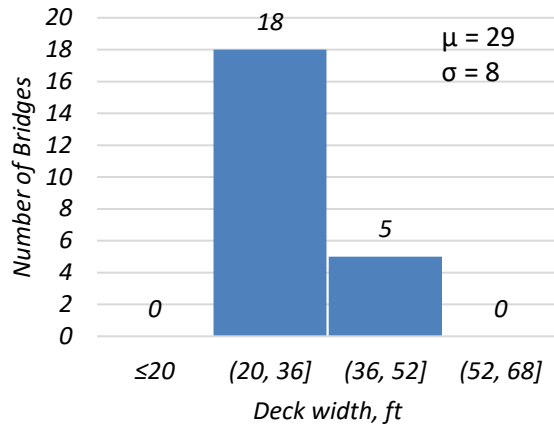
(d) Maximum span length—Selected 23 bridges

Figure 3.9. Distribution of Bridge Parameters for SSLO Concrete Slab Bridges

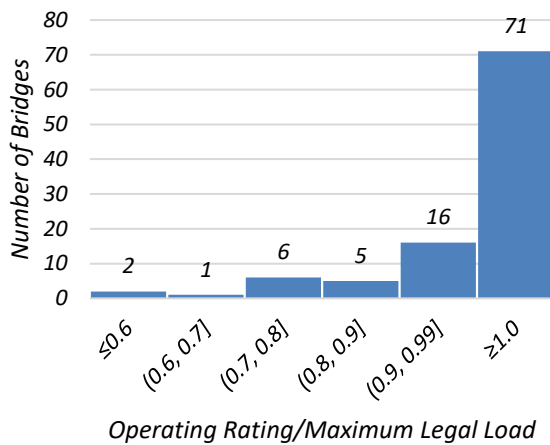
Full dataset of 101 SSLO concrete slab bridges



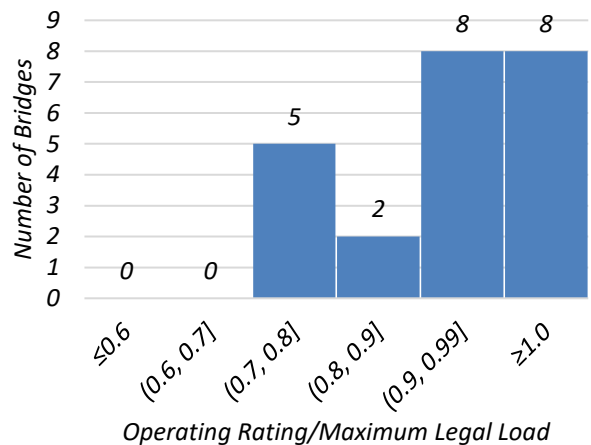
Subset of 23 SSLO concrete slab bridges



(e) Deck width—All 101 bridges



(f) Deck width—Selected 23 bridges



(g) Posting evaluation—All 101 bridges

(h) Posting evaluation—Selected 23 bridges

Figure 3.9. Distribution of Bridge Parameters for SSLO Concrete Slab Bridges (Continued)

Figure 3.10 shows the geographic locations of all 101 SSLO concrete slab bridges (yellow placemark) and the selected 23 SSLO concrete slab bridges (orange placemark). Table 3.5 lists selected SSLO concrete slab bridges, along with their primary geometric properties, condition and operating ratings, ADT, and route information.

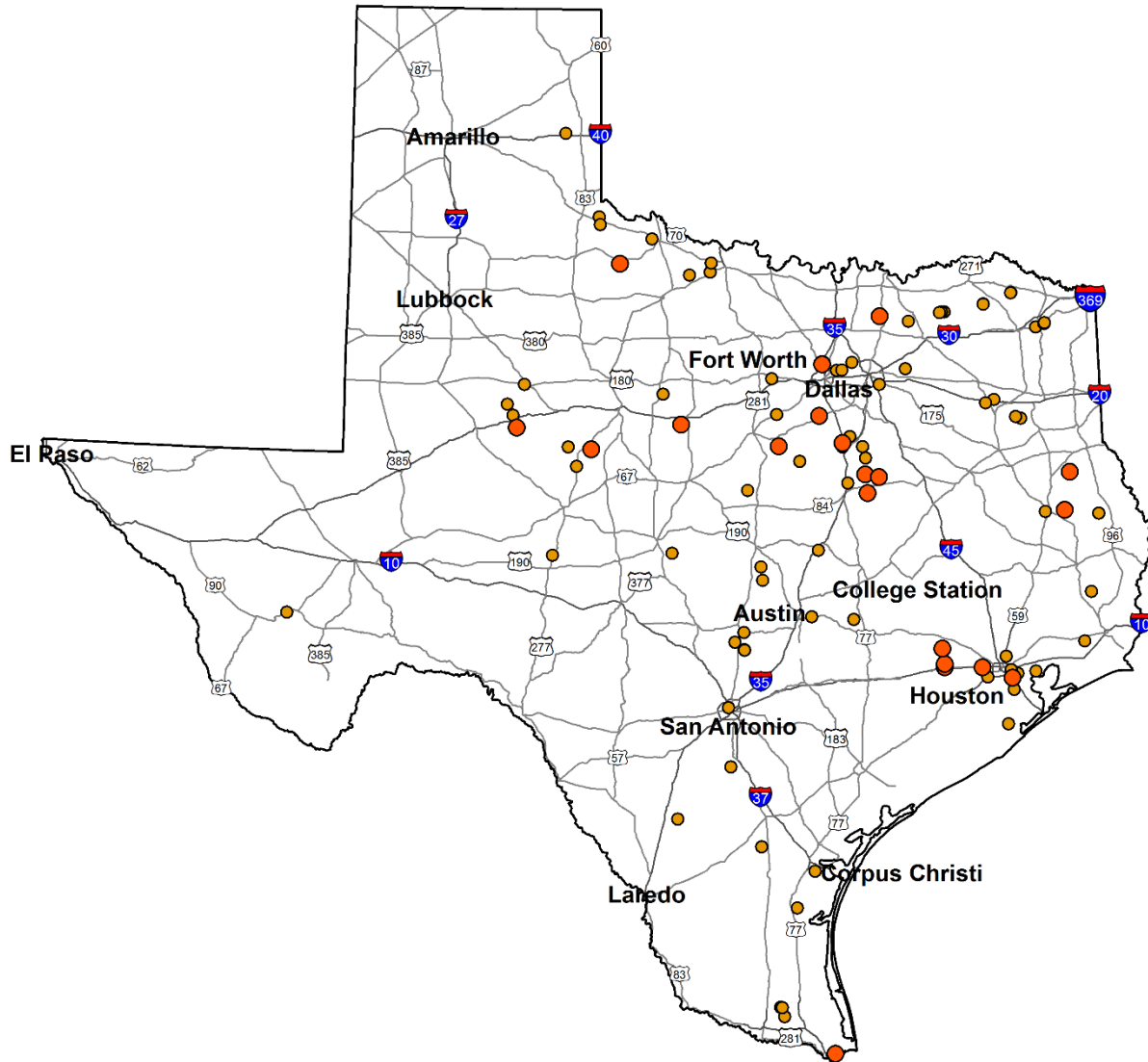


Figure 3.10. Location Map of SSLO Concrete Slab Bridges

*Note: Yellow Placemark = Full dataset of all SSLO concrete slab bridges (101 bridges)
 Orange Placemark = Selected subset of SSLO concrete slab bridges (23 bridges)*

Table 3.5. Selected SSLO Concrete Slab Bridges

ID	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
						Deck	Super-structure	Sub-structure		
CS-1	4	1920	45	18	27	6	6	5	0.99	5
CS-2	4	1930	10	20	21	4	4	4	0.99	5
CS-3	4	1935	45	22	23	5	5	6	0.99	5
CS-4	3	1943	3890	23	46	6	6	6	0.78	4
CS-5	3	1946	1280	25	46	7	7	7	0.69	2
CS-6	4	1947	170	23	25	6	6	5	0.99	5

CS-7	4	1947	60	25	25	6	5	6	0.50	2
CS-8	3	1947	670	25	25	6	6	6	0.91	5
CS-9	3	1948	30	25	21	6	6	7	0.96	4
CS-10	3	1949	9590	25	44	8	6	6	0.85	3
CS-11	3	1949	9590	25	44	6	6	6	0.78	3
CS-12	3	1950	40	25	21	6	6	6	0.96	4
CS-13	3	1951	50	25	21	7	7	6	1.13	4
CS-14	3	1951	310	25	25	7	7	6	0.94	4
CS-15	3	1951	310	25	25	7	7	6	0.94	4
CS-16	3	1951	310	25	25	7	7	6	0.94	4
CS-17	3	1952	270	25	21	7	7	7	0.96	4
CS-18	3	1952	880	25	36	7	7	6	0.74	2
CS-19	3	1953	740	25	35	6	6	6	0.96	4
CS-20	4	1955	60	20	22	7	7	5	0.99	5
CS-21	4	1960	320	25	25	6	6	5	0.99	5
CS-22	5	1962	270	20	34	7	7	7	0.74	2
CS-23	5	1970	1980	25	31	6	6	5	0.99	5

ADT = Average daily traffic.

Route Prefix (NBI Item 5B)

- 1 = Interstate highway
- 2 = US numbered highway
- 3 = State highway
- 4 = County highway
- 5 = City street
- 6 = Federal lands road
- 7 = State lands road
- 8 = Other

Condition Ratings (NBI item 58-62)

- 9 = Excellent
- 8 = Very good
- 7 = Good
- 6 = Satisfactory
- 5 = Fair
- 4 = Poor
- 3 = Series
- 2 = Critical
- 1 = Imminent Failure
- 0 = Failed

Posting evaluation (NBI Item 70)

- 5 = $\Omega \geq 1.0$
 - 4 = $0.99 \geq \Omega > 0.9$
 - 3 = $0.9 \geq \Omega > 0.8$
 - 2 = $0.8 \geq \Omega > 0.7$
 - 1 = $0.7 \geq \Omega > 0.6$
 - 0 = $0.6 \geq \Omega$
- Ω = operating rating/maximum legal load

3.4.5 SSLO Simple-Span Concrete Multi-Girder Bridges

There are a total of 37 simple-span concrete multi-girder SSLO bridges in Texas, which make up 15 percent of all concrete SSLO bridges. Among these, there are 2 on-system and 35 off-system simple-span concrete multi-girder bridges in Texas. Figure 3.11(a, c, e, g) shows the distribution of various characteristics for concrete multi-girder SSLO bridges, including age, main geometric characteristics, and condition ratings. Most of them, 62 percent, have span lengths between 29 and 35 ft, while 70 percent are two-lane bridges having a total bridge width of 24 to 36 ft. Figure 3.11(c) illustrates that these bridges have span lengths either between 29 and 31 ft or 39 and 41 ft. Most SSLO simple-span concrete multi-girder bridges were built between 1952 and 1962, with an average construction year of 1964 and a standard deviation of 12.5 years.

All 37 bridges are load posted per Item 41 of the NBI. However, Figure 3.11(g) indicates that 31 of these concrete multi-girder SSLO bridges have a load posting equal or above the

maximum legal limit. Therefore, posting evaluation criteria were not considered while selecting the short list of concrete multi-girder bridges. Figure 3.11 (b, d, f, h) shows the distribution of the primary variables for the selected subset of 14 concrete multi-girder bridges in Texas. The initial random sampling was done for 10 bridges, and then 4 more bridges were included in the selected subset based on their proximity to College Station for potential field investigation in future tasks. The comparison of the histograms indicates that the selected subset is a good representation of the original set of all SSLO concrete multi-girder bridges.

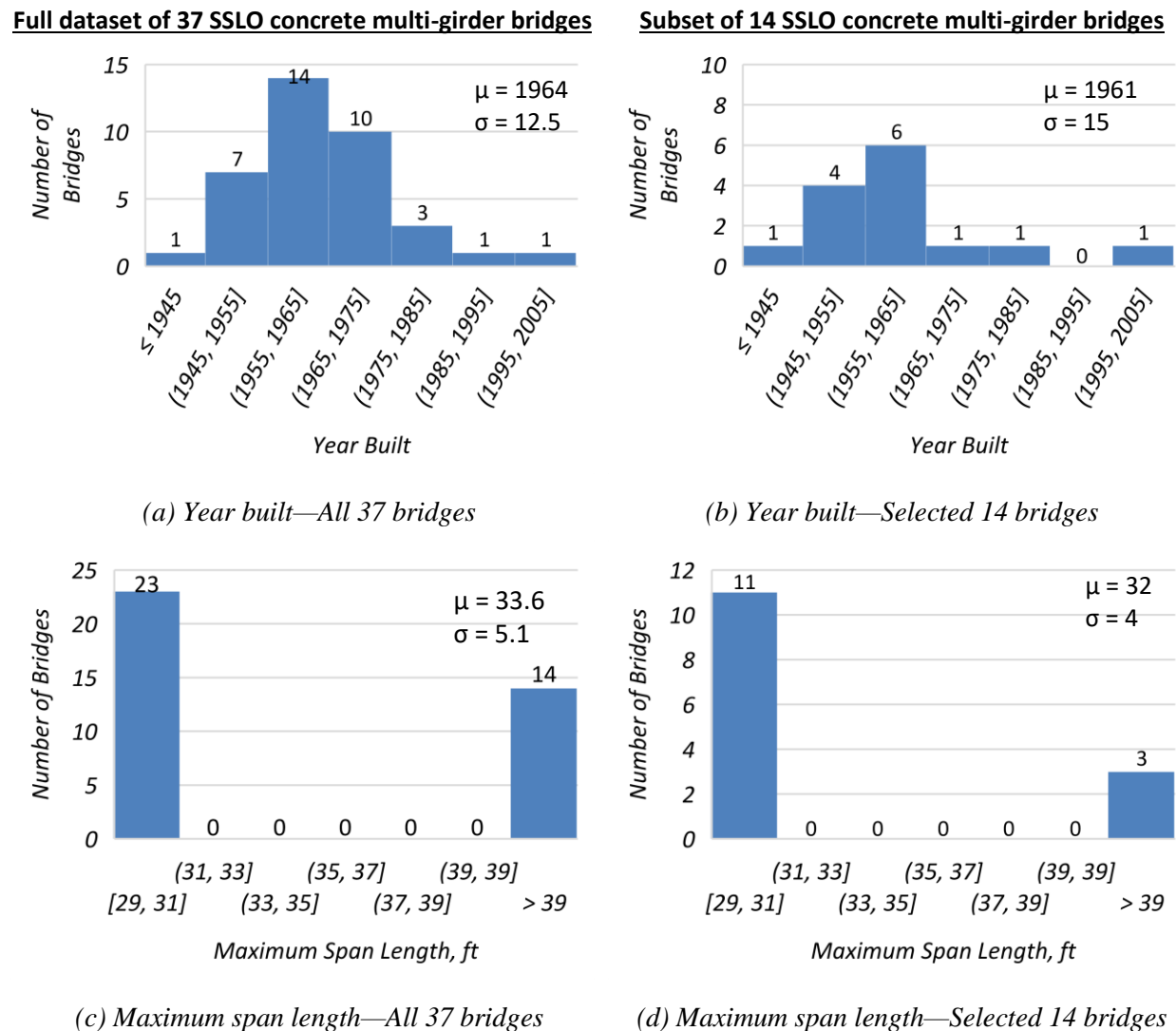
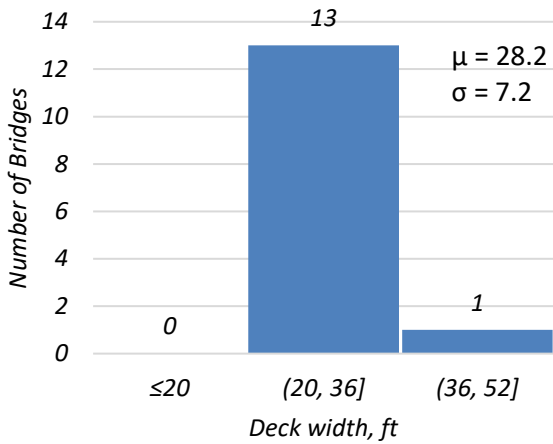


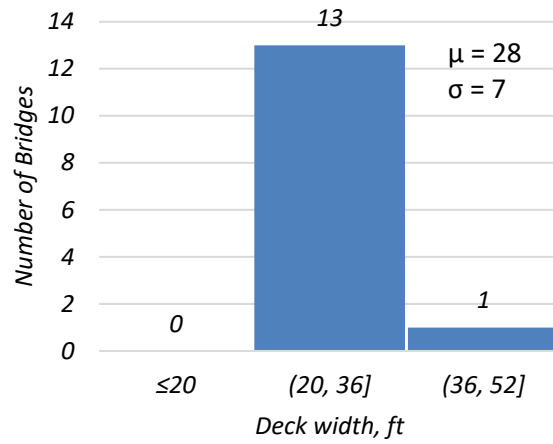
Figure 3.11. Distribution of Bridge Parameters for SSLO Concrete Multi-Girder Bridges

Full dataset of 37 SSLO concrete multi-girder bridges

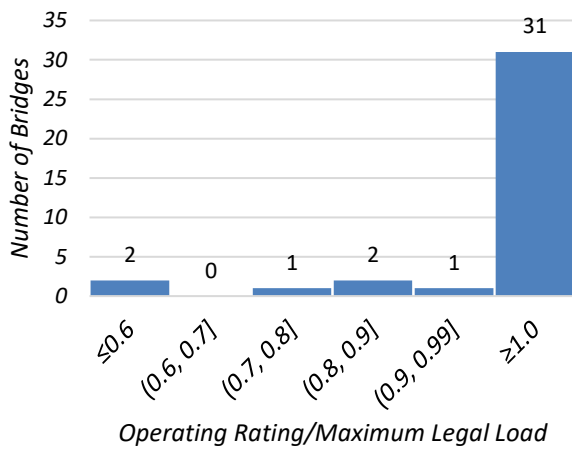


(e) Deck width—All 37 bridges

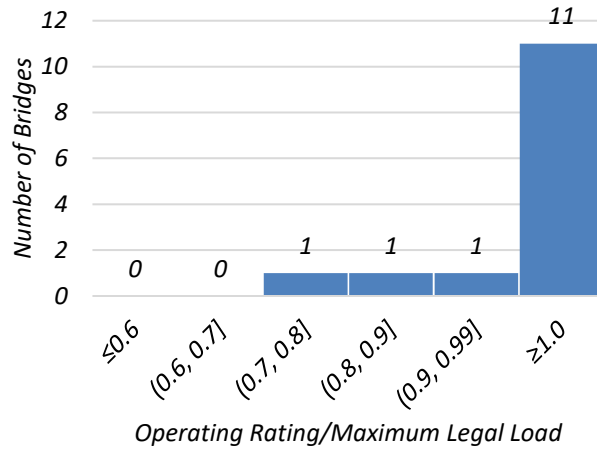
Subset of 14 SSLO concrete multi-girder bridges



(f) Deck width—Selected 14 bridges



(g) Posting evaluation—All 37 bridges



(h) Posting evaluation—Selected 14 bridges

Figure 3.11. Distribution of Bridge Parameters for SSLO Concrete Multi-Girder Bridges (Continued)

Figure 3.12 shows the geographic locations of all 37 SSLO concrete multi-girder bridges (yellow placemark) and the selected 14 SSLO concrete multi-girder bridges (orange placemark) in Texas. Table 3.6 lists selected SSLO concrete multi-girder bridges, along with their primary geometric properties, condition and operating ratings, ADT, and route information.

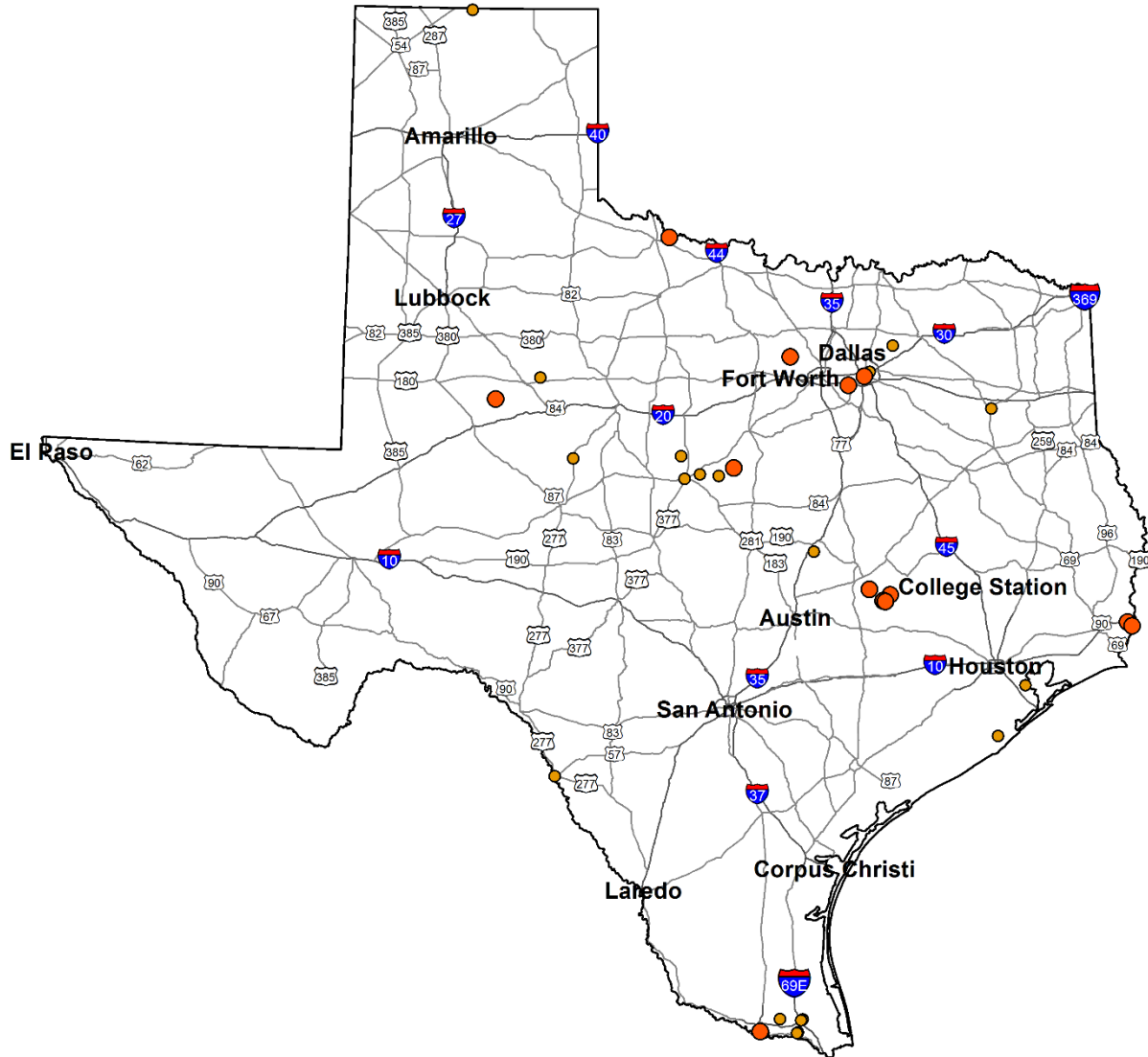


Figure 3.12. Location Map of SSLO Concrete Multi-Girder Bridges

*Note: Yellow Placemark = Full dataset of all SSLO concrete multi-girder bridges (37 bridges)
 Orange Placemark = Selected subset of SSLO concrete multi-girder bridges (14 bridges)*

Table 3.6. Selected SSLO Concrete Multi-Girder Bridges

ID	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
						Deck	Super-structure	Sub-structure		
CM-1	3	1942	170	30	35	7	7	7	0.83	3
CM-2	4	1950	100	29	22	7	7	5	0.99	5
CM-3	4	1950	250	29	22	7	5	5	0.99	5
CM-4	4	1950	250	29	22	7	7	5	0.99	5
CM-5	4	1950	150	29	22	7	7	5	0.99	5
CM-6	3	1957	180	30	25	6	7	6	1.23	5

CM-7	5	1957	2700	40	31	6	5	6	0.74	2
CM-8	4	1960	250	29	22	6	6	5	0.94	4
CM-9	4	1960	950	30	25	7	7	5	0.99	5
CM-10	5	1960	4920	30	33	7	7	5	0.99	5
CM-11	5	1960	2650	40	48	6	6	7	1.27	5
CM-12	4	1975	70	30	25	7	7	6	0.99	5
CM-13	4	1982	80	40	28	7	7	5	1.13	5
CM-14	5	2000	1890	30	32	7	7	8	1.35	5

ADT = Average daily traffic.

<u>Route Prefix (NBI Item 5B)</u>	<u>Condition Ratings (NBI item 58-62)</u>	<u>Posting evaluation (NBI Item 70)</u>
1 = Interstate highway	9 = Excellent	5 = $\Omega \geq 1.0$
2 = US numbered highway	8 = Very good	4 = $0.99 \geq \Omega > 0.9$
3 = State highway	7 = Good	3 = $0.9 \geq \Omega > 0.8$
4 = County highway	6 = Satisfactory	2 = $0.8 \geq \Omega > 0.7$
5 = City street	5 = Fair	1 = $0.7 \geq \Omega > 0.6$
6 = Federal lands road	4 = Poor	0 = $0.6 \geq \Omega$
7 = State lands road	3 = Series	Ω = operating rating/maximum legal load
8 = Other	2 = Critical	
	1 = Imminent Failure	
	0 = Failed	

3.4.6 Summary

The population of load-posted bridges in Texas was reviewed to determine the characteristics that are most typical among these bridges. In particular, details for SSLO bridges are provided because these bridges are the primary candidates to benefit from increases in load rating because they are more likely to remain in service. Key characteristics were reviewed and synthesized to provide an overall understanding of load-posted bridge parameters. These parameters include kind of material and/or design (concrete, steel, wood, etc.), type of design and/or construction (multi-girder, slab, tee beam, box girder, etc.), year built, maximum span length, total bridge length, operating rating, and ratio of operating rating to maximum legal load. As a result of this investigation, SSLO bridges that are load posted based on the MBE and having typical parameters were identified from the following four bridge categories: (1) steel multi-girder, (2) concrete slab, (3) concrete multi-girder, and (4) continuous steel multi-girder bridges. The specific bridges identified are listed in Table 3.3 through Table 3.6. These bridges provide a starting point for further investigation of the Texas load-posted bridge inventory as part of Task 3 and for selection of four typical bridges for the field-testing and refined analysis tasks.

4 LOAD RATING PROCEDURES

4.1 OBJECTIVES AND METHODOLOGY

The primary goal of the current task is to conduct basic load rating analysis for typical bridge populations and to identify areas of opportunity for improving load ratings. A group of representative SSLO bridges were selected on which to conduct basic load rating analysis and evaluate possible approaches to increase load ratings. For all selected bridges, the calculated ratings were compared to the current TxDOT ratings using the same load rating method. The results were analyzed, and reasons for any differences between the calculations and TxDOT ratings were examined and reported. Several areas of opportunities were identified based on the review and synthesis of NBI data and basic load rating analysis of the selected representative bridges. Finally, various key parameters that might be modified were investigated to assess their effect on the resulting load ratings. The identified areas of potential improvement to the ratings are summarized based on these results.

4.2 UPDATES IN THE 2018 MANUAL FOR BRIDGE EVALUATION

The AASHTO released the third edition of the AASHTO MBE in 2018 (AASHTO 2018). While the third edition makes no major changes to the general load rating procedures from the second edition, there are some minor changes that do affect load rating. One of these changes is the reduction of the LRFR live load factors for routine commercial traffic, specialized hauling vehicles, and permit vehicles. However, these new load factors only affect legal load rating and permit load rating; the load factors for design load rating remain the same. This research project is focused on the design load rating, so this change does not affect the calculated load ratings provided in this report.

A second change in the third edition AASHTO MBE that could possibly affect this project is the removal of a statement allowing for concrete bridges that have been “carrying normal traffic for an appreciable length of time and show no signs of distress” (AASHTO 2011) to not require a load posting. This is usually applied to concrete bridges in which the reinforcement details are unknown. TxDOT appears to apply this statement to concrete bridges as well as to the concrete decks of steel bridges since it uses an assumed load rating in both cases when drawings are not available. However, the removal of this statement from the third edition MBE could affect TxDOT

practices for load rating in these cases. The third edition MBE does not provide an alternate option to load rate a concrete bridge in which the reinforcement details are unknown, so TxDOT will need to determine whether to continue this as an in-house practice.

4.3 CURRENT LOAD RATING PROCEDURES

4.3.1 Allowable Stress Rating and Load Factor Rating

Part B of the AASHTO MBE covers the older methods to load rate bridges: Allowable Stress Rating (ASR) and Load Factor Rating (LFR). Both methods use the same general equation to obtain an RF, shown in Equation (4.1).

$$RF = \frac{C - A_1D}{A_2L(1 + I)} \quad (4.1)$$

where:

- RF = Rating factor for the live load-carrying capacity
- C = Capacity of the member
- D = Dead load effect on the member
- L = Live load effect on the member
- I = Impact factor to be used with live load effect
- A_1 = Factor for dead loads
- A_2 = Factor for live load

The ASR and LFR methods do not have the different load rating procedures used for LRFR. The bridge is only evaluated under the design truck load or one of the legal loadings. The A_1 and A_2 factors, which are similar to the dead and live load factors in LRFR, do not change. The design load, and therefore the first loading scenario to be evaluated for ASR and LFR, is the HS-20 truck configuration, as opposed to the HL-93 loading used by LRFR. If the bridge passes (has an RF greater than 1.0) for the HS-20 loading, it also passes for all of the AASHTO and state legal loads.

For the ASR method, the dead and live load factors, A_1 and A_2 , are both equal to 1.0. The capacity of the member changes depending on whether the bridge is being evaluated at inventory level or operating level. The MBE has tables and subsections dedicated to finding the capacity for different materials and situations using the ASR method. As the term *allowable stress rating* suggests, a limit on the maximum stress that a member is allowed to experience is used to

determine the capacity. This limit is usually some portion of the yield or ultimate strength of the member and is lower for inventory level ratings than operating level ratings, thereby producing a smaller inventory RF.

For the LFR method, the dead load factor A_1 is 1.3, and the live load factor A_2 is 2.17 for inventory level rating and 1.3 for operating level rating, thereby producing lower RF for inventory level ratings. Also, unlike the ASR method, the member or component capacity is the same for both the inventory level and operating level ratings. It is calculated using the procedure presented in the *AASHTO Standard Specifications* (AASHTO 2002) and uses different equations depending on the component and material being evaluated.

4.3.2 Load and Resistance Factor Rating

The general equation to obtain the bridge rating factor using the LRFR method is shown in Equation (4.2).

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \quad (4.2)$$

where:

RF = Rating factor

C = Capacity

DC = Dead load effect due to structural components and attachments

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

γ_{DC} = LRFD load factor for structural components and attachments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

γ_P = LRFD load factor for permanent loads other than dead loads = 1.0

γ_{LL} = Evaluation live load factor

For the strength limit states, the capacity is determined from the nominal member resistance multiplied by an LRFD resistance factor ϕ , a system factor ϕ_s , and a condition factor ϕ_c . The product of the condition and system factors is required to be greater than 0.85. An RF is determined for each structural component of the bridge, and the member that has the lowest RF

governs. The RF of the controlling member is then taken to be the RF for the entire structure. The limit state selected for determination of the RF gives specific values for γ_{DC} , γ_{DW} , and γ_{LL} . The primary limit state used for the LRFR method is Strength I; however, the limit state that is evaluated is dependent on the rating procedure used because some limit states only apply to specific rating procedures and not to others. The basic load ratings in this report use the Strength I and Service II limit states at the design load rating level for steel bridges and the Strength I limit state at the design load rating level for reinforced concrete bridges. Table 4.1 shows the dead load and live load factors for these limit states.

Table 4.1. Load Factors for Load Rating for Various Limit States (AASHTO 2018)

Bridge type	Limit State	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load	
				Inventory	Operating
				γ_{LL}	γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35
	Service II	1.00	1.00	1.30	1.00
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35

Design load rating is the first assessment conducted in the LRFR bridge rating procedure and uses the HL-93 load model, which consists of an HS20 truck plus lane load (AASHTO 2014). The design load rating is based on two separate rating levels: inventory rating and operating rating. The inventory rating level applies a higher live load factor (γ_{LL}) than the operating rating level, which means that the inventory RF will be lower than the operating RF. The inventory RF represents the multiple of HL-93 loads that may be applied to the bridge, such that the bridge can be in service for an indefinite period. Similarly, the operating RF represents the multiple of HL-93 loads that is the absolute maximum load that can safely travel on the bridge at one time. Figure 4.1 is a load rating procedure flowchart from the AASHTO MBE (AASHTO 2018).

When load posting a bridge, the MBE leaves much of the decision making up to the owner of the bridge. However, for legal loads such as the AASHTO Type 3, 3S2, and 3-3 trucks, the MBE gives a suggested equation to determine the safe posting load for a bridge if the RF is between 0.3 and 1.0. If the RF is less than 0.3, the bridge is required to be closed. The safe posting load for a structure is given by Equation (4.3).

$$\text{Safe Posting Load} = \frac{W}{0.7} (RF - 0.3) \quad (4.3)$$

where:

RF = Legal load rating factor

W = Weight of rating vehicle

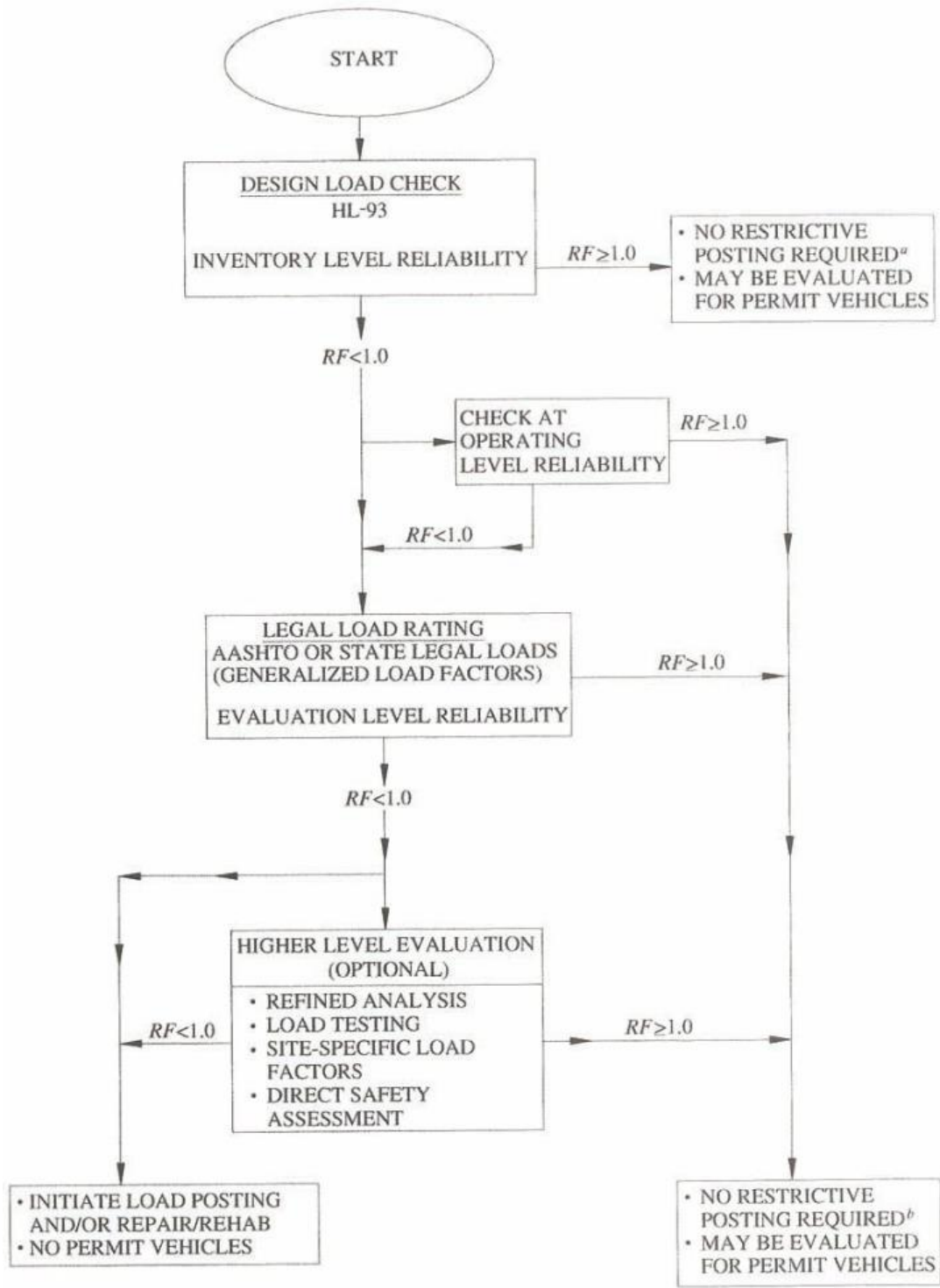


Figure 4.1. AASHTO MBE Load Rating Procedure Flowchart (AASHTO 2018)

4.3.3 Current TxDOT Practices

Currently TxDOT uses the LRFR method to evaluate all bridges designed using LRFD. FHWA requires all bridges on which preliminary engineering was initiated after October 1, 2007, to be designed using LRFD. Most of the bridges that were built after 2007 using LRFD guidelines do not require load posting. All selected load-posted bridges in Texas were built before 2007, and therefore TxDOT analyzes them using either the ASR or LFR methods. TxDOT requires the use of LFR for analysis of all on-system bridges and allows all off-system bridges to be analyzed using either ASR or LFR. In its inspection reports, TxDOT follows in most cases the practice of rating all on-system bridges using LFR and all off-system bridges using ASR. The bridges are analyzed using an H-15 truck and an HS-15 truck, and then one of three flowcharts are followed to determine the posting of the bridge. For on-system bridges, the flowchart shown in Figure 4.2 is followed, for off-system bridges the flowchart shown in Figure 4.3 is applied, and for concrete bridges with no plans, the flowchart shown in Figure 4.4 is used. In the charts, Item 58 is the Deck Condition Rating in the NBI, Item 59 is the Superstructure Condition Rating in the NBI, Item 60 is the Substructure Condition Rating in the NBI, and Item 62 is the Culvert Condition Rating. In the figures, I.F. stands for Inspection Frequency. Figure 4.5 shows the simplified load posting procedure that TxDOT uses to determine the actual posting that goes on the sign. The calculated posting loads in pounds are rounded to the nearest load increment, no matter if it is above or below the computed value.

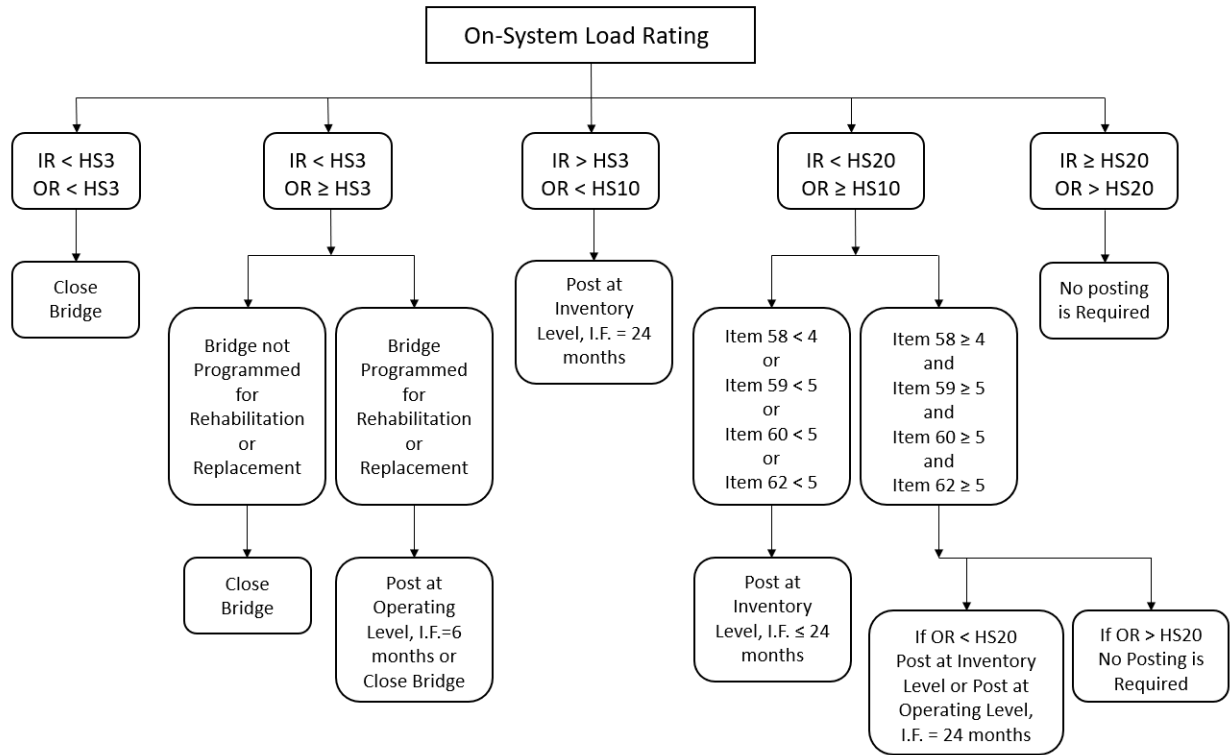


Figure 4.2. TxDOT On-System Load Rating Flowchart (adapted from TxDOT (2018a))

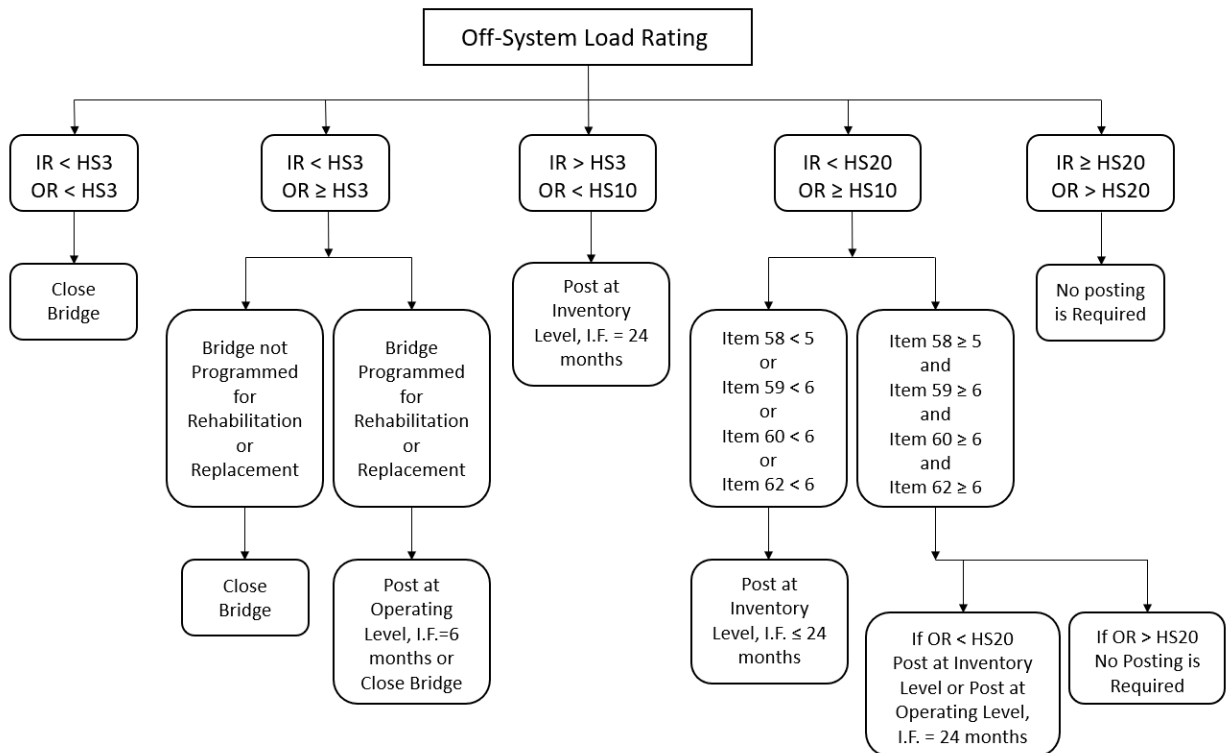


Figure 4.3. TxDOT Off-System Load Rating Flowchart (adapted from TxDOT (2018a))

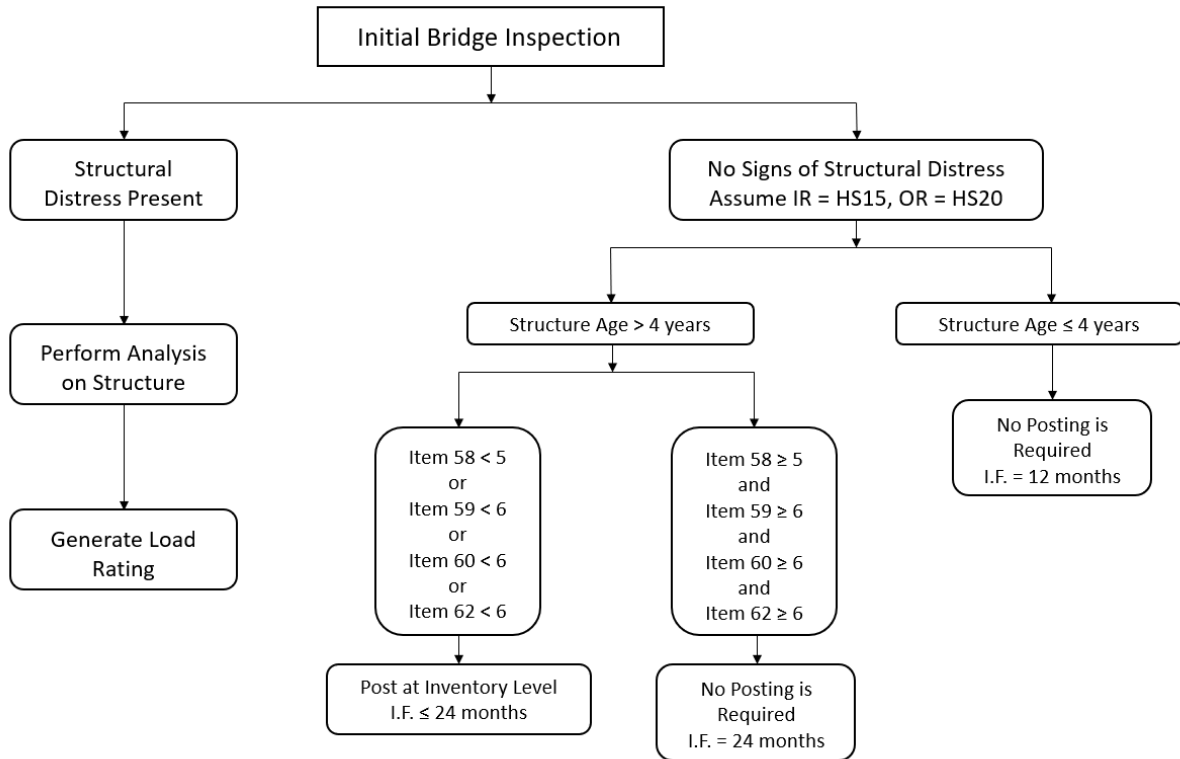


Figure 4.4. TxDOT Load Rating Flowchart for Concrete Bridges with No Plans (adapted from TxDOT (2018a))

SIMPLIFIED LOAD POSTING PROCEDURE

<p>This procedure is appropriate for computing posting loads equivalent to the inventory rating. Approximations are involved which make this procedure unacceptable at load levels higher than the Inventory Rating.</p> <p>The posting load in pounds is the product of the RATING MULTIPLIER and the INVENTORY RATING in tons for the standard "H" truck. In selecting the RATING MULTIPLIER from the table use the longest simple span length or 80% of the longest continuous span length, whichever gives the longest span length for the bridge. If the resulting span length is 160' or greater, then the bridge should receive an analysis more exact than this procedure.</p> <p>The recommended posting increments are listed below. Round off to the nearest increment listed.</p> <p>Post axle and gross load for span lengths 40' and greater. Post axle load only for span lengths 39' and less. Weight limit signs should conform to the Texas Manual on Uniform Traffic Control Devices. The recommended signs are R12-2Tb or R12-4Tb except if the axle load is noted "*" use signs R12-2Tc or R12-4Tc.</p>	SPAN	RATING MULTIPLIER	
		AXLE OR TANDEM	GROSS
	FEET	<u>LBS.</u> H-TON	<u>LBS.</u> H-TON
	≤ 20	1,600	
	25	1,550	
	30	1,500	
	35	1,450	
	40	1,450	3,100
	45	1,450	2,950
	50	1,450	2,800
	60	1,450	2,600
	70	1,450	2,500
	80	1,450	2,450
	90	1,450	2,400
	100	1,450	2,350
	120	1,450	2,300
	140	1,450	2,250
	160	1,450	2,200
		LOAD INCREMENTS FOR AXLE OR TANDEM LBS.	LOAD INCREMENTS FOR GROSS LBS.
		5,000	8,000
		7,500	10,000
		10,000	12,000
		12,500	14,000
		15,000	16,000
		17,500	20,000
		21,000*	24,000
		24,000*	28,000
		28,000*	32,000
		32,000*	36,000
			40,000
			44,000
			48,000
			52,000
			60,000
			68,000
			76,000
		*Axle load exceeds 20,000 lbs. Single axle limit, therefore post for tandem axle (Signs R12-2Tc or R12-4Tc).	

<p>EXAMPLE 1 35' Simple Span Slab & Girder Bridge, H14 Rating Axle = 14 x 1,450 = 20,300 lbs. Post 21,000 tandem axle (Signs R12-2Tc)</p>	<p>EXAMPLE 2 120' Pony Truss, H7 Rating Axle = 7 x 1,450 = 10,100 lbs. Gross = 7 x 2,300 = 16,100 lbs. Post 10,000 lbs. Axle or tandem and 16,000 lbs. Gross (sign R12-4Tb)</p>	<p>EXAMPLE 3 30'-40'-30' Continuous Slab Bridge with 25' slab approach spans, H10 Rating. 0.80 x 40' = 32' > 25' = Use 32' span Axle = 10 x 1,480 = 14,800 lbs. Post 15,000 lbs. Axle or tandem (Sign R12-2Tb)</p>	<p>EXAMPLE 4 25' Simple Span Timber Bridge, H2 Rating Axle = 1,550 x 2 = 3,100 lbs. Recommendation: Close bridge until repair increases capacity.</p>
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<p>WEIGHT LIMIT AXLE OR TANDEM LBS</p> <p>R12-2Tb 24" x 36"</p>	<p>WEIGHT LIMIT TANDEM AXLE LBS</p> <p>R12-2Tc</p>	<p>WEIGHT LIMITS GROSS LBS AXLE OR TANDEM LBS</p> <p>R12-4Tb 24" x 42"</p>	<p>WEIGHT LIMITS GROSS LBS TANDEM AXLE LBS</p> <p>R12-4Tc</p>
-------------------------------------------------------------------------	--------------------------------------------------------	----------------------------------------------------------------------------------------	-----------------------------------------------------------------------

Figure 4.5. TxDOT Simplified Load Posting Procedure Guide (TxDOT 2018a)

Based on its inspections and using the described load rating methods, TxDOT has determined the superstructure operating ratings, superstructure Inventory Ratings, and condition

ratings shown in Table 4.2, Table 4.3, Table 4.4, and Table 4.5 for each selected SSLO bridge, along with the operating and inventory level HS-20 RFs calculated in the inspection reports. The load rating in tons is calculated by multiplying the RF by 20. This research task focused on the superstructure of the bridges. In some instances, the superstructure is not the controlling component of the rating. In many cases, TxDOT also assumes or does not calculate an RF for the substructure, concrete deck, and in some cases for a concrete bridge’s main load-carrying component, based on the flowchart shown in Figure 4.4. The following tables show the load ratings for the superstructure component of the considered bridges.

Table 4.2. TxDOT Superstructure Load Ratings and Condition Ratings for Selected SSLO Simple-Span Steel Multi-Girder Bridges

ID	On/Off System	Condition Rating			Rating Method	Controlling Component	Tonnage Rating (US tons)		HS20 RF	
		Deck	Superstr.	Substr.			Inv.	Oper.	Inv.	Oper.
SM-1	On	6	6	7	LFR	Superstructure	7	11	0.33	0.55
SM-2	On	6	6	6	LFR	Superstructure	7	11	0.34	0.56
SM-3	On	6	6	6	LFR	Superstructure	9	16	0.48	0.80
SM-4	On	6	6	7	N/A	Superstructure	8	11	0.39	0.56
SM-5	On	7	6	7	LFR	Superstructure	9	15	0.47	0.79
SM-6	Off	6	6	7	ASR	Superstructure	15	20	0.89	1.39
SM-7	On	6	5	5	LFR	Superstructure	12	19	0.57	0.95
SM-8	On	6	6	6	LFR	Superstructure	9	14	0.41	0.69
SM-9	Off	7	6	6	ASR	Assumed Deck	15	20	0.82	1.27
SM-10	Off	5	5	6	ASR	Assumed Deck	15	24	0.77	1.20
SM-11	On	6	6	5	LFR	Superstructure	14	24	0.73	1.22
SM-12	Off	6	6	5	LFR	Superstructure	10	17	0.51	0.86
SM-13	Off	5	5	6	ASR	Superstructure	13	20	0.67	1.00
SM-14	Off	7	6	7	ASR	Superstructure	8	15	0.41	0.75
SM-15	On	7	6	6	LFR	Superstructure	10	17	0.51	0.85
SM-16	Off	7	6	7	ASR	Superstructure	16	24	0.82	1.24
SM-17	Off	7	5	5	ASR	Superstructure	11	18	0.55	0.89
SM-18	Off	6	5	7	ASR	Superstructure	12	19	0.59	0.95
SM-19	Off	7	6	5	ASR	Superstructure	10	14	0.71	1.04
SM-20	Off	6	6	5	ASR	Superstructure	11	18	0.56	0.91
SM-21	Off	8	7	7	ASR	Assumed Deck	14	20	1.35	2.02
SM-22	Off	7	6	6	ASR	Superstructure	7	17	0.38	0.87
SM-23	Off	8	6	6	ASR	Superstructure	10	18	0.51	0.88
SM-24	Off	8	5	5	ASR	Superstructure	12	19	0.60	0.99
SM-25	Off	7	6	5	ASR	Assumed Substructure	15	20	3.37	4.77

Note: N/A means that TxDOT did not perform load ratings for this bridge, and it is therefore most likely posted based on precedent.

Condition Ratings:

- 0 = Failed Condition
- 1 = Imminent Failure Condition
- 2 = Critical Condition
- 3 = Serious Condition
- 4 = Poor Condition
- 5 = Fair Condition
- 6 = Satisfactory Condition
- 7 = Good Condition
- 8 = Very Good Condition
- 9 = Excellent Condition

Table 4.3. TxDOT Superstructure Load Ratings and Condition Ratings for Selected SSLO Continuous Steel Multi-Girder Bridges

ID	On/Off System	Condition Rating			TxDOT Rating Method	Controlling Component	Tonnage Rating (US tons)		HS20 RF	
		Deck	Superstr.	Substr.			Inv.	Oper.	Inv.	Oper.
SC-1	Off	6	5	6	ASR	Superstructure	7	11	0.37	0.56
SC-2	Off	5	6	5	ASR	Superstructure	8	14	0.42	0.72
SC-3	Off	5	6	7	ASR	Substructure	13	21	0.99 5	1.46
SC-4	Off	5	6	6	ASR	Substructure	12	20	1.09	1.61
SC-5	Off	5	6	6	ASR	Superstructure	12	18	0.58	0.89
SC-6	Off	5	6	5	ASR	Assumed Deck	15	21	1.08	1.53
SC-7	Off	7	6	5	ASR	Superstructure	15	20	0.84	1.23
SC-8	On	7	6	5	LFR	Superstructure	11	18	0.56	0.93
SC-9	Off	6	6	6	ASR	Superstructure	11	18	0.56	0.93
SC-10	On	6	5	6	LFR	Superstructure	12	19	0.58	0.96
SC-11	Off	6	5	6	ASR	Superstructure	8	12	0.40	0.61
SC-12	On	6	7	7	LFR	Superstructure	10	18	0.55	0.93
SC-13	On	6	6	6	LFR	Superstructure	12	19	0.73	1.22
SC-14	Off	6	6	5	ASR	Substructure	8	12	0.48	0.73
SC-15	Off	7	7	6	ASR	Substructure	11	18	0.56	0.94
SC-16	Off	7	6	6	ASR	Superstructure	7	11	0.34	0.56

Condition Ratings:

- 0 = Failed Condition
- 1 = Imminent Failure Condition
- 2 = Critical Condition
- 3 = Serious Condition
- 4 = Poor Condition
- 5 = Fair Condition
- 6 = Satisfactory Condition
- 7 = Good Condition
- 8 = Very Good Condition
- 9 = Excellent Condition

Table 4.4. TxDOT Superstructure Load Ratings and Condition Ratings for Selected SSLO Concrete Slab Bridges

ID	On/Off System	Condition Rating			TxDOT Rating Method	Controlling Component	Tonnage Rating (US Tons)		HS20 RF	
		Deck	Superstr.	Substr.			Inv.	Oper.	Inv.	Oper.
CS-1	Off	6	6	5	ASR	Substructure	15	20	0.74	0.99
CS-2	Off	4	4	4	N/A	Assumed Superstructure	15	20	0.74	0.99
CS-3	Off	5	5	6	N/A	Assumed Substructure	15	20	0.74	0.99
CS-4	On	6	6	6	LFR	Superstructure	10	16	0.48	0.78
CS-5	On	7	7	7	LFR	Superstructure	8	14	0.41	0.69
CS-6	Off	6	6	5	N/A	Assumed Superstructure	15	20	0.74	0.99
CS-7	Off	6	5	6	LFR	Superstructure	7	10	0.33	0.50
CS-8	On	6	6	6	LFR	Substructure	10	18	0.50	0.91
CS-9	On	6	6	7	LFR	Superstructure	9	18	0.43	0.88
CS-10	On	8	6	7	LFR	Superstructure	10	17	0.52	0.85
CS-11	On	6	6	6	LFR	Superstructure	10	16	0.48	0.78
CS-12	On	6	6	6	LFR	Superstructure	8	18	0.40	0.88
CS-13	On	7	7	6	LFR	Superstructure	10	23	0.52	1.13
CS-14	On	7	7	6	LFR	Superstructure	11	19	0.55	0.94
CS-15	On	7	7	6	LFR	Superstructure	11	19	0.55	0.94
CS-16	On	7	7	6	LFR	Superstructure	11	19	0.55	0.94
CS-17	On	7	7	7	LFR	Superstructure	9	19	0.47	0.96
CS-18	On	7	7	6	LFR	Superstructure	9	15	0.44	0.74
CS-19	On	6	6	6	LFR	Superstructure	10	19	0.50	0.96
CS-20	Off	7	7	5	N/A	Assumed Superstructure	15	20	0.74	0.99
CS-21	Off	6	6	5	N/A	Assumed Superstructure	15	20	0.74	0.99
CS-22	Off	7	7	7	N/A	Assumed Superstructure	10	15	0.50	0.74
CS-23	Off	6	6	5	N/A	Assumed Superstructure	15	20	0.74	0.99

Note: N/A means that TxDOT did not perform load ratings for this bridge, and it is therefore most likely posted based on precedent.

Condition Ratings:

- 0 = Failed Condition
- 1 = Imminent Failure Condition
- 2 = Critical Condition
- 3 = Serious Condition
- 4 = Poor Condition
- 5 = Fair Condition
- 6 = Satisfactory Condition
- 7 = Good Condition
- 8 = Very Good Condition
- 9 = Excellent Condition

Table 4.5. TxDOT Superstructure Load Ratings and Condition Ratings for Selected SSLO Concrete Multi-Girder Bridges

ID	On/Off System	Condition Rating			TxDOT Rating Method	Controlling Component	Tonnage Rating (US Tons)		HS20 RF	
		Deck	Superstr.	Substr.			Inv.	Oper.	Inv.	Oper.
CM-1	On	7	6	6	LFR	Superstructure	8	11	0.40	0.55
CM-2	On	7	7	7	LFR	Assumed Superstructure	10	17	0.50	0.83
CM-3	Off	7	5	5	ASR	Assumed Superstructure	15	20	0.74	0.99
CM-4	Off	7	7	5	ASR	Assumed Superstructure	15	20	0.74	0.99
CM-5	Off	7	7	5	ASR	Assumed Superstructure	14	20	0.72	0.99
CM-6	On	6	7	6	LFR	Superstructure	18	25	0.90	1.23
CM-7	Off	6	5	6	N/A	Assumed Superstructure	11	15	0.55	0.74
CM-8	Off	6	6	5	ASR	Substructure	14	19	0.69	0.94
CM-9	Off	7	7	5	N/A	Assumed Superstructure	15	20	0.74	0.99
CM-10	Off	7	7	5	N/A	Assumed Superstructure	15	20	0.74	0.99
CM-11	Off	6	6	7	ASR	Assumed Superstructure	15	25	0.77	1.27
CM-12	Off	7	7	6	N/A	Assumed Superstructure	15	20	0.74	0.99
CM-13	Off	7	7	5	LFR	Superstructure	13	23	0.66	1.13
CM-14	Off	7	7	8	N/A	Assumed Superstructure	20	27	0.99	1.35

Note: N/A means that TxDOT did not perform load ratings for this bridge, and it is therefore most likely posted based on precedent.

- Condition Ratings:
 0 = Failed Condition
 1 = “Imminent” Failure Condition
 2 = Critical Condition
 3 = Serious Condition
 4 = Poor Condition
 5 = Fair Condition
 6 = Satisfactory Condition
 7 = Good Condition
 8 = Very Good Condition
 9 = Excellent Condition

As can be seen, a number of the selected SSLO bridges have operating RFs greater than 1.0, meaning the operating load rating is greater than an HS-20 truck. Given their condition ratings are greater than or equal to 6 (Satisfactory), these bridges should not be posted according to TxDOT’s on-system and off-system posting flowcharts shown in Figure 4.2 and Figure 4.3. Some of these bridges are posted because of their substructure rating, but some are posted for unknown reasons.

This result could be a continuation of a previous posting based on an observation made by the bridge inspection engineer that is not noted in the database of inspection records (TxDOT 2018c).

5 SSLO STEEL BRIDGES WITH CONCRETE DECKS

5.1 SIMPLE-SPAN STEEL MULTI-GIRDER BRIDGES

5.1.1 Introduction

Twenty-five simple-span steel multi-girder bridges with concrete decks were selected from the entire load-posted bridge inventory of Texas for further analysis. Table 5.1 summarizes these bridges and their main characteristics. Some SM bridges have multiple spans with varying lengths, girder types, or other properties. The information shown in the table contains the characteristics that pertain to the controlling span of the bridge. The girder spacing shown is the largest interior girder spacing in the span if all girders are not equally spaced. Some of the girder cross-sections are historical sections and are named accordingly. A girder named W##x## or S##x## is a current cross-section in the 14th edition of the *AISC Steel Construction Manual* (AISC 2013). A girder named ##WFx##, CB##x##, or B##x## is a historic section found in AISC's *Historic Record, Dimensions and Properties, Rolled Shapes, Steel and Wrought Iron Beams & Columns* (AISC 1953). Detailed characteristics for the selected bridges were compiled from the TxDOT inspection database into a master spreadsheet to efficiently review the relevant information for conducting basic load rating analysis.

Table 5.1. Selected SSLO Simple-Span Steel Multi-Girder Bridges and Characteristics

ID	On/Off System	Year Built	Max. Span Length (ft)	Deck Width (ft)	Steel Yield Strength ² (ksi)	Interior Girder Section	Exterior Girder Section	Interior Girder Spacing (ft)	Concrete Deck Thickness (in.)	Wearing Surface Thickness (in.)
SM-1	On	1931	39	25	30	CB211x58	B18x47	4'-5"	5.50	1.5
SM-2	On	1931	39	25	30	CB211x58	B18x47	4'-5"	5.50	1.5
SM-3	On	1938	30	25	33	21WFx63	21WFx63	7'-1"	8.00	1.0
SM-4	On	1938	30	25	33	21WFx63	21WFx63	7'-1"	6.75	2.0
SM-5	On	1938	41	24	33	S15x42.9	S15x42.9	1'-11"	6.00	1.0
SM-6	Off	1940	24	22	33	S15x42.9	S15x42.9	3'-4.5"	5.50	5.0
SM-7	On	1940	30	23	33	S12x31.8	S12x31.8	1'-10.5"	6.50	2.0
SM-8	On	1940	69	27	33	CB141x34	CB141x34	4'-5"	7.00	4.0
SM-9	Off	1941	40	16	33	21WFx59	B26x91	4'-11"	6.00	0.0
SM-10	Off	1945	29	22	33	S18x54.7	S18x54.7	4'-4"	6.00	1.0
SM-11	On	1947	40	46	33	30WFx108	PC ³	7'-8"	7.25	2.0
SM-12	Off	1955	30	22	33	W18x50	W18x50	6'-0"	6.00	0.0

SM-13	Off	1958	14	26	33	S12x31.8	B64x48	4'-10"	7.00	4.0
SM-14	Off	1960	38	16	33	S15x42.9	C15x33.9	2'-4"	7.50	0.0
SM-15	On	1961	43	26	33	30WFX116	30WFX116	9'-6"	7.00	2.0
SM-16 ¹	Off	1965	36	30	50	W21x55	W21x55	5'-10"	7.00	0.0
SM-17	Off	1970	30	16	33	S12x31.8	C12x20.7	2'-0"	3.50	3.0
SM-18	Off	1977	33	15	27	S18x54.7	S18x54.7	2'-9.5"	5.50	1.0
SM-19	Off	1982	21	14	36	S10x25.4	S10x25.4	2'-8"	4.00	0.0
SM-20	Off	1986	54	24	33	W24x68	W24x68	2'-11"	5.50	0.0
SM-21	Off	1990	53	25	33	W33x130	W33x130	4'-0"	6.00	1.0
SM-22	Off	1990	55	24	36	W33x118	W33x130	6'-8.5"	8.00	5.0
SM-23	Off	1992	48	17	36	W21x44	W21x44	2'-5.5"	6.00	0.5
SM-24	Off	1995	29	14	36	W8x35	W8x35	1'-9.25"	8.00	12.0
SM-25	Off	2000	28	21	36	W24x68	W24x68	2'-7"	7.50	1.0

Notes:

1. SM-16 is the only bridge in this group that is specifically noted as composite in TxDOT's inspection records.
2. Steel yield strength is the value used by TxDOT for load rating calculations, which typically matches the value given in the *TxDOT Bridge Inspection Manual* (TxDOT 2018a) and AASHTO MBE (AASHTO 2018) unless additional information is available.
3. PC = Prestressed Concrete.

5.1.2 Basic Load Rating Analysis

5.1.2.1 General

Each selected steel multi-girder bridge was analyzed using the three rating methods defined in the AASHTO MBE: ASR, LFR, and LRFR. All three rating methods follow a similar general procedure. For the flexure rating factors, the following steps are used:

1. Modified section properties are calculated if the girder has a cover plate in the positive moment region or if it is a composite girder. Composite section properties are calculated using the same procedure as in Example A1 in MBE Appendix A. This procedure finds two different transformed section moduli: one using the width of the transformed concrete deck based on the modular ratio of the steel to concrete elastic moduli ($n=E_s/E_c$) and one using the width of the transformed concrete deck based on $3n$. The section properties based on n are used when considering the live load on the structure, and the section properties based on $3n$ are used when considering the superimposed dead load on the structure. This method is practiced in the rating factor equations for each method.
2. The appropriate LLDFs are determined. For ASR and LFR, these LLDFs are given as the girder spacing over a number based on the number of lanes for interior girders. For exterior girders,

the lever rule is used. For LRFR, the distribution factors are determined as a function of the spacing of the girders and have a range of applicability based on the spacing of the girders, span length, thickness of the slab, and number of girders or determined by using the lever rule. Skew adjustments are also considered.

3. The live load distribution factor, along with the appropriate impact factor, are applied to the live load moment that is extracted from AASHTO MBE Tables E6A-1 and C6B-1 (AASHTO 2018) to get the final maximum girder live load moment for the load being considered. For ASR and LFR, the impact factor is given in Article 3.8.2.1 of the *AASHTO Standard Specifications* and is a function of the span length. For LRFR, the impact factor is always 33 percent. In the AASHTO MBE, Table E6A-1 gives the HL93 live load moment based on span length and Table C6B-1 gives the HS20 live load moment based on span length. Both interior and exterior girders are examined and analyzed.
4. The dead load moments are found by calculating a tributary-distributed load for each girder.
5. The non-composite or composite moment capacity for both the interior and exterior girders is calculated. Consideration is given as to whether the section is compact or noncompact, following the requirements in the respective AASHTO specifications. For ASR, this capacity is given in the form of a stress, as a percentage of the yield stress, from the AASHTO MBE. For the LFR capacity, if the section meets the compact requirements, Equation (5.1) is used. If the section does not meet the compact requirements but does meet the noncompact requirements, Equation (5.2) is used. If the section meets neither the compact nor noncompact requirements, Equation (5.3) is used. For LRFR, if the section meets the compact requirements, Equation (5.4) is used. If the section does not meet the compact requirements but does meet the noncompact requirements, Equation (5.6) is used. The section is also checked using equations based on the braced length of the span. For a composite section for both methods, the plastic neutral axis is determined, and the moments caused by the components are summed about the plastic neutral axis.
6. Load rating is conducted based on the three load rating procedures (ASR, LFR, and LRFR) at the operating and inventory levels by considering both the interior girder and exterior girder moment demands at the strength and service limit states.

$$M_u = F_y Z \quad (5.1)$$

where:

- M_u = Maximum bending strength
- F_y = Specified minimum yield strength of steel
- Z = Plastic section modulus

$$M_u = F_y S_{xt} \quad (5.2)$$

where:

- M_u = Maximum bending strength
- F_y = Specified minimum yield strength of steel
- S_{xt} = Elastic section modulus with respect to the tension flange

$$M_u = M_r R_b \quad (5.3)$$

where:

- M_u = Maximum bending strength
- M_r = Moment capacity—not allowed to exceed yield moment or lateral torsional buckling moment
- R_b = Bending capacity reduction factor

$$M_n = M_y R_b R_h \quad (5.4)$$

where:

- M_n = Nominal flexural resistance of the section
- M_y = Yield moment
- R_b = Web load-shedding factor
- R_h = Hybrid factor

$$M_n = \left[1 - \left(1 - \frac{0.7M_y}{R_h M_y} \right) \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] R_b R_h M_y \quad (5.5)$$

where:

- M_n = Nominal flexural resistance of the section
- M_y = Yield moment
- R_b = Web load-shedding factor
- R_h = Hybrid factor

- λ_f = Slenderness ratio for the compression flange
- λ_{pf} = Limiting slenderness ratio for a compact flange
- λ_{rf} = Limiting slenderness ratio for a noncompact flange

For the shear rating factors, the following steps are used.

1. Since the web of the steel section is the shear resisting component, section properties such as web thickness, web height, and web area are found and calculated.
2. The appropriate LLDFs are determined. For ASR and LFR, LLDFs are given as the girder spacing over a number based on the number of lanes for interior girders. For exterior girders the lever rule is used. For LRFR, LLDFs are determined as a function of the spacing of the girders and have a range of applicability based on the spacing of the girders, span length, thickness of the slab, and number of girders or determined by using the lever rule. Skew adjustments are also considered.
3. The live load distribution factor, along with the appropriate impact factor, are applied to the live load shear that is extracted from MBE Table E6B-1 (AASHTO 2018) to get the final maximum live load shears applied to the bridge for the load being analyzed. For ASR and LFR, the impact factor is given in Article 3.8.2.1 of the *AASHTO Standard Specifications* and is a function of the span length. For LRFR, the impact factor is always 33 percent. Table E6B-1 gives the HS20 live load shear based on span length. For LRFR, the lane load shear was added to this figure based on statics. Both interior and exterior girders are examined and analyzed.
4. The dead load shears are found by calculating a tributary-distributed load for each girder.
5. The web buckling coefficient is determined, and the shear capacity is determined using Equation (5.6).
6. Load rating is conducted based on the three load rating procedures (ASR, LFR, and LRFR) at the operating and inventory levels by considering both the interior girder and exterior girder shear demands at the strength limit state.

$$V_u = 0.58CF_yDt_w \quad (5.6)$$

where:

- V_u = Maximum shear force
- C = Web buckling coefficient

- F_y = Specified minimum yield strength of steel
- D = Clear unsupported distance between flange components
- t_w = Web thickness

5.1.2.2 Allowable Stress Rating and Load Factor Rating

The ASR and LFR calculations follow the *AASHTO Standards Specifications*. The LLDFs for ASR and LFR are calculated according to Article 3.23.2 of the *AASHTO Standard Specifications*.

The non-composite moment capacity is calculated based on the compactness of the section by checking the compact section requirements given in Articles 10.48.1 through 10.48.4 of the *AASHTO Standard Specifications*. For the composite moment capacity, first the location of the plastic neutral axis is determined by equating the compressive and tensile forces in the fully plastic slab and steel section. The plastic moment capacity is found by summing the moments of each component about the plastic neutral axis. Article 10.50.1 of the *AASHTO Standard Specifications* is followed to determine the final composite moment capacity as a value between the plastic moment capacity and elastic moment capacity based on the location of the plastic neutral axis. The service capacities, which are provided as a percentage of the yield strength in Articles 10.57.1 and 10.57.2 of the *AASHTO Standard Specifications*, are also checked. The shear coefficients and capacities are calculated using Equations 10-113 through 10-117 in Article 10.48.8 of the *AASHTO Standard Specifications* because no selected bridge is transversely stiffened.

The ASR inventory and operating capacities for both flexure and shear are determined from Tables 6B.5.2.1-1 and 6B5.2.1-2 in the AASHTO MBE. The dead load moments and live load moments previously found are converted to stresses by dividing by the section modulus. The dead load moments and live load shears previously found are converted to stresses by dividing by the area of the web of the steel section. Then, ASR is conducted for the strength limit state for interior and exterior girders in moment and shear at the inventory and operating levels.

Finally, LFR is conducted for the strength and serviceability limit states for interior and exterior girders in moment and shear at the inventory and operating levels. The procedure for the serviceability limit state for composite sections follows that shown in Example A1 of MBE Appendix A (AASHTO 2018) and involves considering the dead load moment and superimposed dead load moment under different transformed sections for the member.

For both LFR and ASR methods, the controlling RFs are multiplied by the weight of the truck being analyzed to determine a gross weight of that truck allowed on the bridge.

5.1.2.3 Load and Resistance Factor Rating

The LRFR calculations follow the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2014). The LLDFs are found according to Article 4.6.2.2 of the *AASHTO LRFD Specifications*. These distribution factors are adjusted appropriately, accounting for skew using Table 4.6.2.2.1-1 in the *AASHTO LRFD Specifications*.

When determining non-composite moment capacity, the slenderness parameters, web plastification, web load-shedding, and hybrid factors are first calculated. Then, if the skew of the bridge is less than 20 degrees, the non-composite moment capacity is calculated using Sections A6.1 through A6.3 of the *AASHTO LRFD Specifications*. If the skew is over 20 degrees, the non-composite moment capacity is calculated using Article 6.10.8.2 of the *AASHTO LRFD Specifications*. The composite moment capacity is found using the same procedure as is used in the ASR/LFR section and Article 6.10.7 of the *AASHTO LRFD Specifications*. The appropriate capacity is then chosen based on whether the bridge is classified as composite or non-composite.

The shear coefficient and capacity are found using Article 6.10.9.2 of the *AASHTO LRFD Specifications* since no selected bridge is transversely stiffened. The service capacities are given in Article 6.10.4.2 of the *AASHTO LRFD Specifications* as a percentage of the yield strength.

Finally, LRFR rating is conducted at the Strength I and Service II limit states, checking interior and exterior girders in moment and shear at the inventory and operating levels. The controlling LRFR RFs are found at the inventory and operating levels and are also multiplied by the weight of the truck being analyzed to determine a gross weight of that truck allowed on the bridge using the LRFR method.

5.1.3 Calculated Load Rating Results

Each of the 25 selected simple-span, steel multi-girder bridges with concrete decks was rated using each of the three rating methods. The bridges were rated for an HS-20 truck for ASR and LFR since that is the truck TxDOT uses when reporting to the NBI, and the HL-93 design load was used for LRFR. The bridges were examined and rated for interior girder flexure, exterior girder flexure, interior girder shear, and exterior girder shear. The bridges were initially rated by

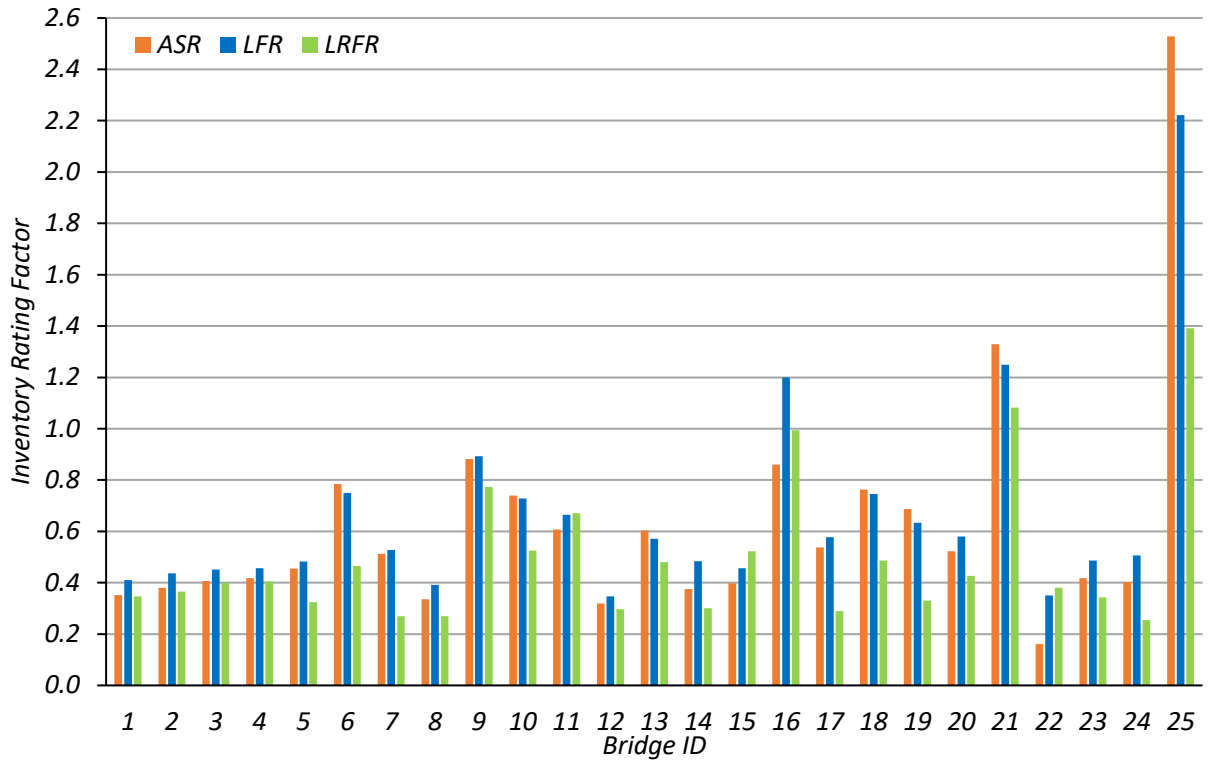
following the procedures laid out in the *AASHTO MBE*, *AASHTO Standard Specifications*, and *AASHTO LRFD Specifications*.

5.1.3.1 Interior Girder Flexure Analysis

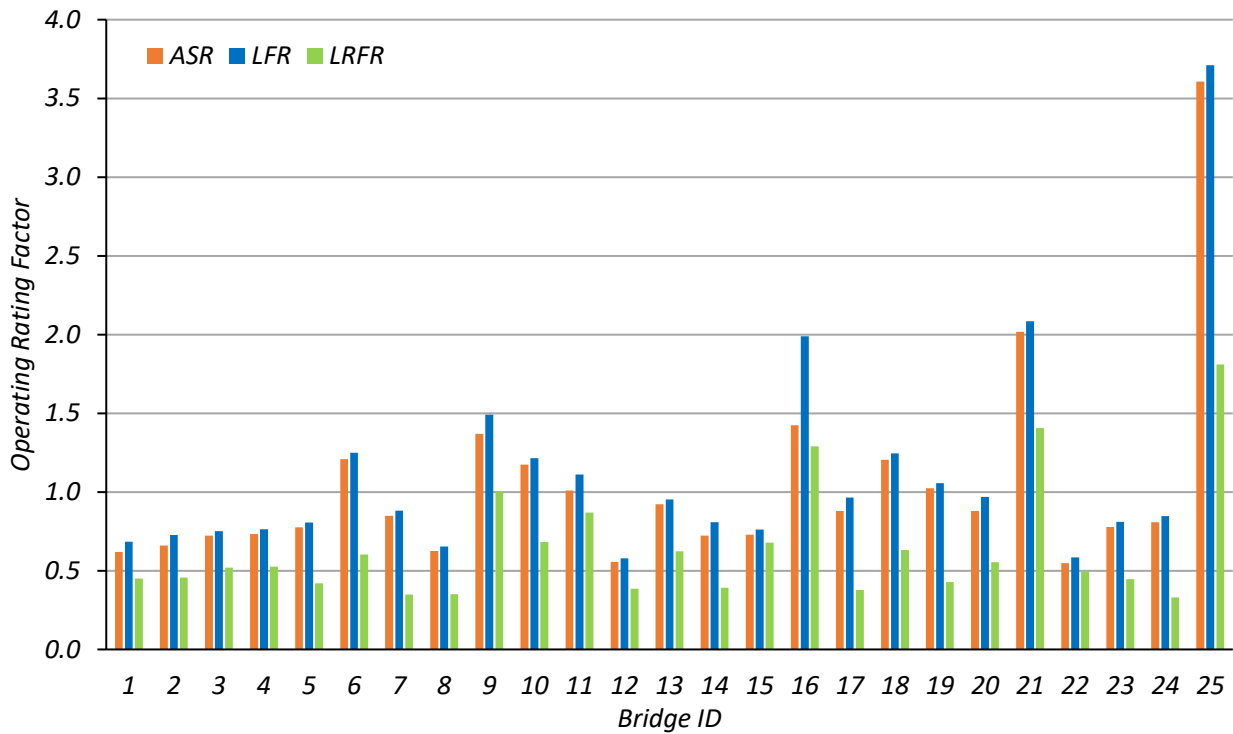
The flexure RFs found for the design loads using each method for the interior girders of each SM bridge are shown in Table 5.2 and Figure 5.1.

Table 5.2. Interior Girder Flexure RFs for SM Bridges

Bridge ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SM-1	0.35	0.41	0.35	0.62	0.69	0.45
SM-2	0.38	0.44	0.37	0.66	0.73	0.46
SM-3	0.41	0.45	0.40	0.72	0.75	0.52
SM-4	0.42	0.46	0.41	0.73	0.76	0.53
SM-5	0.46	0.48	0.32	0.78	0.81	0.42
SM-6	0.78	0.75	0.47	1.21	1.25	0.60
SM-7	0.51	0.53	0.27	0.85	0.88	0.35
SM-8	0.34	0.39	0.27	0.63	0.65	0.35
SM-9	0.88	0.89	0.77	1.37	1.49	1.01
SM-10	0.74	0.73	0.53	1.17	1.22	0.68
SM-11	0.61	0.67	0.67	1.01	1.11	0.87
SM-12	0.32	0.35	0.30	0.56	0.58	0.39
SM-13	0.60	0.57	0.48	0.92	0.95	0.62
SM-14	0.38	0.48	0.30	0.72	0.81	0.39
SM-15	0.40	0.46	0.52	0.73	0.76	0.68
SM-16	0.86	1.20	0.99	1.43	1.99	1.29
SM-17	0.54	0.58	0.29	0.88	0.97	0.38
SM-18	0.76	0.75	0.49	1.20	1.25	0.63
SM-19	0.69	0.63	0.33	1.02	1.06	0.43
SM-20	0.52	0.58	0.43	0.88	0.97	0.56
SM-21	1.33	1.25	1.08	2.02	2.08	1.41
SM-22	0.16	0.35	0.38	0.55	0.59	0.50
SM-23	0.42	0.49	0.34	0.78	0.81	0.45
SM-24	0.40	0.51	0.25	0.81	0.85	0.33
SM-25	2.53	2.22	1.39	3.61	3.71	1.81
Average	0.63	0.66	0.50	1.03	1.11	0.64
Std. Dev.	0.46	0.39	0.28	0.62	0.65	0.36



(a) Inventory RFs



(b) Operating RFs

Figure 5.1. Interior Girder Flexure RFs for SM Bridges

For the interior girder flexure of the selected SM bridges, the LFR method produced a higher average inventory RF (0.66) than the ASR method (0.63), which produced a higher average inventory RF than the LRFR method (0.50). The LFR method also produced a higher average operating RF (1.11) than the ASR method (1.03), which produced a higher average operating RF than the LRFR method (0.64).

Of the 25 bridges whose interior girders were analyzed, two had HS-20 inventory ratings at or above 1.0, and nine had HS-20 operating ratings at or above 1.0 using the ASR method. Three bridges had HS-20 inventory ratings at or above 1.0, and nine had HS-20 operating ratings at or above 1.0 using the LFR method. Two bridges had HL-93 inventory ratings at or above 1.0, and four had HL-93 operating ratings at or above 1.0 using the LRFR method. Bridges SM-21 and SM-25 were the only SM bridges that had operating and inventory design-level ratings at or above 1.0 using all three methods. SM-16 was the only bridge in addition to these two that also had both operating and inventory design-level ratings at or above 1.0 using the LFR method. The three bridges that had operating RFs greater than 1.0 using all three rating methods were Bridges SM-16, SM-21, and SM-25.

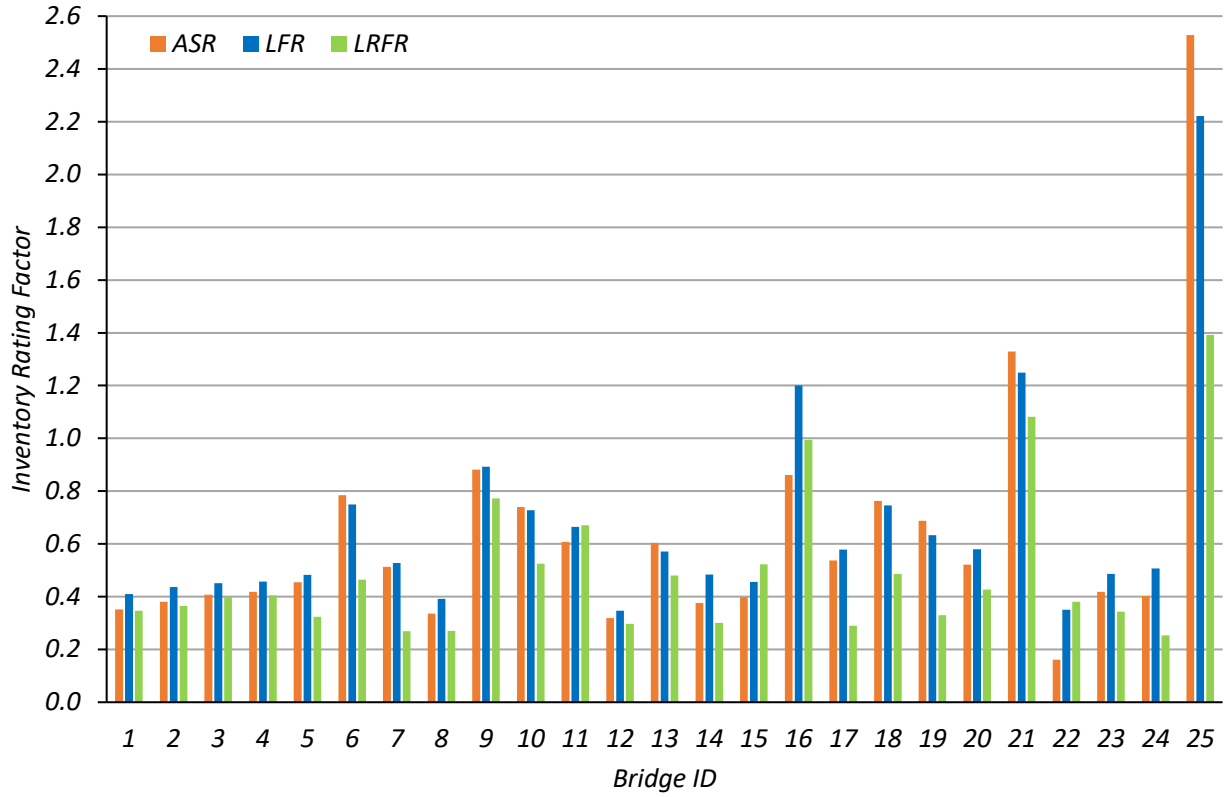
5.1.3.2 Exterior Girder Flexure Analysis

The exterior girders of the bridges were rated separately for this project. It should be noted that the TxDOT practice appears to rate the superstructure of SM bridges based on the interior girders only. Each SM bridge was analyzed in this project considering the exterior girders and the related AASHTO requirements, which mainly resulted in differences in distribution factors. Table 5.3 and Figure 5.2 show the exterior girder flexure RFs found using each rating method for each simple-span SM bridge. Note that an exterior girder RF was not determined for Bridge SM-11 because it was widened, and the new exterior girder is a prestressed concrete I-girder instead of a steel section.

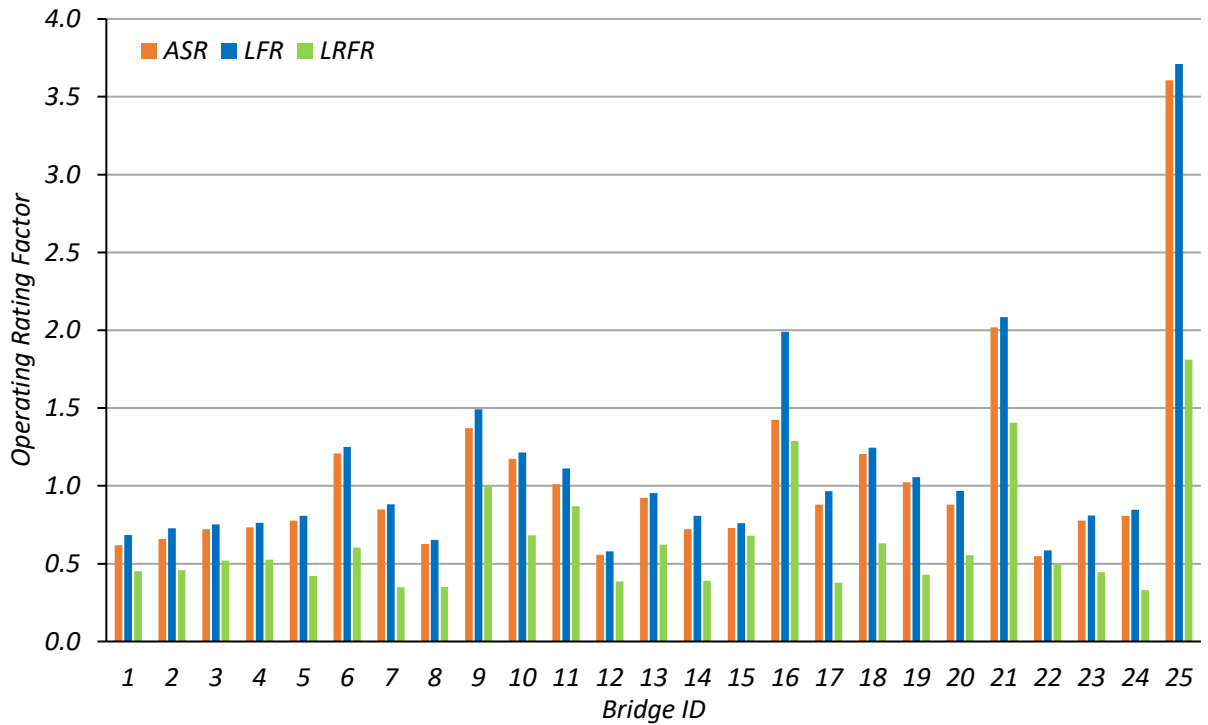
Table 5.3. Exterior Girder Flexure RFs for SM Bridges

Bridge ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SM-1	0.64	0.63	0.30	0.96	1.04	0.39
SM-2	0.68	0.66	0.31	1.01	1.10	0.40
SM-3	0.51	0.52	0.44	0.84	0.87	0.57
SM-4	0.52	0.53	0.44	0.85	0.88	0.57

SM-5	0.53	0.53	0.35	0.86	0.89	0.46
SM-6	0.93	0.84	0.52	1.36	1.40	0.67
SM-7	0.51	0.53	0.27	0.85	0.88	0.35
SM-8	0.52	0.50	0.34	0.81	0.84	0.44
SM-9	0.79	0.76	0.84	1.18	1.27	1.09
SM-10	0.71	0.65	0.59	1.05	1.08	0.77
SM-11	–	–	–	–	–	–
SM-12	0.36	0.37	0.32	0.60	0.62	0.41
SM-13	1.73	1.50	1.31	2.43	2.50	1.69
SM-14	0.28	0.29	0.25	0.47	0.49	0.32
SM-15	0.54	0.58	0.54	0.93	0.97	0.70
SM-16	1.02	1.12	0.99	1.56	1.87	1.29
SM-17	0.26	0.26	0.18	0.42	0.43	0.23
SM-18	0.21	0.22	0.19	0.36	0.37	0.25
SM-19	0.60	0.54	0.35	0.87	0.90	0.46
SM-20	0.64	0.65	0.48	1.00	1.09	0.62
SM-21	1.48	1.34	1.16	2.17	2.24	1.50
SM-22	0.46	0.57	0.58	0.91	0.96	0.76
SM-23	0.46	0.46	0.40	0.75	0.78	0.52
SM-24	0.42	0.46	0.29	0.74	0.77	0.38
SM-25	2.58	2.23	1.44	3.62	3.72	1.87
Average	0.72	0.70	0.54	1.11	1.16	0.70
Std. Dev.	0.52	0.44	0.35	0.71	0.73	0.45



(a) Inventory RFs



(b) Operating RFs

Figure 5.2. Exterior Girder Flexure RFs for SM Bridges

For the exterior girder flexure of the selected SM bridges, the ASR method produced a higher average inventory RF (0.72) than the LFR method (0.70), which produced a higher average inventory RF than the LRFR method (0.54). However, the LFR method produced a higher average operating RF (1.16) than the ASR method (1.11), which produced a higher average operating RF than the LRFR method (0.70).

The exterior girder controlled the rating for seven of the considered SM bridges. Table 5.4 and Figure 5.3 show the calculated operating and inventory design-level RFs for the interior and exterior girders of these bridges. It should be noted, however, that the results obtained when considering an exterior girder may be overly conservative. For these seven bridges (SM-10, SM-14, SM-17, SM-18, SM-19, SM-23, and SM-24), the exterior girder controls the rating due to restrictions in Article 3.23.2.3.1.5 of the *AASHTO Standard Specifications*. This article puts a minimum on the exterior girder LLDF determined using the lever rule. Without this minimum, the exterior girder would not control the rating. However, in order to follow the *AASHTO Standard Specifications* as closely as possible, this article was considered. Further investigation into this limit and refined LLDFs will be conducted in Tasks 4 and 5 of this project.

Table 5.4. SM Bridges Controlled by Exterior Girder Flexure RFs

ID	TxDOT Rating Method	Interior Girder Rating		Exterior Girder Rating		Exterior Rating/ Interior Rating	
		Inventory	Operating	Inventory	Operating	Inventory	Operating
SM-10	ASR	0.74	1.17	0.71	1.05	0.96	0.90
SM-14	ASR	0.38	0.72	0.28	0.47	0.74	0.65
SM-17	ASR	0.54	0.88	0.26	0.42	0.48	0.48
SM-18	ASR	0.76	1.20	0.21	0.36	0.28	0.30
SM-19	ASR	0.69	1.02	0.60	0.87	0.87	0.85
SM-23	ASR	0.42	0.78	0.42	0.75	1.00	0.96
SM-24	ASR	0.40	0.81	0.40	0.74	1.00	0.91
Average		0.56	0.94	0.41	0.67	0.76	0.72

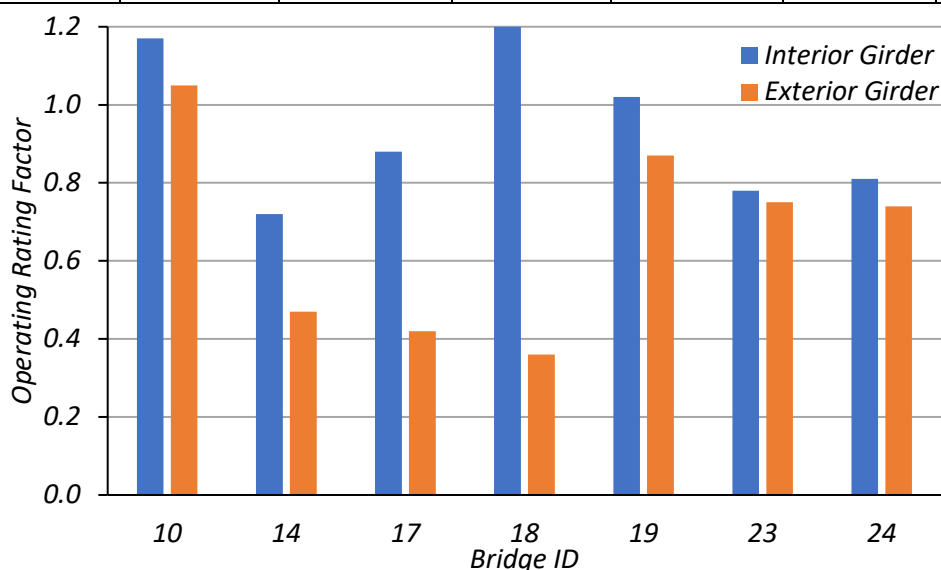


Figure 5.3. Interior Girder and Exterior Girder RFs for Select SM Bridges

5.1.3.3 Interior Girder Shear Analysis

Table 5.5 and Figure 5.4 show the shear RFs calculated for the design loads on the interior girders for each selected simple-span steel multi-girder bridge.

Table 5.5. Interior Girder Shear RFs for SM Bridges

ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SM-1	2.16	1.81	1.58	3.05	3.02	2.05
SM-2	2.18	1.83	1.60	3.09	3.06	2.07
SM-3	1.81	1.54	1.57	2.60	2.57	2.03
SM-4	1.82	1.55	1.57	2.61	2.58	2.04
SM-5	4.58	3.78	1.75	6.37	6.31	2.27

SM-6	3.13	2.60	1.90	4.37	4.33	2.46
SM-7	3.53	2.93	1.14	4.93	4.89	1.48
SM-8	1.45	1.25	0.99	2.11	2.09	1.28
SM-9	2.99	2.50	1.84	4.21	4.17	2.38
SM-10	3.91	3.24	2.10	5.46	5.41	2.73
SM-11	2.97	2.49	2.30	4.19	4.16	2.99
SM-12	1.60	1.35	1.30	2.27	2.25	1.68
SM-13	2.01	1.69	0.92	2.84	2.82	1.19
SM-14	4.78	3.96	1.70	6.68	6.62	2.20
SM-15	2.34	1.99	2.17	3.35	3.32	2.81
SM-16	2.97	2.48	2.14	4.18	4.14	2.77
SM-17	4.21	3.47	1.26	5.85	5.80	1.63
SM-18	4.69	3.87	1.72	6.53	6.47	2.23
SM-19	2.97	2.45	1.13	4.12	4.08	1.47
SM-20	4.64	3.84	2.38	6.47	6.41	3.09
SM-21	6.68	5.51	4.31	9.29	9.20	5.58
SM-22	3.64	3.12	2.95	5.25	5.20	3.82
SM-23	5.79	4.79	2.08	8.07	8.00	2.70
SM-24	2.74	2.33	0.77	3.93	3.89	0.99
SM-25	7.13	5.84	3.49	9.84	9.75	4.52
Average	3.47	2.89	1.87	4.87	4.82	2.42
Std. Dev.	1.51	1.23	0.79	2.07	2.05	1.02

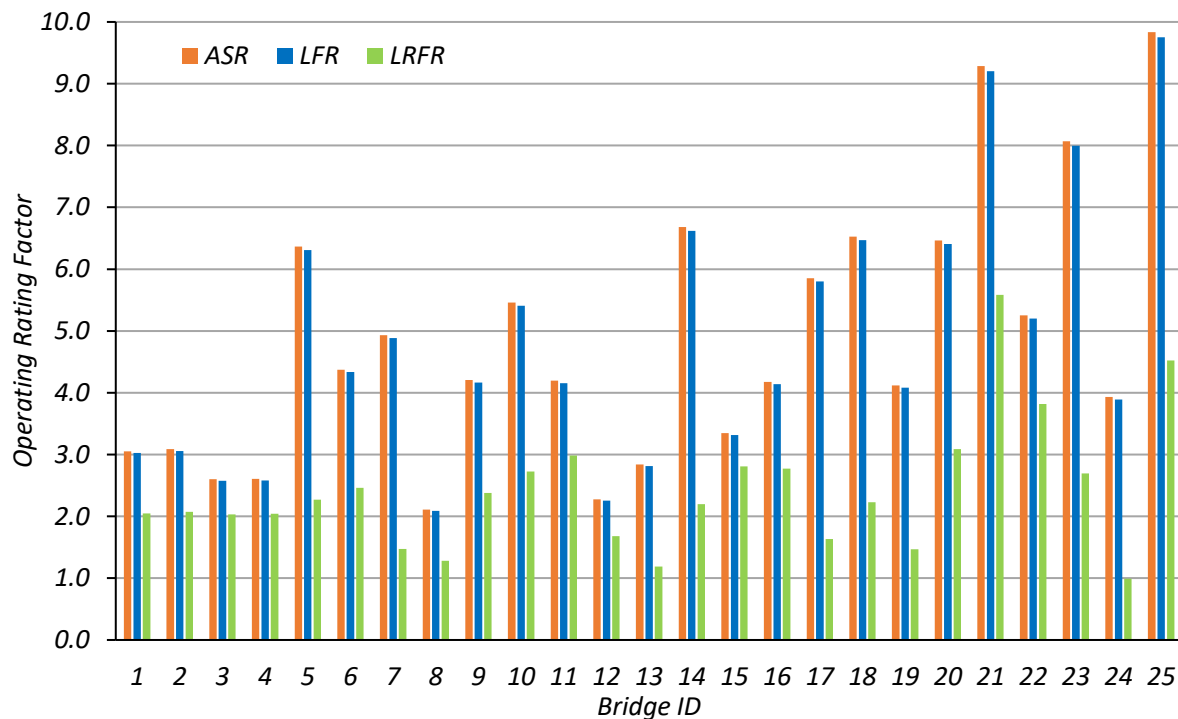


Figure 5.4. Interior Girder Shear Operating RFs for SM Bridges

For the interior girder shear of the selected SM bridges, the ASR method produced a higher average inventory RF (3.47) than the LFR method (2.89), which produced a higher average inventory RF than the LRFR method (1.87). The ASR method also produced a higher average operating RF (4.87) than the LFR method (4.82), which produced a higher average operating RF than the LRFR method (1.02).

For all bridges, both the ASR method and the LFR method produced operating and inventory interior girder shear RFs greater than or equal to 1.0. There were only two bridges, SM-8 and SM-24, for which the LRFR method did not produce inventory RFs greater than 1.0. The LRFR method produced operating RFs greater than or equal to 1.0 for all bridges except for one, SM-24, which had an RF of 0.99. None of the interior girder shear RFs controlled over the corresponding interior girder flexure RFs.

5.1.3.4 Exterior Girder Shear Analysis

Table 5.6 and Figure 5.5 show the shear RFs calculated for the design loads using each method for the exterior girders of all selected simple-span steel multi-girder bridges.

Table 5.6. Exterior Girder Shear RFs for SM Bridges

ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SM-1	15.50	12.74	1.38	21.46	21.27	1.79
SM-2	15.66	12.87	1.40	21.68	21.48	1.81
SM-3	2.47	2.08	1.60	3.50	3.47	2.08
SM-4	2.48	2.08	1.61	3.51	3.48	2.08
SM-5	16.18	13.31	1.77	22.42	22.22	2.29
SM-6	5.37	4.41	1.94	7.42	7.36	2.52
SM-7	12.04	9.98	1.14	16.81	16.66	1.48
SM-8	2.26	1.89	1.05	3.18	3.15	1.37
SM-9	6.43	5.28	2.90	8.90	8.82	3.76
SM-10	7.22	5.93	2.15	9.98	9.90	2.79
SM-11	-	-	-	-	-	-
SM-12	2.05	1.72	1.32	2.90	2.87	1.71
SM-13	6.04	4.96	1.64	8.35	8.27	2.13
SM-14	16.06	13.19	1.69	22.21	22.01	2.19
SM-15	2.81	2.37	2.20	3.99	3.95	2.85
SM-16	5.44	4.50	2.19	7.58	7.51	2.84

SM-17	9.98	8.19	1.04	13.79	13.66	1.35
SM-18	1.52	1.26	1.10	2.12	2.10	1.43
SM-19	3.36	2.75	1.15	4.63	4.59	1.49
SM-20	6.51	5.36	2.42	9.03	8.95	3.14
SM-21	9.91	8.13	4.37	13.70	13.58	5.66
SM-22	4.50	3.80	3.19	6.40	6.34	4.14
SM-23	20.76	17.08	2.11	28.77	28.51	2.74
SM-24	7.19	6.05	0.79	10.20	10.10	1.02
SM-25	15.46	12.63	3.52	21.26	21.07	4.57
Average	8.22	6.77	1.90	11.41	11.31	2.47
Std. Dev.	5.57	4.56	0.86	7.68	7.61	1.11

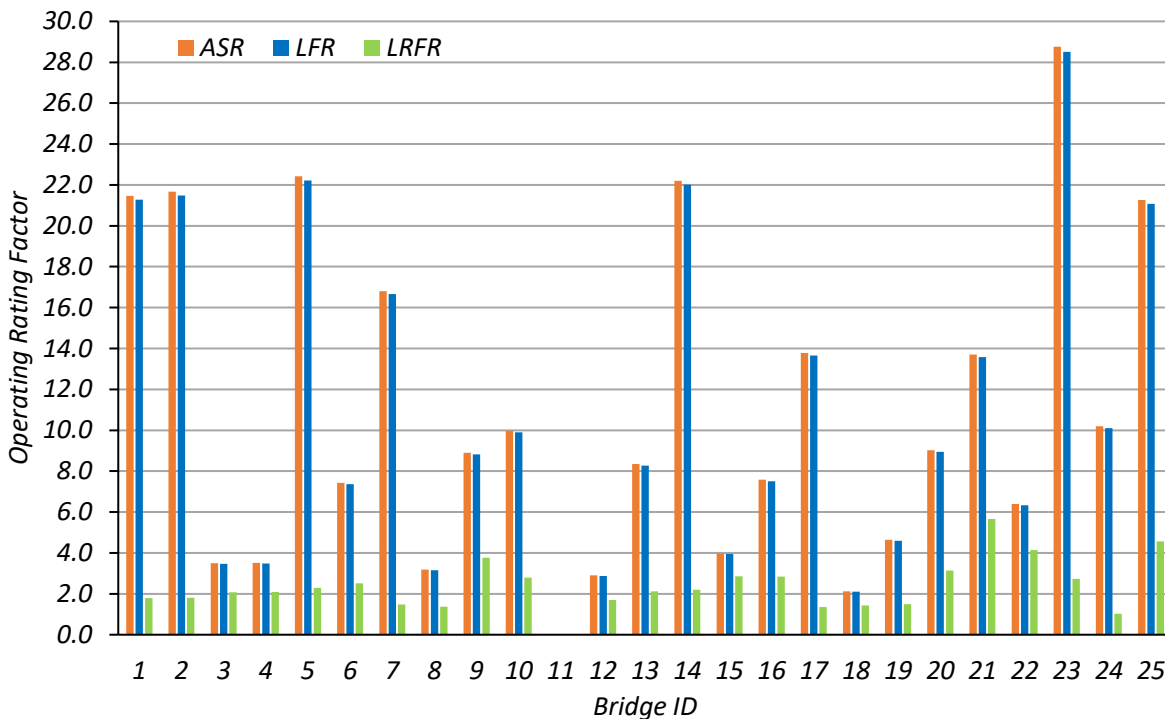


Figure 5.5. Exterior Girder Shear Operating RF for SM Bridges

For the exterior girder shear of the selected SM bridges, the ASR method produced a higher average inventory RF (8.22) than the LFR method (6.77), which produced a higher average inventory RF than the LRFR method (1.90). The LRFR shear RFs are significantly lower than the ASR and LFR RFs because the LRFR live load distribution factors for shear are much higher than the corresponding ASR and LFR live load distribution factors. The ASR method also produced a higher average operating RF (11.41) than the LFR method (11.31), which produced a higher average operating RF than the LRFR method (2.47). Bridge SM-11 does not have an exterior girder

shear RF because it was widened, and the exterior girder is now a prestressed concrete I-beam instead of a steel cross-section.

For all bridges, both the ASR method and the LFR method produced inventory and operating exterior girder shear RFs greater than or equal to 1.0. The LRFR method produced operating RFs greater than or equal to 1.0 for all bridges and an inventory RF less than 1.0 for only one bridge, SM-24, which had an inventory RF of 0.79. None of the exterior girder shear RFs control the corresponding exterior girder flexure RFs.

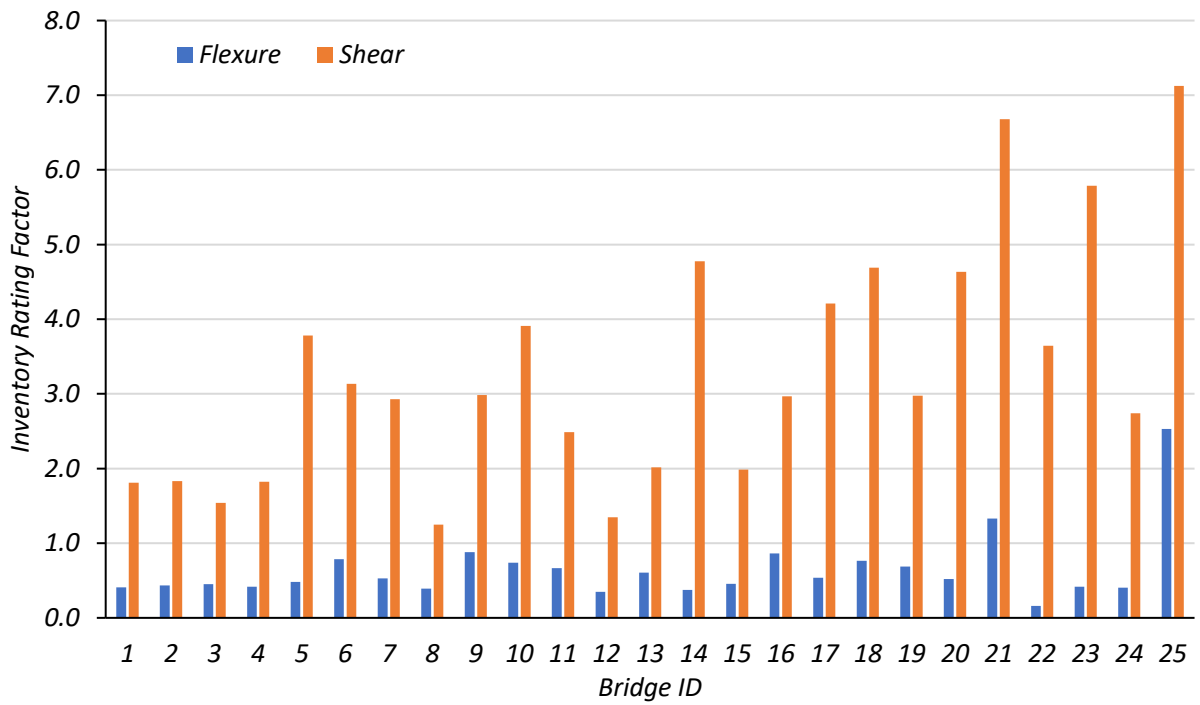
5.1.3.5 Comparison of SM Bridge Interior Girder Flexure and Shear RFs

After calculating the flexure and shear RFs for the selected SM bridges, the RFs were compared to determine if girder flexure or girder shear was the controlling limit state for each bridge. Table 5.7 and Figure 5.6 show the interior girder inventory and operating flexure and shear RFs found using the same method TxDOT uses in analysis. For every SM bridge, the flexure RF controlled the interior girder rating at both the inventory and operating level.

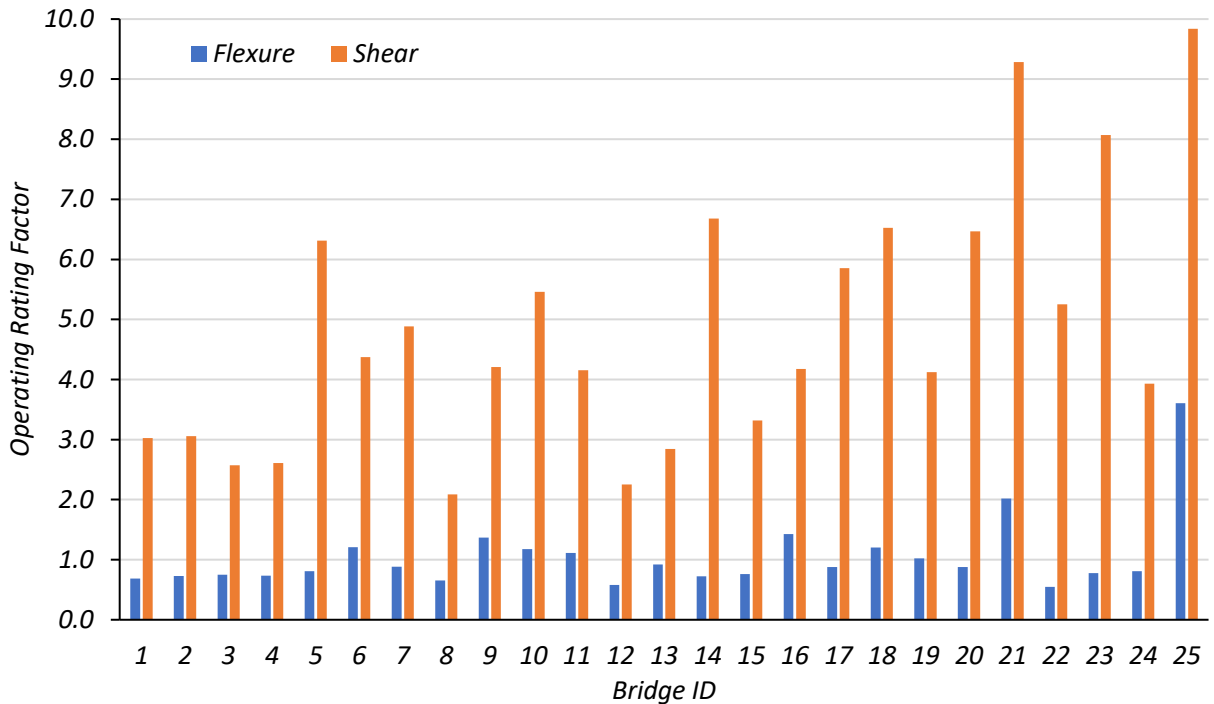
Table 5.7. Interior Girder Flexure and Shear RFs for SM Bridges

ID	TxDOT Rating Method	Inventory HS20 RF		Operating HS20 RF	
		Flexure	Shear	Flexure	Shear
SM-1	LFR	0.41	1.81	0.69	3.02
SM-2	LFR	0.44	1.83	0.73	3.06
SM-3	LFR	0.45	1.54	0.75	2.57
SM-4	ASR	0.42	1.82	0.73	2.61
SM-5	LFR	0.48	3.78	0.81	6.31
SM-6	ASR	0.78	3.13	1.21	4.37
SM-7	LFR	0.53	2.93	0.88	4.89
SM-8	LFR	0.39	1.25	0.65	2.09
SM-9	ASR	0.88	2.99	1.37	4.21
SM-10	ASR	0.74	3.91	1.17	5.46
SM-11	LFR	0.67	2.49	1.11	4.16
SM-12	LFR	0.35	1.35	0.58	2.25
SM-13	ASR	0.60	2.01	0.92	2.84
SM-14	ASR	0.38	4.78	0.72	6.68
SM-15	LFR	0.46	1.99	0.76	3.32
SM-16	ASR	0.86	2.97	1.43	4.18
SM-17	ASR	0.54	4.21	0.88	5.85
SM-18	ASR	0.76	4.69	1.20	6.53

SM-19	ASR	0.69	2.97	1.02	4.12
SM-20	ASR	0.52	4.64	0.88	6.47
SM-21	ASR	1.33	6.68	2.02	9.29
SM-22	ASR	0.16	3.64	0.55	5.25
SM-23	ASR	0.42	5.79	0.78	8.07
SM-24	ASR	0.40	2.74	0.81	3.93
SM-25	ASR	2.53	7.13	3.61	9.84
Avg.		0.65	3.32	1.05	4.85
Std. Dev.		0.45	1.58	0.61	2.08



(a) Inventory RFs



(b) Operating RFs

Figure 5.6. Comparison of Interior Girder Flexure and Shear RFs for SM Bridges

5.1.4 Check of SM Bridge Calculations

After conducting the rating analysis of each bridge, a check was conducted to confirm that the load rating process followed the AASHTO MBE and TxDOT practices. Therefore, for each bridge, the input characteristics were changed to match TxDOT’s values where differences were noted and the same standard practices employed by TxDOT were used. Any differences within 5 percent could be attributed to a slight rounding difference. Table 5.8 shows the corresponding results for each selected SM bridge.

There were only two SM bridges for which changing the input characteristics did not produce a result within 5 percent of TxDOT’s rating. For Bridge SM-2, this result was due to the calculated capacity being slightly higher than TxDOT’s capacity for the same steel cross-section. The underlying reason behind this seems to be that TxDOT does not have an entry for this shape in its rating program, and therefore section properties are calculated based on flange and web dimensions instead of taking them directly from a shape table. For Bridge SM-4, as previously mentioned, TxDOT does not provide rating calculations, so there were no results to compare.

Through this exercise, the calculated ratings were found to be close to TxDOT’s ratings, thus confirming the process used in the rating calculations for this project.

Table 5.8. Interior Girder RFs for SM Bridges with Adjusted Characteristics

Bridge ID	Rating Method	Revised Calculations HS20 RF		Revised Calculations RF/ TxDOT RF	
		Inventory	Operating	Inventory	Operating
SM-1	LFR	0.35	0.58	1.05	1.05
SM-2	LFR	0.36	0.60	1.07	1.08
SM-3	LFR	0.48	0.81	1.02	1.02
SM-4	ASR	0.43	0.73	1.11	1.31
SM-5	LFR	0.49	0.82	1.04	1.04
SM-6	ASR	0.94	1.46	1.05	1.05
SM-7	LFR	0.57	0.95	1.00	1.00
SM-8	LFR	0.41	0.68	0.99	0.99
SM-9	ASR	0.85	1.32	1.05	1.04
SM-10	ASR	0.79	1.22	1.02	1.02
SM-11	LFR	0.72	1.21	0.99	0.99
SM-12	LFR	0.51	0.85	1.00	0.99
SM-13	ASR	0.63	0.95	0.95	0.95
SM-14	ASR	0.42	0.76	1.02	1.02
SM-15	LFR	0.48	0.80	0.95	0.95
SM-16	ASR	0.80	1.22	0.99	0.99
SM-17	ASR	0.58	0.92	1.05	1.04
SM-18	ASR	0.59	0.94	1.01	0.99
SM-19	ASR	0.71	1.05	1.00	1.00
SM-20	ASR	0.57	0.93	1.02	1.02
SM-21	ASR	1.37	2.06	1.02	1.02
SM-22	ASR	0.36	0.85	0.96	0.99
SM-23	ASR	0.48	0.85	0.95	0.96
SM-24	ASR	0.59	1.00	1.00	1.01
SM-25	ASR	3.24	4.61	0.96	0.97
Avg.		0.71	1.13	1.01	1.02
Std. Dev.		0.56	0.77	0.04	0.07

5.1.5 Comparison of Calculated Ratings with TxDOT Ratings

5.1.5.1 General

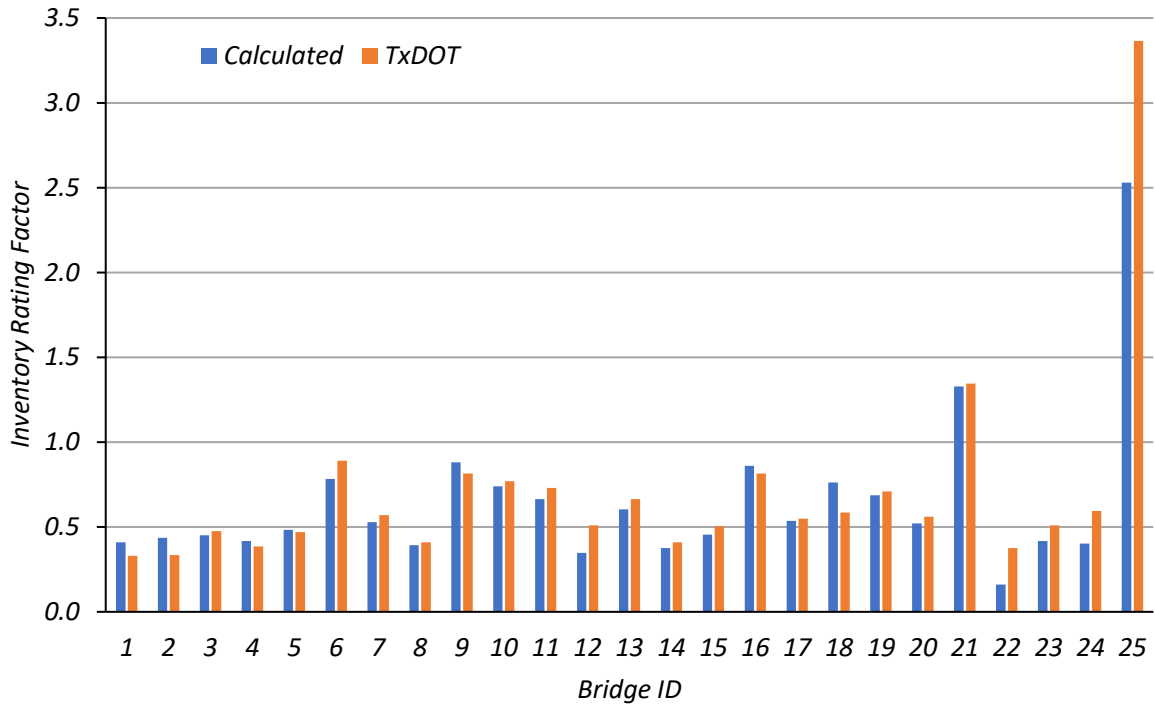
After confirming the rating process, the original calculated RFs were compared to the RFs that TxDOT documents in its inspection reports. The exterior girder was ignored in this analysis in order to compare corresponding calculated RFs to the TxDOT RFs since TxDOT does not analyze exterior girders in the rating calculations provided in its inspection reports.

Table 5.9 and Figure 5.7 show the TxDOT RFs for each bridge, the method it used, and the calculated RF (from this project) based on the same rating method (ASR, LRF, or LRFR). The only bridge that did not have a rating method reported in the NBI was Bridge SM-4. In the NBI, it was reported as “no rating analysis performed,” and in TxDOT’s inspection report, a note is written that says, “Original posting recommendation. No calculations to verify ratings.” For this bridge, the calculated ASR RF was used for the comparison.

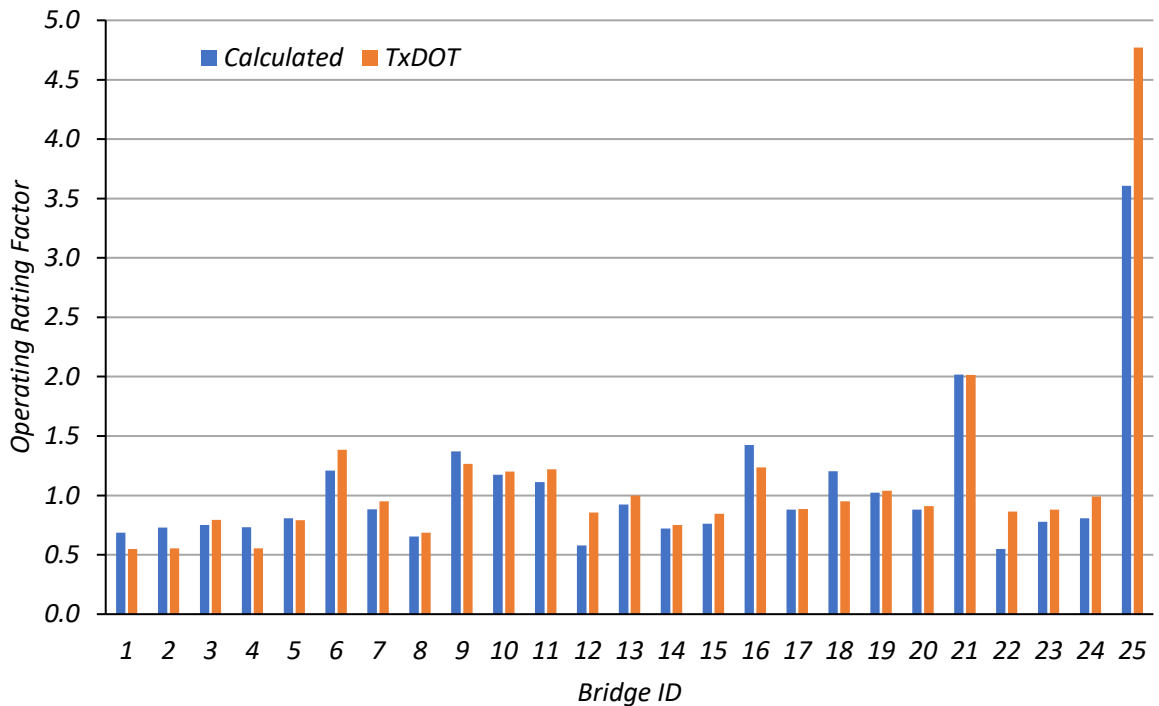
Of the 25 bridges whose interior girders were analyzed using the same method as TxDOT, seven had inventory and operating design-level ratings that were greater than or equal to TxDOT’s rating, and eight had operating design-level ratings that were greater than or equal to TxDOT’s rating. Of the bridges that had lower design-level ratings than TxDOT’s at the both the inventory and operating level, 12 had inventory and operating design-level ratings that were within 10 percent of TxDOT’s rating. For these bridges, the difference in ratings can usually be attributed to rounding differences or differences in the consideration of certain aspects related to loads. Of the bridges that had lower design-level ratings than TxDOT’s at both the inventory and operating levels, 14 had inventory and operating design-level ratings that were within 20 percent of TxDOT’s rating. In addition, four bridges had inventory or operating design-level ratings with more than a 20 percent difference relative to the TxDOT RF values.

Table 5.9. Comparison of Interior Girder Flexure RFs to TxDOT RFs for SM Bridges

ID	Calculated HS20 RF		TxDOT HS20 RF		Rating Method	Calculated/TxDOT	
	Inventory	Operating	Inventory	Operating		Inventory	Operating
SM-1	0.41	0.69	0.33	0.55	LFR	1.24	1.25
SM-2	0.44	0.73	0.34	0.56	LFR	1.30	1.31
SM-3	0.45	0.75	0.48	0.80	LFR	0.95	0.95
SM-4	0.42	0.73	0.39	0.56	N/A (ASR)	1.19	1.32
SM-5	0.48	0.81	0.47	0.79	LFR	1.03	1.02
SM-6	0.78	1.21	0.89	1.39	ASR	0.88	0.87
SM-7	0.53	0.88	0.57	0.95	LFR	0.93	0.93
SM-8	0.39	0.65	0.41	0.69	LFR	0.96	0.95
SM-9	0.88	1.37	0.82	1.27	ASR	1.08	1.08
SM-10	0.74	1.17	0.77	1.20	ASR	0.96	0.98
SM-11	0.67	1.11	0.73	1.22	LFR	0.91	0.91
SM-12	0.35	0.58	0.51	0.86	LFR	0.68	0.68
SM-13	0.60	0.92	0.67	1.00	ASR	0.91	0.92
SM-14	0.38	0.72	0.41	0.75	ASR	0.92	0.96
SM-15	0.46	0.76	0.51	0.85	LFR	0.90	0.90
SM-16	0.86	1.43	0.82	1.24	ASR	1.06	1.15
SM-17	0.54	0.88	0.55	0.89	ASR	0.98	0.99
SM-18	0.76	1.20	0.59	0.95	ASR	1.30	1.27
SM-19	0.69	1.02	0.71	1.04	ASR	0.97	0.98
SM-20	0.52	0.88	0.56	0.91	ASR	0.93	0.97
SM-21	1.33	2.02	1.35	2.02	ASR	0.99	1.00
SM-22	0.16	0.55	0.38	0.87	ASR	0.43	0.63
SM-23	0.42	0.78	0.51	0.88	ASR	0.82	0.88
SM-24	0.40	0.81	0.60	0.99	ASR	0.68	0.82
SM-25	2.53	3.61	3.37	4.77	ASR	0.75	0.76
Avg.	0.65	1.05	0.71	1.12	-	0.95	0.98
Std. Dev.	0.45	0.61	0.58	0.81	-	0.19	0.17



(a) Inventory RFs



(b) Operating RFs

Figure 5.7. Calculated RF vs. TxDOT RF for Interior Girder Flexure of SM Bridges

After the selected SM bridges were rated, the interior girder flexure RF for each bridge was analyzed to determine the source of any difference with respect to the RFs in the TxDOT inspection

reports. The load rating calculations were compared with the TxDOT calculations as closely as possible, given that in some cases TxDOT did not have complete calculations provided. The interior girder RFs that were obtained were used for comparison because TxDOT’s ratings are based on the interior girder. Table 5.10 shows each SM bridge considered and notes reasons why each bridge has a different interior girder RF than the RF determined by TxDOT.

Table 5.10. Reasons for Differences between Calculated and TxDOT Interior Girder Flexure RFs for SM Bridges

Bridge ID	Reasons for Difference in Rating Factor			
	Dead Load Components	Live Load Distribution Factors	Section Properties	Unknown
SM-1			x	
SM-2			x	
SM-3	x			
SM-4				x
SM-5				
SM-6	x	x		
SM-7	x			
SM-8	x			
SM-9				
SM-10	x			
SM-11	x			
SM-12	x	x		
SM-13	x			
SM-14	x			
SM-15	x			
SM-16			x	
SM-17	x			
SM-18		x		
SM-19	x			
SM-20	x			
SM-21				
SM-22	x	x		
SM-23	x			
SM-24	x			
SM-25	x	x		

For every bridge that exhibited a calculated RF lower than the TxDOT RF, the consideration of certain dead load components played into the difference. TxDOT only considers

the girders, concrete deck, and wearing surface in most of the dead load calculations. Some of these bridges have significant concrete curbs, lateral bracing members, or railings that are considered in the calculations developed in this project. The rating calculations include a best estimate of the weight of these components and distribute them to the girders according to the AASHTO MBE. Although these are not major components of the dead load of the structure, this process produced a higher dead load moment for almost every bridge. This difference affected every bridge overall and is the main reason for slight differences in the calculated and TxDOT RFs. This reason was in some cases paired with another reason to further increase the difference. This difference still applied to bridges that exhibited higher calculated RFs than the TxDOT RFs; however, some other difference in the calculations outweighed this item.

Another reason for differences in RFs for many of the bridges involves the braced length used for the non-composite moment capacity of the steel girder. In Article 6A.6.9.3 of the LRFR section of the MBE, AASHTO allows the top flange of a girder to be assumed to be fully braced if it is “fully in contact with the deck and no sign of cracking, rust, or separation along the steel-concrete interface is evident” (AASHTO 2018). However, no such clause exists in the ASR/LFR section of the MBE, and in Article 10.48.1.1 of the LFR part of the *AASHTO Standard Specifications*, which addresses compact section moment capacity, it states that “the required lateral bracing shall be provided by braces capable of preventing lateral displacement and twisting of the main members or by embedment of the top and sides of the compression flange in concrete.” No such article requiring embedment of the top flange exists in the ASR section for the girder to be fully braced. TxDOT assumes the girder to be fully braced by the deck when using ASR. In consideration of these articles, each bridge was analyzed considering the braced lengths provided by lateral members for LFR and assumed to be fully braced for ASR and LRFR. TxDOT, on the other hand, assumes the deck provides full bracing in almost all cases. This difference causes TxDOT ratings to use a slightly higher moment capacity for LFR ratings; however, it does not affect the controlling rating for any of the bridges for which LFR is used by TxDOT because the serviceability limit state controls the rating.

Several bridges that displayed larger differences in the RFs were further examined. Table 5.11 shows the main reason for the difference in RF for every simple-span bridge that had an RF greater than 10 percent lower than the TxDOT RF. Table 5.12 shows the main reason for the

difference in RF for every simple-span bridge that had an RF greater than 10 percent higher than TxDOT’s RF.

Table 5.11. SM Bridges with Lower Calculated Interior Girder Flexure RFs—Main Reason for Differences

Main Reason for Difference	Number of Bridges	Bridge IDs
Live Load Distribution Factors	4	SM-6, SM-12, SM-22, SM-25
Consideration of Components for Dead Load	2	SM-23, SM-24

Table 5.12. SM Bridges with Higher Calculated Interior Girder Flexure RFs—Main Reason for Differences

Main Reason for Difference	Number of Bridges	Bridge IDs
Section Properties	3	SM-1, SM-2, SM-16
Live Load Distribution Factors	1	SM-18
Unknown—calculations not provided	1	SM-4

The following subsections describe the differences observed between the calculated ratings and TxDOT ratings for each bridge that exhibited a difference of 10 percent or greater.

5.1.5.2 Comparison of Ratings for Bridge SM-1

TxDOT uses a 6.5 in. thick slab instead of the 5.5 in. thick slab that is used in the calculations. The drawings attached to this bridge show a 5.5 in. thick slab, so this value was selected. TxDOT has a higher dead load moment, thereby decreasing its rating. TxDOT also states in its bridge inventory record, dated May 16, 2018, that the interior girder steel cross-section is a WF(CB)21x58, however the load rating calculations dated March 31, 2004, use a WF21x55. Based on the more recent record, the calculations use a WF(CB)21x58 as the steel cross-section. Although it is not a significantly different cross-section, it still increases the capacity and therefore increases the RF.

5.1.5.3 Comparison of Ratings for Bridge SM-2

TxDOT also uses a 6.5 in. thick slab for this bridge instead of the 5.5 in. thick slab used in the calculations. The drawings attached to this bridge show a 5.5 in. thick slab, so this value was selected. TxDOT therefore has a higher dead load moment, thereby decreasing its rating.

5.1.5.4 Comparison of Ratings for Bridge SM-4

TxDOT does not provide rating calculations for Bridge SM-4. It only provides posting recommendations dated June 15, 1997, and a statement dated February 17, 2015, saying, “Original posting recommendation. No calculations to verify ratings.” With no calculations to compare to, the basis for the higher TxDOT RFs for Bridge SM-4 are not known.

5.1.5.5 Comparison of Ratings for Bridges SM-6, SM-12, SM-22, and SM-25

For Bridges SM-6, SM-12, SM-22, and SM-25, TxDOT uses a one-lane distribution factor because these bridges are narrow, having a roadway width varying between 20 ft 5 in. and 24 ft 0 in. Based on these roadway widths and physical inspections, TxDOT is assuming that only one truck will travel on the bridge at a time, and it is analyzed accordingly. However, the structural drawings indicate two design lanes, which is supported by the inspection reports and photos. Article 3.6.3 of the *AASHTO Standard Specifications* and Article 3.6.1.1.1 of the *AASHTO LRFD Specifications* both state that roadway widths between 20 ft and 24 ft shall have two design lanes, each equal to one half of the roadway width. TxDOT follows these articles when stating the number of lanes on its bridge inventory sheets; however, when conducting its load rating calculations, the number of lanes on the bridge is reduced in some cases. However, the calculations in this report consider these bridges as two-lane bridges. This produces higher LLDFs, and therefore lower RFs than TxDOT uses for these bridges.

5.1.5.6 Comparison of Ratings for Bridge SM-16

TxDOT uses a steel yield strength of 36 ksi; however, in its bridge inventory record, a note states that the yield strength is 50 ksi per city records. Therefore, a yield strength of 50 ksi was used in the calculations. This change increases the capacity of the bridge; however, TxDOT also uses a different composite section and composite section procedure than the rating calculations follow, thereby giving a larger section modulus and reducing this difference somewhat.

5.1.5.7 Comparison of Ratings for Bridge SM-18

TxDOT evenly distributed both wheel lines to the four main girders of this bridge, giving each girder one-half of a wheel line. The *AASHTO Standard Specifications* LLDFs produce a lower

distribution factor for the interior girder than one-half of a wheel line. This difference decreases the live load moment applied to the interior girder and increases the RF. Although this result produces a larger calculated difference for the interior girder, it also causes the exterior girder to control the calculations because greater than one-half of a wheel line is distributed to the exterior girder through the lever rule. The exterior girder controls the calculated ratings such that the calculated exterior girder rating is lower than the TxDOT rating for this particular bridge.

5.1.5.8 Comparison of Ratings for Bridges SM-23 and SM-24

When load rating bridges, TxDOT considers the interior girders and does not include some components that can add a significant dead load moment in the calculations, such as curbs, railings, and lateral bracing members. For Bridge SM-23, TxDOT does not consider the deck form, lateral bracing, or railing. The project calculations also consider a 2 percent section loss due to corrosion mentioned in a 2017 inspection report, but TxDOT calculations do not consider this loss. For Bridge SM-24, TxDOT does not consider these same features and does not consider a 12 in. timber crane mat used as a wearing surface. In the TxDOT load rating calculations dated August 19, 2013, a note is written saying, “Ignore new timber girders for analysis (see attached email).” However, no attachment is provided. TxDOT’s bridge inventory record dated September 2, 2017, lists a 12 in. crane mat as a wearing surface, and photos dated June 2, 2017, show a timber wearing surface present. Thus, a timber wearing surface was included in the reported rating calculations. For both of these bridges, TxDOT calculates a lower dead load moment, which raises the corresponding RFs of the bridges.

5.1.6 Conclusion

The girders of 25 simple-span steel bridges with concrete decks were analyzed according to the *AASHTO MBE*, *AASHTO Standard Specifications*, and *AASHTO LRFD Specifications*. The exterior girder was initially not considered in order to compare RFs of the same structural components to TxDOT. The LFR and ASR methods produced significantly higher RFs than the LRFR method, with the LFR method being slightly higher than the ASR method. Most bridges were calculated to have RFs within 10 percent of the corresponding TxDOT RFs, with the differences being attributed to differences in the dead loads considered for analysis. These differences are described herein. If bridges exhibited calculated RFs having greater than a

10 percent difference with the TxDOT RFs, the reason for this discrepancy was examined further. These reasons were expanded upon and justified. Based on the analysis performed on the selected simple-span bridges, possible areas for improvement to load posting are identified and elaborated upon in Section 5.3 of this report.

5.2 CONTINUOUS STEEL MULTI-GIRDER BRIDGES

5.2.1 Introduction

Sixteen continuous steel multi-girder (SC) bridges with concrete decks were selected from the load posted bridge inventory of Texas for further analysis. Table 5.13 summarizes these bridges and their main characteristics.

The information shown in this table contains the characteristics that pertain to the controlling span of the bridge. In order to obtain comparable ratings, this same process was used in the calculated rating. In this table, the girder spacing shown is the largest interior girder spacing in the span, if all girders are not equally spaced. Some of the girder cross-sections are historical sections and are named accordingly. A girder named W###x###, S###x###, or C#x### is a current cross section in the 14th edition of the *AISC Steel Construction Manual* (AISC 2013). A girder named ##Wf# is a historic section found in AISC’s *Historic Record, Dimensions and Properties, Rolled Shapes, Steel and Wrought Iron Beams & Columns* (AISC 1953). Detailed characteristics for the selected bridges were compiled from the TxDOT inspection database into a master spreadsheet. This process enabled the efficient collection of bridge properties and other relevant information so that the basic load ratings could be conducted.

Table 5.13. Selected SSLO Continuous Steel Multi-Girder Bridges and Characteristics

ID	On/Off System	Year Built	Span Lengths (ft)	Deck Width (ft)	Steel Yield Strength ² (ksi)	Interior Girder Section	Exterior Girder Section	Interior Girder Spacing (ft)	Concrete Deck Thickness (in.)	Wearing Surface Thickness (in.)
SC-1 ¹	Off	1910	19' – 18'	19	30	S6x17.25	W6x9	2'-4"	5.50	0.0
SC-2	Off	1925	26' – 26'	14	30	S12x31.8	S12x31.8	3'-7"	3.00	0.0
SC-3	Off	1935	18' – 18'	18	30	S10x25.4	S10x25.4	1'-11.5"	4.00	5.0
SC-4	Off	1937	18' – 15'	18	33	S10x25.4	S10x25.4	2'-2"	4.75	5.8
SC-5 ¹	Off	1940	21' – 21'	23	33	S10x25.4	S10x25.4	5'-6.5"	6.25	0.0
SC-6	Off	1942	23' – 22'	24	33	S12x31.8	B15x39	2'-10.5"	5.50	0.0

SC-7	Off	1943	12' – 11'	24	30	S9x21.8	C9x13.4	2'-5"	7.00	7.0
SC-8	On	1949	60' – 70' – 60'	26	33	30WFx108	30WFx108	7'-0"	6.00	1.0
SC-9	Off	1950	11' – 11'	21	33	S6x12.5	S6x12.5	2'-0"	6.00	0.0
SC-10 ³	On	1950	21'	34	33	27WFx94	27WFx94	10'-0"	7.50	0.0
SC-11	Off	1955	15' – 19' – 19'	21	33	S6x12.5	C6x8.2	1'-8"	5.50	0.0
SC-12	On	1959	60' – 75' – 60'	26	33	30WFx108	30WFx108	6'-8"	6.00	2.0
SC-13	On	1965	60' – 75' – 60'	28	36	W30x108	W30x108	7'-0"	6.50	0.0
SC-14	Off	1970	36' – 19'	14	30	S9x21.8	S9x21.8	2'-8"	6.00	0.0
SC-15	Off	1997	24' – 24'	24	36	16WFx88	16WFx88	4'-3.5"	6.00	0.0
SC-16	Off	1999	39' – 37'	20	36	W27x94	W27x94	3'-3"	10.00	0.0

Notes:

1. SC-1 and SC-5 are the only bridges in this group that are specifically noted as composite in TxDOT's inspection records.
2. Steel yield strength is the value used by TxDOT for load rating calculations, which typically matches the value given in the *TxDOT Bridge Inspection Manual* (TxDOT 2018a) and AASHTO MBE (AASHTO 2018) unless additional information is available.
3. Floor beam characteristics are shown for SC-10.

5.2.2 Basic Load Rating Analysis

5.2.2.1 General

All the selected continuous steel multi-girder bridges were rated using ASR, LFR, and LRFR. All three methods follow the same general procedure as described in Section 5.1.2. For the flexure rating factors, the following steps are used.

1. Modified section properties are calculated if the girder has a cover plate in the positive moment region or if it is a composite girder. Composite section properties are calculated using the same procedure as in Example A1 in MBE Appendix A (AASHTO 2018). This procedure finds two different transformed section moduli: one using the width of the transformed concrete deck based on the modular ratio of the steel to concrete elastic moduli ($n=E_s/E_c$) and one using the width of the transformed concrete deck based on $3n$. The section properties based on n are used when considering the live load on the structure, and the section properties based on $3n$ are used when considering the superimposed dead load on the structure. This process is practiced in the rating factor equations for each method.
2. The appropriate LLDFs are determined for both the positive and negative moment regions. For ASR and LFR, LLDFS are given as the girder spacing over a number based on the number of lanes for interior girders. For exterior girders, the lever rule is used. For LRFR,

LLDFs are determined as a function of the spacing of the girders and have a range of applicability based on the spacing of the girders, span length, thickness of the slab, and number of girders or determined by using the lever rule. For LRFR, the average distribution factor for adjacent spans is used as the negative moment distribution factor. Skew adjustments are also considered.

3. The live load distribution factor, along with the appropriate impact factor, is applied to the positive and negative live load moments that are obtained for a single girder of the bridge. For ASR and LFR, the impact factor is given in Article 3.8.2.1 of the *AASHTO Standard Specifications* and is a function of the span length. For LRFR, the impact factor is always 33 percent. Both interior and exterior girders are examined and analyzed.
4. The dead load moments are found by calculating a tributary-distributed load for each girder and applying the continuous beam coefficients based on the number of spans and maximum span length (AISC 2013).
5. The appropriate non-composite or composite moment capacity for positive and negative flexure for both the interior and exterior girders is calculated. Consideration is given to whether the section is compact or noncompact, following the requirements in the respective AASHTO specifications. For ASR, this capacity is given in the form of a stress, as a percentage of the yield stress, from the AASHTO MBE. For the LFR capacity, if the section meets the compact requirements, Equation (5.1) is used. If the section does not meet the compact requirements but does meet the noncompact requirements, Equation (5.2) is used. If the section meets neither the compact nor the noncompact requirements, Equation (5.3) is used. For LRFR, if the section meets the compact requirements, Equation (5.4) is used. If the section does not meet the compact requirements but does meet the noncompact requirements, Equation (5.6) is used. The section is also checked using equations based on the braced length of the span. For a composite section for both methods, the plastic neutral axis is determined, and the moments caused by the components are summed about the plastic neutral axis.
6. Load rating is conducted based on the three load rating procedures (ASR, LFR, and LRFR) at the operating and inventory levels, considering both the interior girder and exterior girder positive and negative moment demands at the strength and service limit states.

For the shear rating factors, the following steps are used:

1. Since the web of the steel section is the shear resisting component, section properties such as web thickness, web height, and web area are found and calculated.
2. The appropriate LLDFs are determined. For ASR and LFR, LLDFs are given as the girder spacing over a number based on the number of lanes for interior girders. For exterior girders, the lever rule is used. For LRFR, LLDFs are determined as a function of the spacing of the girders and have a range of applicability based on the spacing of the girders, span length, thickness of the slab, and number of girders or determined by using the lever rule. Skew adjustments are also considered.
3. The live load distribution factor, along with the appropriate impact factor, are applied to the live load shear that is obtained for the bridge. For ASR and LFR, the impact factor is given in Article 3.8.2.1 of the *AASHTO Standard Specifications* and is a function of the span length. For LRFR, the impact factor is always 33 percent. Both interior and exterior girders are examined and analyzed.
4. The dead load shears are found by calculating a tributary-distributed load for each girder and applying the approximate continuous beam coefficients based on the number of spans and maximum span length (AISC 2013).
5. The web buckling coefficient is determined, and the shear capacity is determined using Equation (5.6).
6. Load rating is conducted based on the three load rating procedures (ASR, LFR, and LRFR) at the operating and inventory levels by considering both the interior girder and exterior girder shear demands at the strength limit state.

Some bridges had special considerations that had to be made when conducting the load rating analysis. Bridges SC-1 and SC-5 have their top flanges embedded into the concrete deck, making them composite sections. Their composite section properties were calculated based on provided structural drawings. However, TxDOT assumed that the neutral axis was located at the surface of the top flange and ignored the part of concrete below the top surface of the flange when calculating moment capacity. The moment capacity calculations use the same approach to have comparable ratings. However, when the ASR method and the LFR and LRFR serviceability limit states are checked, the section modulus used to find stresses must be the actual section modulus of the cross-section, and for consistency, the same procedure that is used in Example A1 in MBE Appendix A (AASHTO 2018) is used. Therefore, the composite section moduli for SC-1 and SC-5

were calculated separately and used for these ratings. Bridge SC-10 involved the analysis of a floor beam and not the girders of the bridge. Bridges SC-12 and SC-13 have top and bottom cover plates in the negative moment region, for which extra section property calculations were conducted. Bridge SC-14 is labeled as continuous; however, it contains two different-sized steel cross-sectional shapes that are spliced and bolted at the interior support. TxDOT, in its rating calculations, says this is not a true moment connection and analyzes the bridge as a simple span. For this reason, the bridge was analyzed as a simple-span bridge in the calculations. Finally, Bridges SC-15 and SC-16 each have one or more girders that do not touch the interior support and therefore act as simply supported over the entire bridge length. TxDOT analyzes both of these bridges as a single simply supported span, and therefore the project calculations analyzed them in the same way.

5.2.2.2 *Allowable Stress Rating and Load Factor Rating*

The capacities of continuous steel multi-girder bridges were determined using the same process as for the capacity calculations for the simple-span steel multi-girder bridges, with the addition of the negative moment capacity calculations. For a non-composite bridge, the negative moment capacity is taken as the same as the non-composite positive moment capacity unless the bridge has a negative moment region cover plate. For a composite bridge, the *AASHTO Standard Specifications* allow the rebar contribution to be considered in the negative moment region, per Article 10.50.2.1 (AASHTO 2002). Therefore, for composite cross-sections, a composite negative moment section modulus and moment capacity were calculated and used in determination of the negative moment region RFs. The RFs are calculated in the same way as the simple-span RFs, with the consideration of the negative moment region as well.

5.2.2.3 *Load and Resistance Factor Rating*

As with the ASR and LFR continuous calculations, the LRFR continuous calculations determined the capacities using the same process as for the simple-span bridges, with the addition of the negative moment region moment capacity calculations. However, the *AASHTO LRFD Specifications* do not have a clause allowing for the consideration of composite rebar in the negative moment region, so this factor was not included. Also, for negative moment region LLDFs, the *AASHTO LRFD Specifications* require the averaging of the two adjacent span lengths when

calculating the distribution factors. This process was conducted in the LRFR continuous bridge rating calculations.

5.2.3 Calculated Load Rating Results

Each of the 16 selected continuous steel multi-girder bridges with concrete decks was rated using each of the three rating methods. The bridges were rated for an HS-20 truck for ASR and LFR since that is the truck TxDOT uses when reporting to the NBI and for the HL-93 design load for LRFR. They were examined and rated for interior girder flexure, exterior girder flexure, interior girder shear, and exterior girder shear. The bridges were initially rated by following the procedures laid out in the *AASHTO MBE*, *AASHTO Standard Specifications*, and *AASHTO LRFD Specifications* as closely as possible.

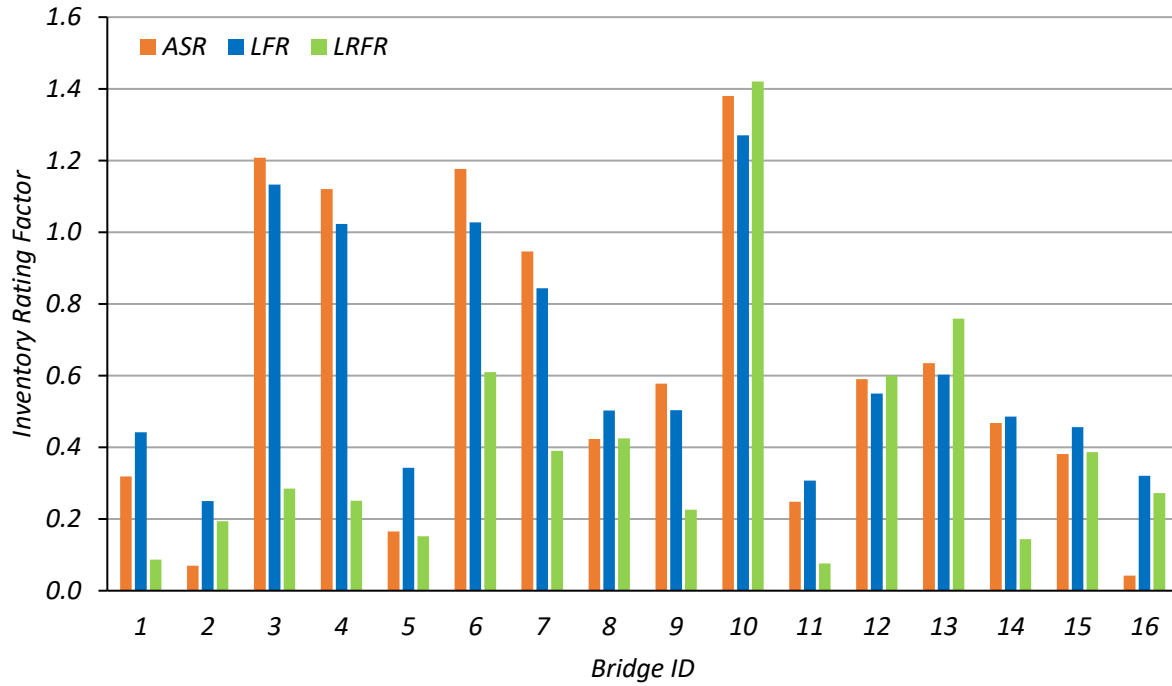
5.2.3.1 Interior Girder Flexure Analysis

The flexure RFs found for the design loads using each method for the interior girders of each bridge are shown in Table 5.14 and Figure 5.8. In the interior girder flexure analysis, for all of the selected bridges except SC-2, the LFR method produced a higher operating RF than the ASR method. For all of the selected bridges, the LFR method produced a higher operating RF than the LRFR method.

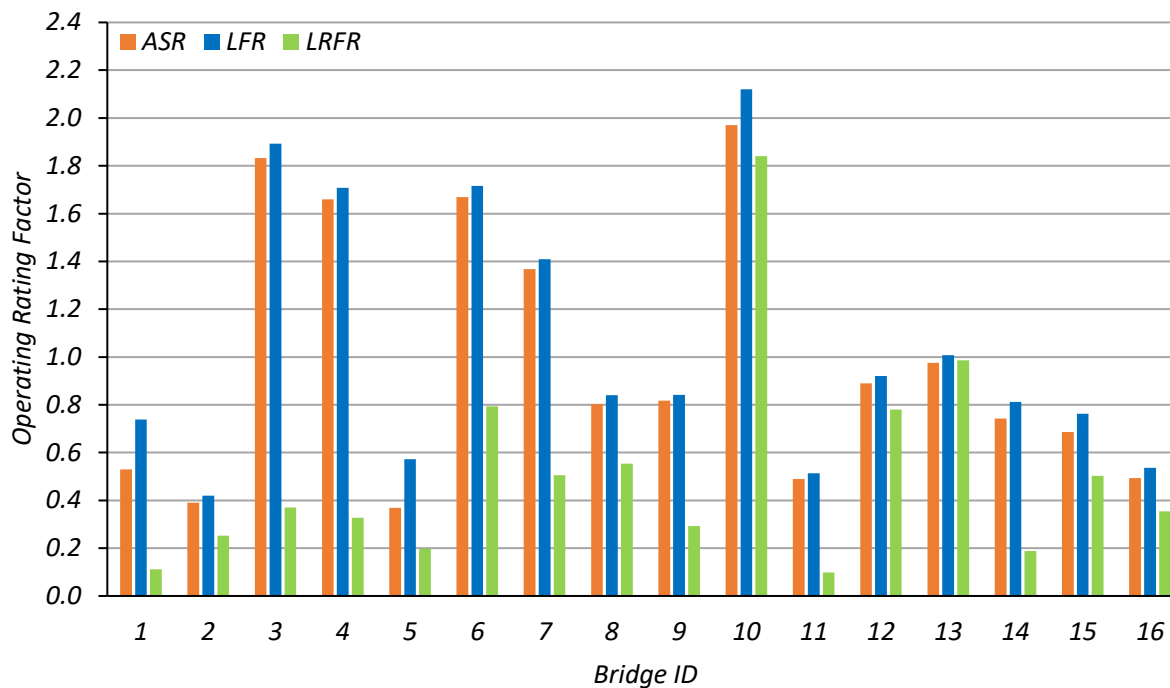
Table 5.14. Interior Girder Flexure RFs for SC Bridges

ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SC-1	0.32	0.44	0.09	0.53	0.74	0.11
SC-2	0.07	0.25	0.19	0.39	0.42	0.25
SC-3	1.21	1.13	0.29	1.83	1.89	0.37
SC-4	1.12	1.02	0.25	1.66	1.71	0.33
SC-5	0.17	0.34	0.15	0.37	0.57	0.20
SC-6	1.18	1.03	0.61	1.67	1.72	0.79
SC-7	0.95	0.84	0.39	1.37	1.41	0.51
SC-8	0.42	0.50	0.43	0.80	0.84	0.55
SC-9	0.58	0.50	0.23	0.82	0.84	0.29
SC-10	1.38	1.27	1.42	1.97	2.12	1.84
SC-11	0.25	0.31	0.08	0.49	0.51	0.10
SC-12	0.59	0.55	0.60	0.89	0.92	0.78
SC-13	0.64	0.60	0.76	0.98	1.01	0.99
SC-14	0.47	0.49	0.14	0.74	0.81	0.19
SC-15	0.38	0.46	0.39	0.69	0.76	0.50

SC-16	0.04	0.32	0.27	0.49	0.54	0.35
Average	0.61	0.63	0.39	0.98	1.05	0.51
Std. Dev.	0.42	0.31	0.33	0.52	0.52	0.43



(a) Inventory RFs



(b) Operating RFs

Figure 5.8. Interior Girder Flexure RFs for SC Bridges

For the interior girder flexure of the selected SC bridges, the LFR method produced a higher average inventory RF (0.63) than the ASR method (0.61), which produced a higher average inventory RF than the LRFR method (0.39). The LFR method also produced a higher average operating RF (1.05) than the ASR method (0.98), which produced a higher average operating RF than the LRFR method (0.51).

Of the 16 bridges whose interior girders were analyzed, four had HS-20 inventory ratings at or above 1.0, and five had HS-20 operating ratings at or above 1.0 using both the ASR method and the LFR method. One had an HL-93 inventory rating at or above 1.0 and the same one had an HL-93 operating rating at or above 1.0 using the LRFR method. This bridge was SC-10. Three bridges along with this one had both operating and inventory ratings above 1.0 using both the ASR and LFR method. These bridges were SC-3, SC-4, and SC-6.

5.2.3.2 Exterior Girder Flexure Analysis

As with the simple-span bridges, the exterior girders of the continuous bridges were rated separately since it appears that the TxDOT practice is to rate the SC bridges based on the interior

girders only. However, each bridge was analyzed again, this time considering the exterior girders and the AASHTO articles relevant to them, with the main differences again being the LLDFs. Table 5.15 and Figure 5.9 show the exterior girder flexure RFs found using each rating method for each continuous SC bridge. Because the rating for Bridge SC-10 consisted of only analyzing a floor beam, the exterior girder RF shown is the same as the interior girder RF, which is the singular RF for the floor beam.

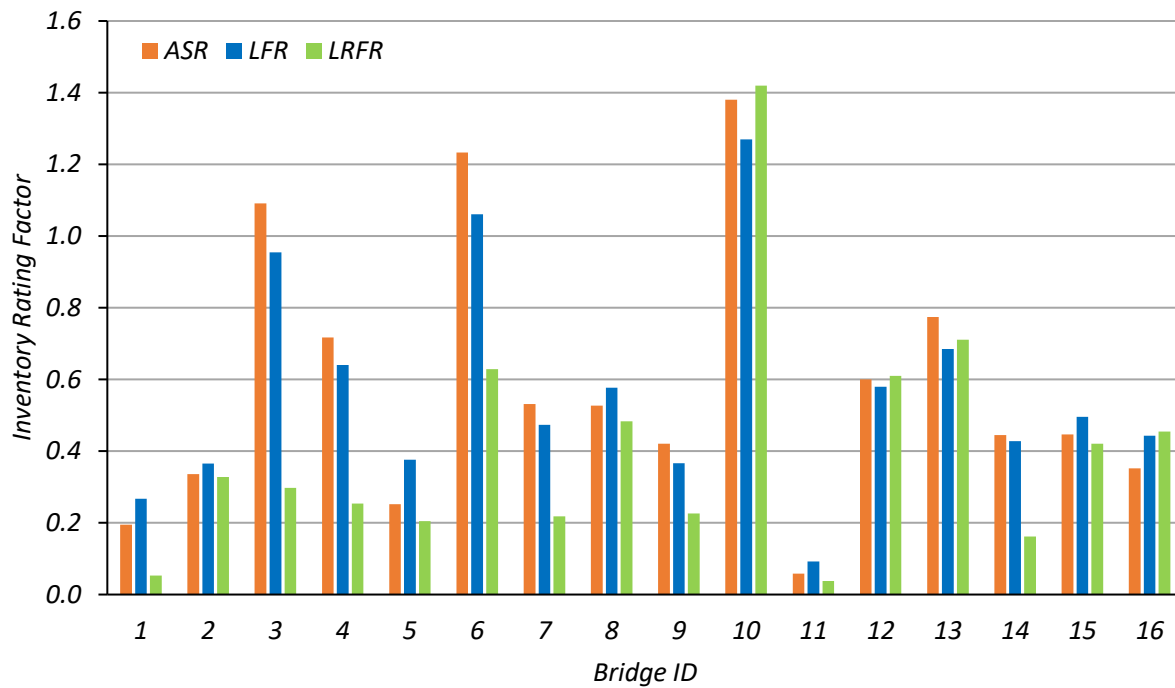
For the exterior girder flexure of the selected SC bridges, the ASR method produced a higher average inventory RF (0.58) than the LFR method (0.57), which produced a higher average inventory RF than the LRFR method (0.41). However, the LFR method produced a higher average operating RF (0.95) than the ASR method (0.89), which produced a higher average operating RF than the LRFR method (0.53).

There were seven continuous bridges for which the exterior girder controlled the rating. Table 5.16 and Figure 5.10 show the operating and inventory design-level RFs found for each of these bridges when the exterior was and was not considered and the difference between the two RFs.

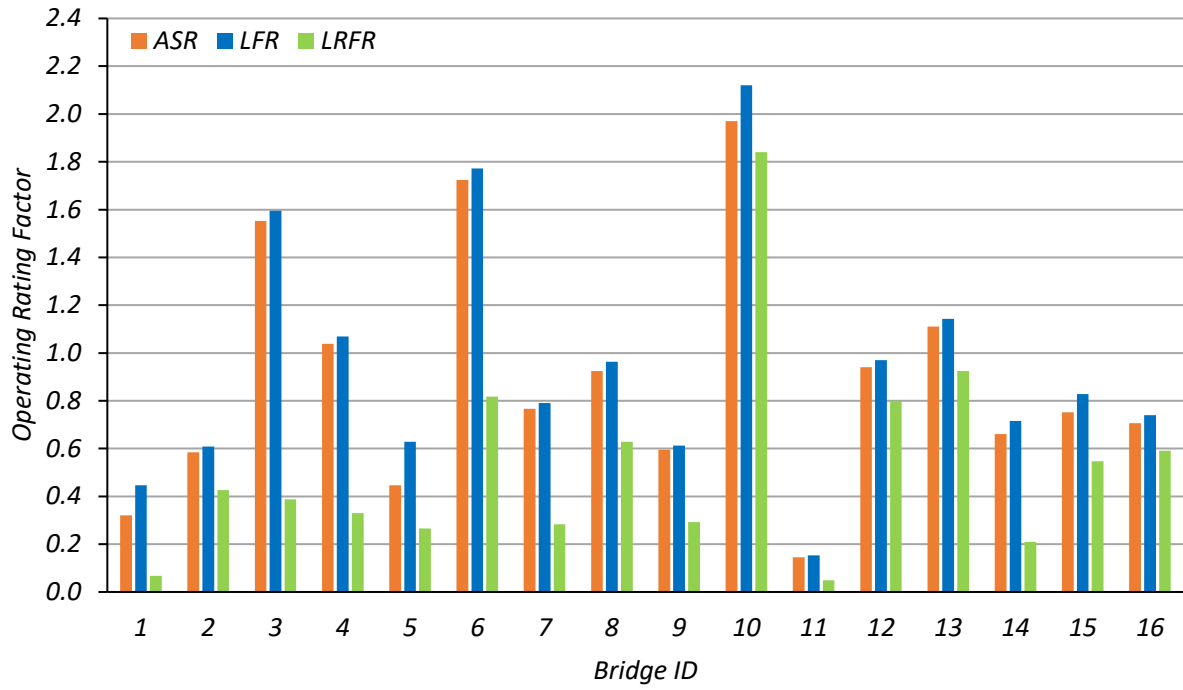
Table 5.15. Exterior Girder Flexure RFs for SC Bridges

ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SC-1	0.20	0.27	0.05	0.32	0.45	0.07
SC-2	0.34	0.37	0.33	0.59	0.61	0.43
SC-3	1.09	0.96	0.30	1.55	1.60	0.39
SC-4	0.72	0.64	0.25	1.04	1.07	0.33
SC-5	0.25	0.38	0.21	0.45	0.63	0.27
SC-6	1.23	1.06	0.63	1.72	1.77	0.82
SC-7	0.53	0.47	0.22	0.77	0.79	0.28
SC-8	0.53	0.58	0.48	0.93	0.96	0.63
SC-9	0.42	0.37	0.23	0.60	0.61	0.29
SC-10	1.38	1.27	1.42	1.97	2.12	1.84
SC-11	0.06	0.09	0.04	0.15	0.15	0.05
SC-12	0.75	0.67	0.70	1.08	1.12	0.91
SC-13	0.77	0.69	0.71	1.11	1.14	0.92
SC-14	0.45	0.43	0.16	0.66	0.72	0.21
SC-15	0.45	0.50	0.42	0.75	0.83	0.55
SC-16	0.35	0.44	0.46	0.71	0.74	0.59
Average	0.58	0.57	0.41	0.89	0.95	0.53
Std. Dev.	0.36	0.29	0.32	0.48	0.49	0.42

It should be noted, however, that the results obtained when considering an exterior girder may be overly conservative. For Bridges SC-1, SC-3, SC-7, SC-11, and SC-14, the exterior girder control due to restrictions in Article 3.23.2.3.1.5 of the *AASHTO Standard Specifications*. As mentioned with the simple-span bridges, this article puts a minimum on the exterior girder live load distribution factor, thereby decreasing the exterior girder rating. For Bridges SC-4, SC-9, and SC-13, the lever rule produces higher exterior distribution factors that outweigh the reduction in dead load the exterior girder experiences. Nevertheless, the bridges were analyzed according to the *AASHTO Standard Specifications* and the lever rule.



(a) Inventory RFs



(b) Operating RFs

Figure 5.9. Exterior Girder Flexure RFs for SC Bridges

Table 5.16. SC Bridges Controlled by Exterior Girder Flexure RFs

ID	TxDOT Rating Method	Interior Girder Rating		Exterior Girder Rating		Exterior Rating/ Interior Rating	
		Inventory	Operating	Inventory	Operating	Inventory	Operating
SC-1	ASR	0.32	0.53	0.20	0.32	0.61	0.61
SC-3	ASR	1.21	1.83	1.09	1.55	0.90	0.85
SC-4	ASR	1.12	1.66	0.72	1.04	0.64	0.63
SC-7	ASR	0.95	1.37	0.53	0.77	0.56	0.56
SC-9	ASR	0.58	0.82	0.42	0.60	0.73	0.73
SC-11	ASR	0.25	0.49	0.06	0.15	0.23	0.30
SC-14	ASR	0.47	0.74	0.45	0.66	0.95	0.89
Average		0.70	1.06	0.49	0.73	0.66	0.65

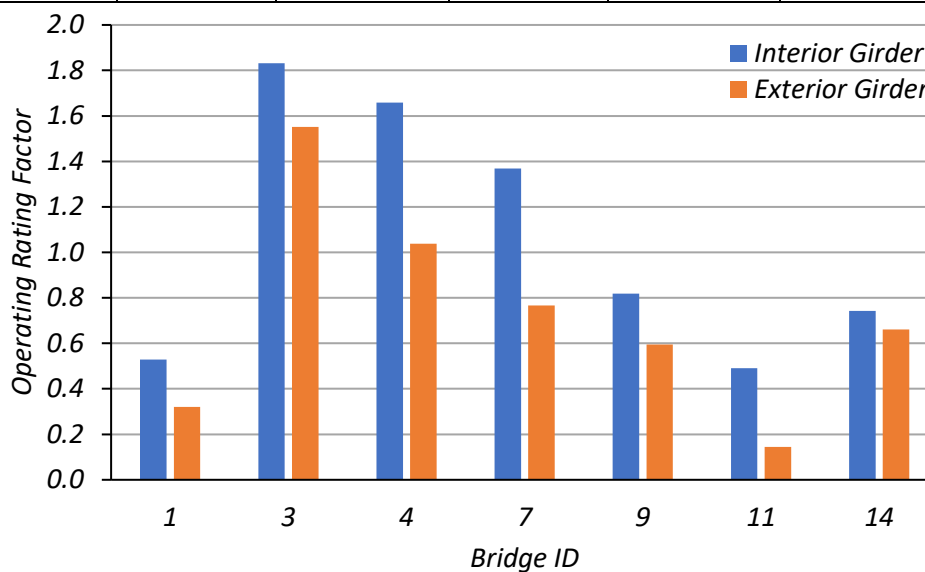


Figure 5.10. Interior Girder and Exterior Girder RFs for Select SC Bridges

5.2.3.3 Interior Girder Shear Analysis

The shear RFs found for the design loads using each method for the interior girders of each continuous bridge are shown in Table 5.17 and Figure 5.11. In the interior girder shear analysis, for all of the selected bridges, the ASR method produced a higher operating RF than both the LFR and LRFR methods.

Table 5.17. Interior Girder Shear RFs for SC Bridges

ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SC-1	1.94	1.61	0.70	2.72	2.69	0.91
SC-2	1.79	1.56	0.87	2.63	2.60	1.13
SC-3	3.89	3.24	0.94	5.45	5.40	1.21
SC-4	3.13	2.62	0.94	4.41	4.37	1.22

SC-5	0.78	0.69	0.59	1.17	1.15	0.76
SC-6	2.41	2.00	1.21	3.37	3.33	1.57
SC-7	2.17	1.84	0.68	3.10	3.07	0.88
SC-8	2.71	2.32	2.11	3.91	3.87	2.74
SC-9	1.66	1.38	0.50	2.32	2.30	0.64
SC-10	3.94	3.26	3.83	5.48	5.43	4.97
SC-11	1.98	1.67	0.42	2.81	2.78	0.55
SC-12	2.96	2.52	2.10	4.25	4.21	2.73
SC-13	3.20	2.69	2.28	4.53	4.49	2.95
SC-14	2.24	1.87	0.79	3.15	3.12	1.03
SC-15	2.61	2.19	1.79	3.69	3.66	2.32
SC-16	6.98	5.88	2.92	9.91	9.82	3.79
Average	2.77	2.33	1.42	3.93	3.89	1.84
Std. Dev.	1.34	1.13	0.96	1.90	1.88	1.24

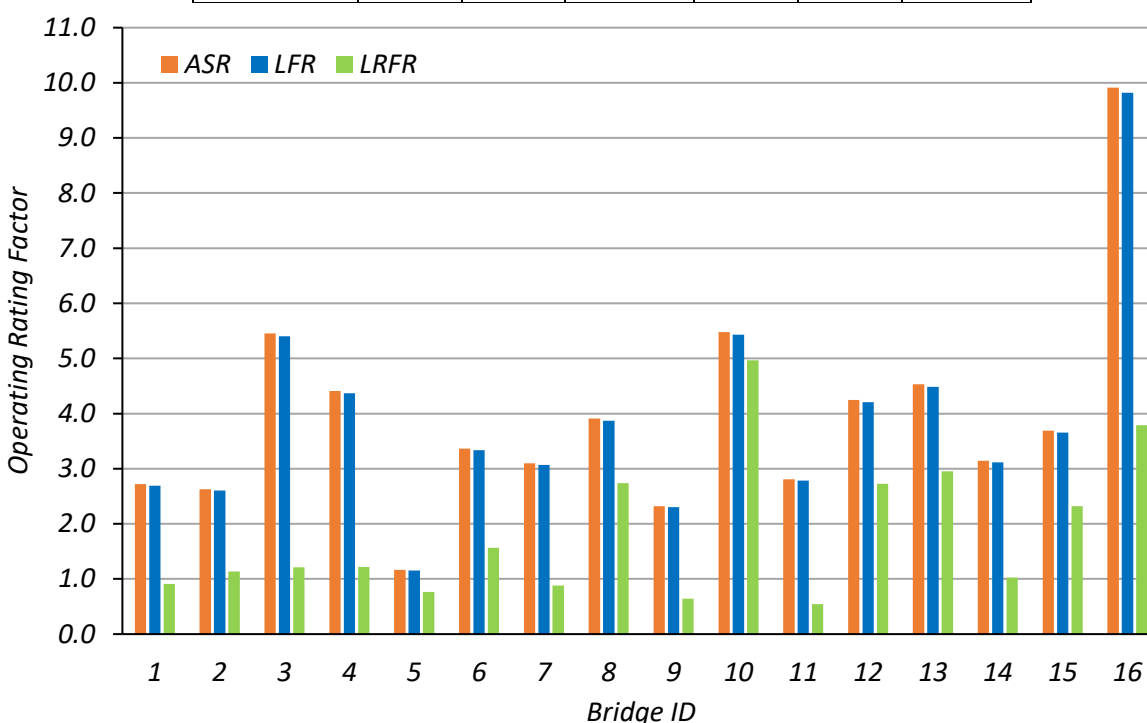


Figure 5.11. Interior Girder Shear Operating RFs for SC Bridges

For the interior girder shear of the selected SC bridges, the ASR method produced a higher average inventory RF (2.89) than the LFR method (2.42), which produced a higher average inventory RF than the LRFR method (1.47). The ASR method also produced a higher average operating RF (4.07) than the LFR method (4.03), which produced a higher average operating RF than the LRFR method (1.90).

For all but one continuous bridge, SC-5, both the ASR method and the LFR method produced inventory and operating interior girder shear RFs greater than or equal to 1.0. The LRFR

method produced operating RFs greater than or equal to 1.0 for 11 of the continuous bridges. There were nine SC bridges for which the LRFR method did not produce inventory RFs greater than 1.0. None of the interior girder shear RFs control over the corresponding interior girder flexure RFs.

5.2.3.4 Exterior Girder Shear Analysis

The shear RFs found for the design loads using each method for the exterior girders of each bridge are shown in Table 5.18 and Figure 5.12. In the exterior girder shear analysis, for all of the selected bridges, the ASR method produced a higher operating RF than both the LFR and LRFR methods.

Table 5.18. Exterior Girder Shear RFs for SC Bridges

ID	Inventory HS20 RF			Operating HS20 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SC-1	1.38	1.17	0.26	1.97	1.95	0.33
SC-2	2.22	1.87	0.95	3.14	3.11	1.23
SC-3	11.29	9.30	0.96	15.67	15.53	1.24
SC-4	1.96	1.63	0.95	2.75	2.72	1.23
SC-5	1.80	1.54	0.63	2.60	2.58	0.81
SC-6	3.65	3.00	1.35	5.05	5.00	1.76
SC-7	8.18	6.83	0.57	11.50	11.40	0.74
SC-8	3.55	3.01	2.15	5.07	5.02	2.79
SC-9	1.21	1.00	0.50	1.69	1.67	0.64
SC-10	3.94	3.26	3.83	5.48	5.43	4.97
SC-11	4.20	3.51	0.37	5.91	5.86	0.48
SC-12	3.30	2.80	2.12	4.73	4.68	2.75
SC-13	3.16	2.66	2.27	4.48	4.44	2.95
SC-14	2.36	1.95	0.81	3.28	3.25	1.05
SC-15	2.67	2.24	1.82	3.77	3.73	2.36
SC-16	10.23	8.50	3.03	14.32	14.19	3.93
Average	4.07	3.39	1.41	5.71	5.66	1.83
Std. Dev.	2.98	2.46	1.00	4.15	4.11	1.30

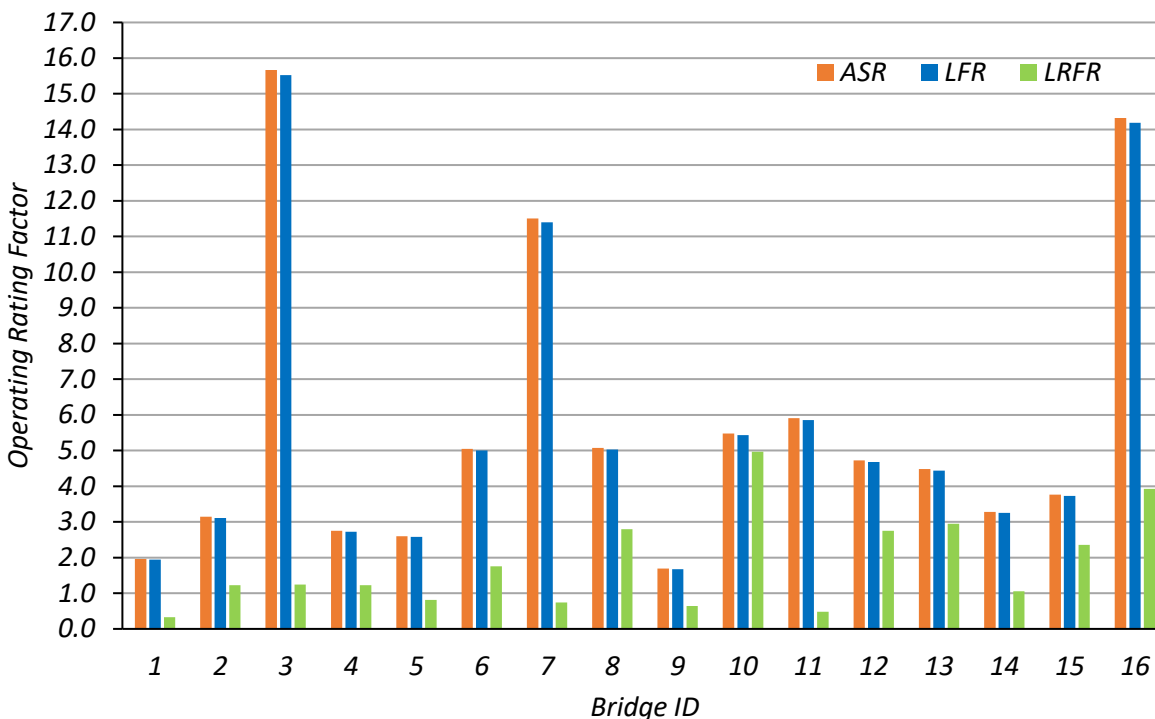


Figure 5.12. Exterior Girder Shear Operating RFs for SC Bridges

For the exterior girder shear of the selected SC bridges, the ASR method produced a higher average inventory RF (4.20) than the LFR method (3.49), which produced a higher average inventory RF than the LRFR method (1.46). The ASR method also produced a higher average operating RF (5.88) than the LFR method (5.82), which produced a higher average operating RF than the LRFR method (1.89).

Again, the interior girder and exterior girder ratings for Bridge SC-10 are the same: the singular rating for the floor beam. For all bridges, both the ASR method and the LFR method produced inventory and operating exterior girder shear RFs greater than or equal to 1.0. The LRFR method produced an inventory RF greater than or equal to 1.0 for seven SC bridges and operating RFs greater than or equal to 1.0 for 11 SC bridges. None of the exterior girder shear RFs control over the corresponding exterior girder flexure RFs.

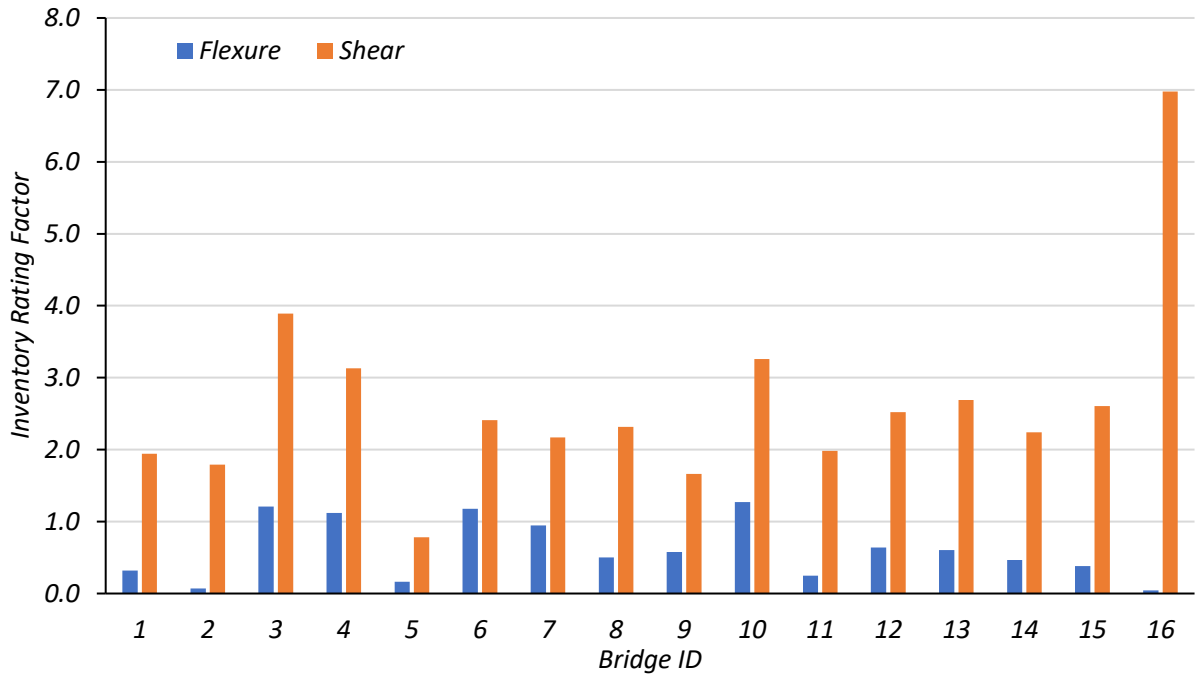
5.2.3.5 Comparison of SC Bridge Interior Girder Flexure and Shear RFs

After calculating the flexure and shear RFs for the selected SC bridges, the RFs were compared to determine if girder flexure or girder shear was the controlling limit state for each bridge. Table 5.19 and Figure 5.13 show the interior girder inventory and operating flexure and shear RFs found

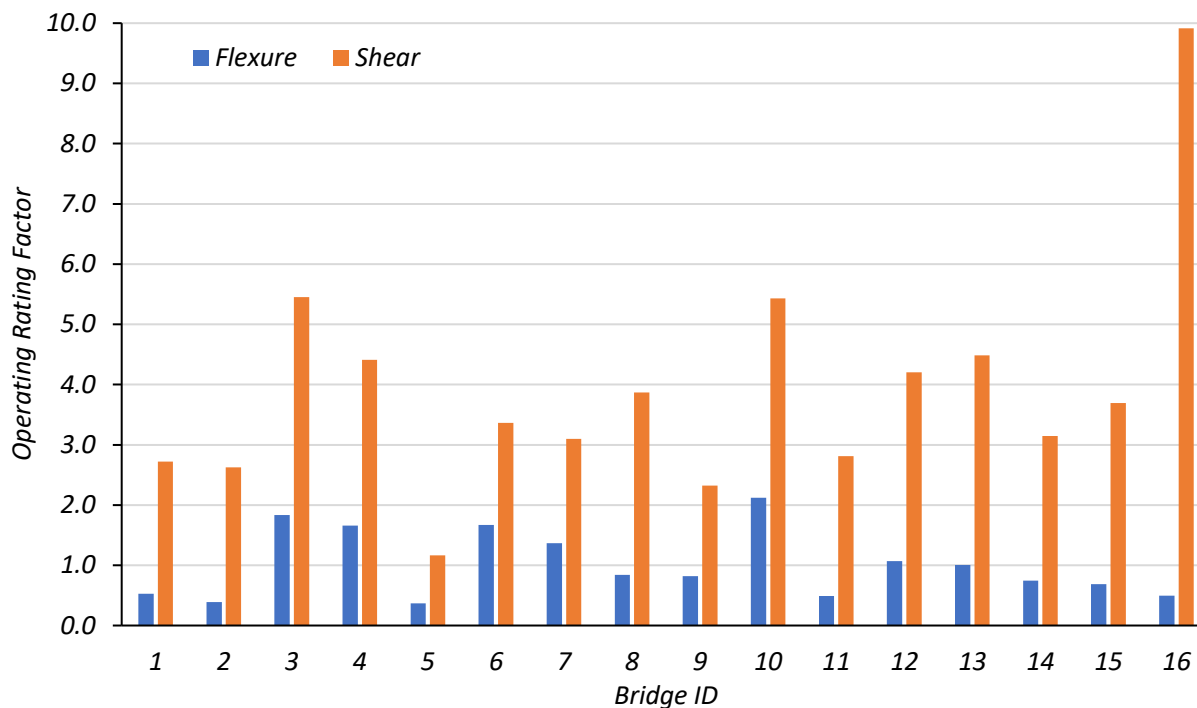
using the same method TxDOT uses in analysis. For every SC bridge, the flexure RF controlled the interior girder rating at both the inventory and operating level.

Table 5.19. Interior Girder Flexure and Shear RFs for SC Bridges

ID	TxDOT Rating Method	Inventory HS20 RF		Operating HS20 RF	
		Flexure	Shear	Flexure	Shear
SC-1	ASR	0.32	1.94	0.53	2.72
SC-2	ASR	0.07	1.79	0.39	2.63
SC-3	ASR	1.21	3.89	1.83	5.45
SC-4	ASR	1.12	3.13	1.66	4.41
SC-5	ASR	0.17	0.78	0.37	1.17
SC-6	ASR	1.18	2.41	1.67	3.37
SC-7	ASR	0.95	2.17	1.37	3.10
SC-8	LFR	0.50	2.32	0.84	3.87
SC-9	ASR	0.58	1.66	0.82	2.32
SC-10	LFR	1.27	3.26	2.12	5.43
SC-11	ASR	0.25	1.98	0.49	2.81
SC-12	LFR	0.64	2.52	1.07	4.21
SC-13	LFR	0.60	2.69	1.01	4.49
SC-14	ASR	0.47	2.24	0.74	3.15
SC-15	ASR	0.38	2.61	0.69	3.69
SC-16	ASR	0.04	6.98	0.49	9.91
Avg.		0.61	2.65	1.01	3.92
Std. Dev.		0.40	1.31	0.54	1.89



(c) Inventory RFs



(d) Operating RFs

Figure 5.13. Comparison of Interior Girder Flexure and Shear RFs for SC Bridges

5.2.4 Check of SC Bridge Calculations

After conducting the rating analysis of each bridge, a check was conducted to confirm that the load rating process followed the AASHTO MBE and TxDOT practices. Therefore, for each bridge, the input characteristics were changed to match TxDOT’s values where differences were noted, and the same standard practices employed by TxDOT were used. Any differences within 5 percent could be attributed to slight rounding differences, especially with the choice of the span length. Table 5.20 shows the results of this procedure for each selected SC bridge.

There was only one SC bridge for which changing the input characteristics did not produce a result within 5 percent of TxDOT’s rating. For Bridge SC-13, as previously mentioned, TxDOT uses the pan girder ratings as the ratings for the overall bridge instead of the steel multi-girder ratings that control the ratings, according to both TxDOT RFs and the calculated RFs. Through this approach, the calculated ratings were close to or the same as the TxDOT ratings, confirming the process used in the rating calculations.

Table 5.20. Interior Girder Results for SC Bridges with Adjusted Characteristics

Bridge ID	Rating Method	Adjusted Calculations HS20 RF		Adjusted Calculations RF/TxDOT RF	
		Inventory	Operating	Inventory	Operating
SC-1	ASR	0.38	0.59	1.05	1.05
SC-2	ASR	0.43	0.75	1.04	1.04
SC-3	ASR	0.98	1.46	0.99	1.00
SC-4	ASR	1.08	1.60	0.99	1.00
SC-5	ASR	0.58	0.84	0.99	0.95
SC-6	ASR	1.13	1.61	1.05	1.05
SC-7	ASR	0.86	1.25	1.03	1.02
SC-8	LFR	0.53	0.89	0.96	0.97
SC-9	ASR	0.56	0.89	1.01	0.96
SC-10	LFR	0.60	1.01	1.04	1.05
SC-11	ASR	0.38	0.60	0.97	0.98
SC-12	LFR	0.56	0.93	1.01	1.00
SC-13	LFR	0.59	0.99	0.81	0.82
SC-14	ASR	0.50	0.76	1.04	1.04
SC-15	ASR	0.56	0.94	1.00	1.00
SC-16	ASR	0.33	0.56	0.99	1.00
Avg.	-	0.63	0.98	1.00	1.00
Std. Dev.	-	0.24	0.33	0.06	0.06

5.2.5 Comparison of Calculated Ratings with TxDOT Ratings

5.2.5.1 General

After confirming the rating process, the originally calculated RFs were then compared to the RFs that TxDOT provides in its inspection reports. The exterior girder RFs are not considered in this comparison because TxDOT does not analyze exterior girders in the rating calculations provided in its inspection reports. For each selected SC bridge, Table 5.21 and Figure 5.14 show the TxDOT RFs, the rating method used by TxDOT, and the ratio of calculated RF to TxDOT RF using the same rating method.

Of the 16 SC bridges whose interior girders were analyzed for flexure using the same method as TxDOT, five had inventory and operating design-level ratings that were greater than or equal to the TxDOT ratings. Of the bridges that had lower design-level ratings than TxDOT's at both the inventory and operating level, three had inventory and operating design-level ratings that were within 10 percent of the TxDOT ratings. For these bridges, the difference in ratings can

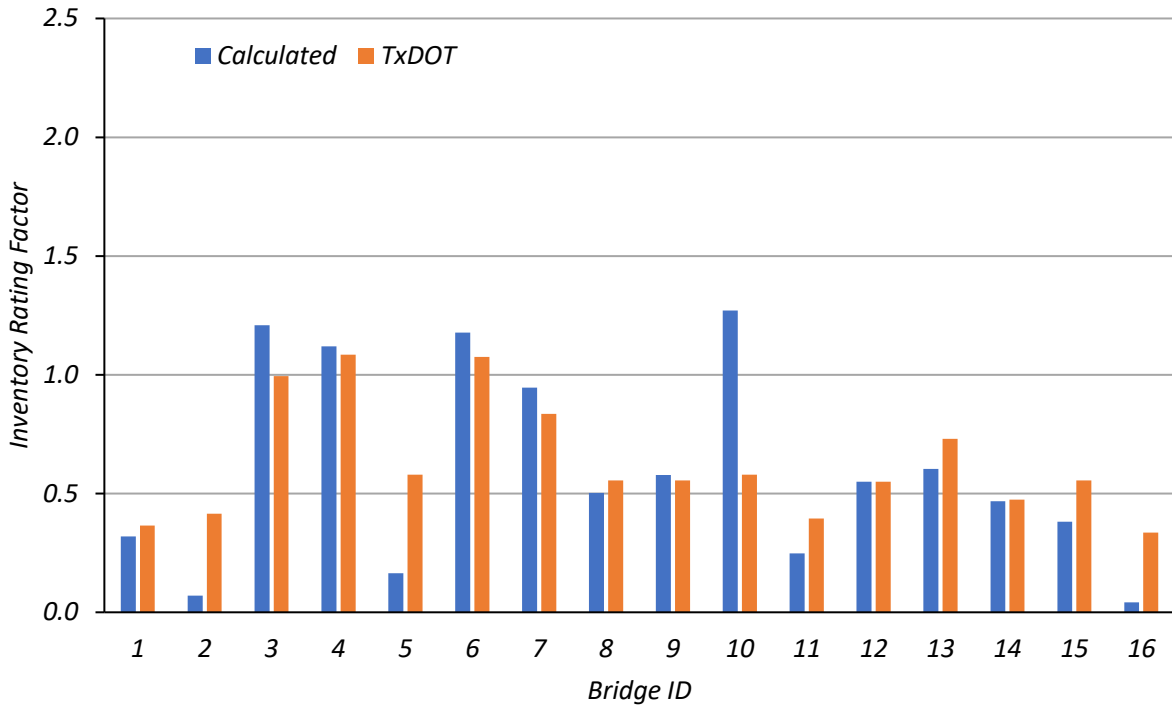
usually be attributed to rounding differences or differences in the consideration of certain aspects of loads. Of the bridges that had lower design-level ratings than TxDOT's at both the inventory and operating levels, three had inventory and operating design-level ratings that were within 20 percent of the corresponding TxDOT ratings. There were five bridges that had inventory or operating design-level ratings with more than a 20 percent difference relative to the TxDOT ratings.

After the selected SC bridges were rated, the interior girder flexure RF for each bridge was analyzed to determine the source of any difference with respect to the RFs in the TxDOT inspection reports. The load rating calculations were compared with the TxDOT calculations as closely as possible, given that in some cases the inspection records did not have complete calculations. The interior girder RFs that were obtained from the calculations were used for comparison since TxDOT's ratings are based on the interior girder. Table 5.22 shows every continuous bridge and notes reasons why each bridge has a different interior girder flexure RF than the corresponding TxDOT RF.

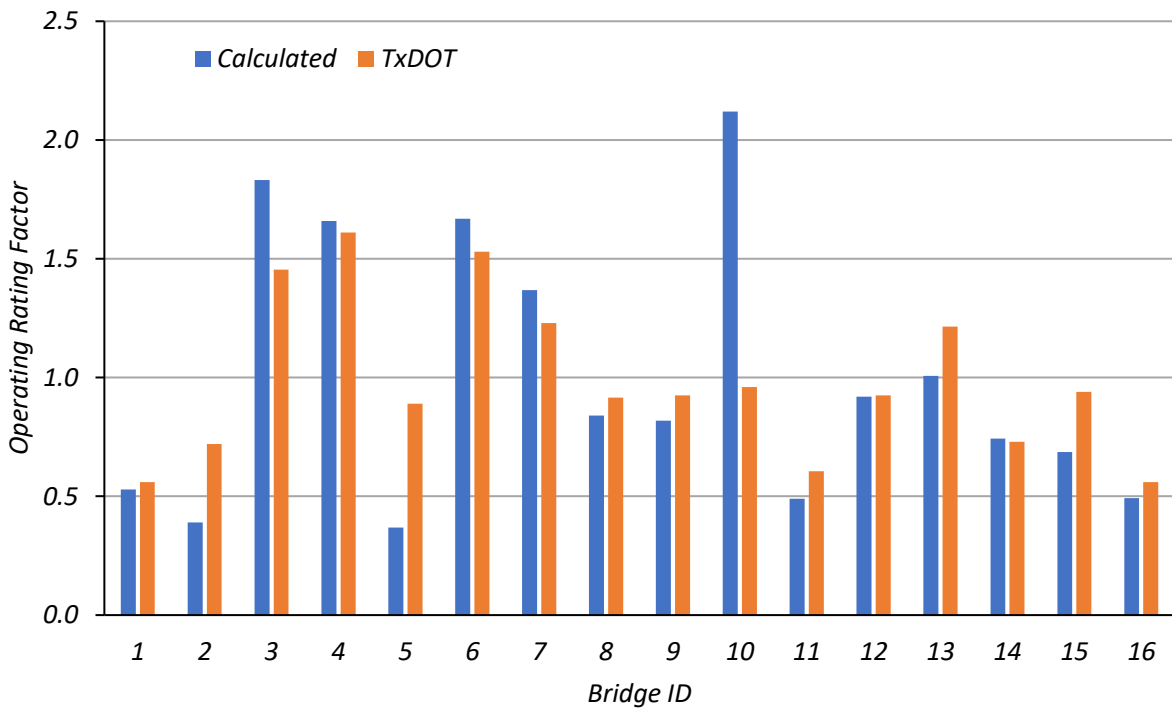
Table 5.21. Comparison of Interior Girder Flexure RFs for SC Bridges to TxDOT RFs Using the Same Rating Method

ID	Calculated HS20 RF		HS20 RF (TxDOT)		TxDOT Rating Method	Calculated/TxDOT	
	Inventory	Operating	Inventory	Operating		Inventory	Operating
SC-1	0.32	0.53	0.37	0.56	ASR	0.87	0.94
SC-2	0.07	0.39	0.42	0.72	ASR	0.17	0.54
SC-3	1.21	1.83	1.00	1.46	ASR	1.21	1.26
SC-4	1.12	1.66	1.09	1.61	ASR	1.03	1.03
SC-5	0.17	0.37	0.58	0.89	ASR	0.28	0.41
SC-6	1.18	1.67	1.08	1.53	ASR	1.09	1.09
SC-7	0.95	1.37	0.84	1.23	ASR	1.13	1.11
SC-8	0.50	0.84	0.56	0.92	LFR	0.91	0.92
SC-9	0.58	0.82	0.56	0.93	ASR	1.04	0.88
SC-10	1.27	2.12	0.58	0.96	LFR	2.19	2.21
SC-11	0.25	0.49	0.40	0.61	ASR	0.63	0.81
SC-12	0.55	0.92	0.55	0.93	LFR	1.00	0.99
SC-13	0.60	1.01	0.73	1.22	LFR	0.83	0.83
SC-14	0.47	0.74	0.48	0.73	ASR	0.99	1.02
SC-15	0.38	0.69	0.56	0.94	ASR	0.69	0.73
SC-16	0.04	0.49	0.34	0.56	ASR	0.13	0.88

Average	0.61	1.01	0.63	0.99	-	0.89	0.98
Std. Dev.	0.40	0.54	0.24	0.32	-	0.47	0.38



(a) Inventory RFs



(b) Operating RFs

Figure 5.14. Calculated RF vs. TxDOT RF for Interior Girder Flexure for SC Bridges

Table 5.22. Reasons for Differences between Calculated and TxDOT Interior Girder Flexure RFs for SC Bridges

ID	Reasons for Difference in Rating Factor					
	Dead Load Components	Analysis Considerations	Section Properties	Live Load Distribution Factors	Various Moment Differences	Unknown
SC-1	x	x				
SC-2	x	x				
SC-3					x	
SC-4						
SC-5	x		x	x		
SC-6						
SC-7					x	
SC-8	x					
SC-9	x		x			
SC-10			x			
SC-11	x	x				
SC-12						
SC-13	x	x				
SC-14						
SC-15	x			x		
SC-16	x	x				

For every bridge that exhibited a calculated RF lower than the TxDOT RF, the consideration of certain dead load components played into the difference. TxDOT only considers the girders, concrete deck, and wearing surface in most of its dead load calculations. Some of these bridges have significant concrete curbs, lateral bracing members, or railings that are considered in the calculations developed in this project. The rating calculations include a best estimate of the weight of these components and distribute them to the girders according to the AASHTO MBE. Although these are not major components of the dead load of the structure, this addition produced a higher dead load moment for almost every bridge. This difference affected each SC bridge overall and is the main reason for slight differences in the calculated RFs and the TxDOT RFs. This reason was in some cases paired with another reason to increase the difference. This difference also applies to bridges that exhibited higher calculated RFs than the TxDOT RF; however, some other difference in the calculations outweighed this item, causing a higher RF to be calculated.

Another difference between the calculations and TxDOT load rating is that TxDOT does not analyze off-system continuous bridges in negative flexure when using the ASR method. These

bridges are assumed to have an equivalent simple-span with a length usually between 70 percent and 90 percent of the actual bearing support to bearing support length. Then, TxDOT analyzes the bridge as it would any other simple-span in positive flexure. However, the calculations for this project evaluate these off-system bridges in both the negative and positive moment regions. The calculated RFs determine the live load moments acting on the continuous bridge using analytical tools, whereas TxDOT uses an approximate method for its ASR analysis. This approximate method uses the moment values from the MBE with the determined equivalent simple-span length. This is another source of slight differences in ratings of all continuous bridges analyzed using ASR.

One final difference between the project calculations and TxDOT ratings is related to the braced length of the girders. First, the project calculations analyze each bridge considering the braced lengths provided by lateral members for LFR and assumed to be fully braced for ASR and LRFR. However, TxDOT assumes the deck provides full bracing for both LFR and LRFR. This assumption gives them a slightly higher capacity for the LFR method in some cases; however, the serviceability limit state controls for these bridges.

Several bridges that displayed larger differences in the RFs were further examined. Table 5.23 shows the main reason for the difference in RF for every simple-span bridge that had an RF greater than 10 percent lower than TxDOT’s RF. Table 5.24 shows the main reason for the difference in RF for every simple-span bridge that had an RF greater than 10 percent higher than the TxDOT RF.

Table 5.23. SC Bridges with Lower Calculated Interior Girder Flexure RFs—Main Reason for Differences

Main Reason for Difference	Number of Bridges	Bridge IDs
Analysis Considerations	5	SC-1, SC-2, SC-11, SC-13, SC-16
Section Properties	2	SC-5, SC-9
Live Load Distribution Factors	1	SC-15

Table 5.24. SC Bridges with Higher Calculated Interior Girder Flexure RFs—Main Reason for Differences

Main Reason for Difference	Number of Bridges	Bridge IDs
Section Properties	1	SC-10
Moment Differences	1	SC-3, SC-7

The following subsections describe the differences observed between the calculated ratings and TxDOT ratings for each bridge that exhibited a difference of 10 percent or greater.

5.2.5.2 Comparison of Ratings for Bridge SC-1

The project calculations determine a negative moment region RF, while TxDOT considers the section in positive flexure. Because the section is composite, the negative moment region has significantly less load-carrying capacity than the positive moment region, and this factor controls the calculated ratings. In addition, many other differences exist between the rating calculations for the SC-1 bridge and the corresponding TxDOT calculations. The calculations follow the composite section procedures outlined in Example A1 in MBE Appendix A (AASHTO 2018), while TxDOT follows a different procedure using different composite section properties. Moreover, as mentioned in Section 5.2.2.1, a different composite section modulus using the entire concrete slab was calculated for the serviceability ratings.

5.2.5.3 Comparison of Ratings for Bridge SC-2

The calculations find a negative moment region RF, while TxDOT considers the section in positive bending. Analysis of the negative moment region using the applied live load moments controls the rating. The calculated positive moment region RFs are 0.38 for inventory and 0.66 for operating. These are much more comparable to TxDOT's RFs of 0.42 and 0.72 for inventory and operating, respectively.

5.2.5.4 Comparison of Ratings for Bridge SC-3

TxDOT does not reduce its span length to account for the bridge being continuous. This factor leads to a higher live load moment than what is calculated for a continuous bridge. Because the drawings show the bridge as continuous and it is listed as a continuous bridge in NBI, the bridge was analyzed as continuous in the project calculations.

5.2.5.5 *Comparison of Ratings for Bridge SC-5*

The project calculations find the effective width of a composite slab as prescribed in Article 10.38.3.1 of the *AASHTO Standard Specifications*. TxDOT uses the spacing of the girders, giving a higher composite section modulus. Further, for this bridge, TxDOT uses a one-lane distribution factor instead of a two-lane distribution factor, which is similar to a number of the simple-span SM bridges, where the project calculations follow the AASHTO guidelines and TxDOT inventory sheets, identifying any bridge having over 20 ft of roadway width as a two-lane bridge. This difference produces a higher LLDF and therefore a lower calculated RF than TxDOT for this bridge.

5.2.5.6 *Comparison of Ratings for Bridge SC-7*

The calculations use a slightly lower live load moment and a slightly lower dead load moment. This difference can be attributed to the effective span length that TxDOT uses being an approximation and not correlating exactly to RISA or to the continuous span moment coefficients.

5.2.5.7 *Comparison of Ratings for Bridge SC-9*

TxDOT and the calculations seem to use slightly different steel cross-section values. The rating calculations take all of the properties for the cross-section from the 14th edition of the *AISC Steel Construction Manual*. For this bridge, TxDOT does not enter a cross-section designation into its RATE program and instead enters the actual physical dimensions of the section, which renders a slightly different cross-section, higher capacity, and higher rating at the operating level because it does not take into account 5 percent corrosion mentioned in TxDOT's inspection report.

5.2.5.8 *Comparison of Ratings for Bridge SC-10*

Bridge SC-10 had ratings controlled by a deteriorated floor beam. TxDOT analyzes the floor beam as a deteriorated cross-section, while the project calculations were conducted assuming that the section is not deteriorated. In an inspection report dated April 15, 2016, TxDOT notes that the deteriorated floor beams have been strengthened with additional floor beams and that the bridge should be re-evaluated for posting. The TxDOT ratings uses a significantly lower capacity based on deterioration, which causes the differences in the rating. However, for this particular bridge,

more details are necessary to confirm that the strengthening is adequate to remove the consideration of deterioration. The project calculations provide insight into the potential RFs that can be achieved if the deterioration of the floor beams is no longer an issue.

5.2.5.9 Comparison of Ratings for Bridge SC-11

TxDOT uses a reduced effective span length for Bridge SC-11, which renders a significantly lower live load moment than that found using a continuous bridge model, and a lower dead load moment than what was calculated. The TxDOT rating also does not consider the curb, which adds significant weight to the dead load in the project calculations.

5.2.5.10 Comparison of Ratings for Bridge SC-13

Bridge SC-13 is comprised of four, 40 ft concrete pan girder spans and three continuous steel spans of 60 ft, 75 ft, and 60 ft. TxDOT analyzes both the concrete pan girder spans and the steel continuous spans. TxDOT obtains the same rating factor as the project calculations for the steel continuous spans and finds this to be lower than the concrete pan girder rating. However, TxDOT uses the concrete pan girder rating as the rating for the entire bridge, even though it is not shown to control.

5.2.5.11 Comparison of Ratings for Bridge SC-15

Like several other narrow bridges, TxDOT uses a one-lane LLDF, and the calculations use a two-lane distribution factor. Using two-lane LLDFs increases the applied live load found by the calculations and therefore decreases the rating factor.

5.2.5.12 Comparison of Ratings for Bridge SC-16

The dead load components that are considered in the calculations add 17 kip-ft of applied dead load moment, which reduces the rating factor. TxDOT also averages its operating RF and inventory RF for this bridge to obtain a new inventory RF, making it much higher than the calculated inventory RF. The project calculations use the usual process to compute RFs, assuming that the bridge is a simple span over its full length with no adjustments. The reason for the bridge being analyzed as a simple span was mentioned in Section 5.2.2.1 and is elaborated upon in Section 5.2.5.13.

5.2.5.13 *Special Considerations for Bridge SC-15 and Bridge SC-16*

Bridge SC-15 and SC-16 are both two-span continuous bridges that have at least one girder that does not touch the interior support and therefore spans the entire length of the bridge. For SC-15, TxDOT highlights the words “without bent support” and uses the full bridge length as the span length in the load rating calculations but does not state the size of the gap between the girder and the interior support. For SC-16, TxDOT states that three beams out of seven do not touch the bridge pier—one of the beams is 1.5 in. above the gap—and it uses the full bridge length as the span length in its load rating calculations. Since the TxDOT practice is to analyze both of these bridges as simple spans over their full length, the same practice is also assumed for these calculations. However, it should be noted that, under live load, the girders will most likely come into contact with the support in SC-15. Furthermore, the live load required to cause that occurrence to happen in SC-16 was determined to be 11 percent of the full HS20 truck.

Because the girders are likely to come into contact with the support during loading, and because the load on the bridge will be distributed transversely through the deck and lateral bracing, thereby increasing the potential for these girders to come into contact before the flexure limit state is exceeded, Bridges SC-15 and SC-16 were also analyzed as fully continuous for comparison. For the continuous girder analysis, using the ASR method, the interior girder inventory and operating RFs were 2.78 and 3.89 for SC-15 and 2.83 and 3.96 for SC-16. The values presented initially in Table 5.14 and these values can be viewed as the lower and upper bounds of the inventory and operating RFs for these bridges. Further in-depth analysis would need to be conducted to determine the most appropriate RFs for Bridges SC-15 and SC-16.

5.2.6 **Conclusion**

The girders of 16 continuous steel bridges with concrete decks were analyzed according to the *AASHTO MBE*, *AASHTO Standard Specifications*, and *AASHTO LRFD Specifications*. The exterior girder was initially not considered in order to compare RFs of the same structural components to TxDOT. The ASR and LFR methods produced significantly higher RFs than the LRFR method, with the LFR method being slightly higher than the ASR method. Most bridges were calculated to have RFs within 10 percent of the corresponding TxDOT RFs, with the differences attributable to the dead loads considered for analysis. For the bridges that exhibited calculated RFs with more than a 10 percent difference than the TxDOT RFs, further examination

was performed to determine the reason for these differences. The associated contributing factors were identified and discussed. Based on the analysis performed for the selected continuous steel multi-girder bridges, possible areas for improvement to load posting are identified and elaborated upon in Section 5.3 of this report.

5.3 AREAS OF POSSIBLE IMPROVEMENT FOR STEEL BRIDGES

5.3.1 Partial Composite Action

It is well known that a composite slab-girder bridge exhibits a higher moment capacity than a non-composite bridge. This behavior was observed in the rating calculations conducted for this project while both the composite and non-composite capacities of the bridge girders were determined and the appropriate value was selected for the load rating analysis. The composite moment capacity was, for most bridges, significantly higher than the non-composite capacity. Bridges without shear studs or embedded girders may not be able to be analyzed as fully composite; however, as Chajes et al. (1997) showed, non-composite bridges do exhibit some amount of partial composite behavior. Chajes et al. (1997) instrumented and load tested a simply-supported, three-span, non-composite steel multi-girder and concrete slab bridge. Based on the strains obtained from the load test, Chajes was able to determine that the bridge was acting as partially composite and then calculate new section properties. These properties were then used in a finite element model of the bridge that was developed. A similar procedure could be used for any of the selected bridges that could increase the section modulus, increase the capacity, and improve the rating of the bridge.

To examine the effects of applying partial composite action, two steel bridges that have been identified as possible candidates for further investigation were re-analyzed. Bridge SM-5 was analyzed using full composite action because its flange is embedded 0.5 in. into the concrete deck, according to TxDOT records. Bridge SC-12 was analyzed as being 50 percent composite by taking the average of the fully composite and non-composite section properties and moment capacities. Table 5.25 shows the results of this procedure in the form of a ratio of revised RF to initially calculated RF. This procedure increased the RF of SM-5 by more than double the amount of the originally calculated factor and increased the RF of SC-12 by almost 50 percent of the originally calculated value.

Table 5.25. Effect of Composite Behavior on Interior Girder Ratings

Bridge ID	Rating Method	TxDOT HS20 RF		Non-composite HS20 RF		Composite HS20 RF		Composite RF/ Non-composite RF	
		Inv.	Oper.	Inv.	Oper.	Inv.	Oper.	Inv.	Oper.
SM-5	LFR	0.47	0.79	0.48	0.81	1.01	1.69	2.09	2.09
SC-12	LFR	0.55	0.93	0.55	0.92	0.87	1.46	1.58	1.59

Notes:

1. SM-5 was analyzed as fully composite due to flange embedment.
2. SC-12 was analyzed as partially (50 percent) composite.

5.3.2 Number of Lanes

In its load rating calculation, TxDOT is already analyzing some narrow two-lane bridges using one-lane LLDfFs, but could expand upon and continue to practice this approach as appropriate. One-lane distribution factors are lower than two-lane distribution factors, meaning that by using one-lane distribution factors, the applied live load moment on the bridge can be decreased, and the rating can be increased. This result was shown for many of the selected simple-span and continuous steel multi-girder bridges, and TxDOT calculated higher RFs than the project calculations for this reason. If a bridge is observed to have only one set of wheel-lines shown on the wearing surface, indicating vehicles are using it as a one-lane bridge, or if an engineering decision is made that the bridge is too narrow to carry two lanes, the bridge could be converted into a one-lane bridge if deemed appropriate.

The effects of applying a reduction to the number of lanes was not performed for the two bridges selected for further investigation because both bridges are on major routes with high ADT. It would not be practical, or even possible, for TxDOT to reduce the number of lanes on these bridges. This option was tested, however, for a few bridges that exhibited narrow deck widths. Table 5.26 shows the effect of reducing the number of lanes to one lane for selected bridges that have a deck width smaller than 24 ft and currently carry two lanes, according to the NBI. This approach increased the RFs for bridges analyzed using the LFR method by 35 percent of the originally calculated factor and improved the RFs for bridges analyzed using the ASR method by 27 percent of the originally calculated value.

Table 5.26. The Effect of Number of Design Lanes to Interior Girder Ratings for Two-Lane Bridges with Deck Widths Below 24 ft

Bridge ID	Rating Method	TxDOT HS20 RF		Two-Lanes HS20 RF		One-Lane HS20 RF		One-Lane RF/Two-Lanes RF	
		Inv.	Oper.	Inv.	Oper.	Inv.	Oper.	Inv.	Oper.
SM-7	LFR	0.57	0.95	0.53	0.88	0.71	1.19	1.35	1.35
SM-12	LFR	0.51	0.86	0.35	0.58	0.47	0.78	1.35	1.35
SM-25	ASR	3.37	4.77	2.53	3.61	3.22	4.59	1.27	1.27
SC-6	ASR	1.08	1.53	1.18	1.67	1.50	2.12	1.27	1.27
SC-9	ASR	0.56	0.93	0.58	0.82	0.74	1.04	1.27	1.27

5.3.3 Material Properties

As shown for Bridge SM-16, the steel yield strength used in analysis can significantly affect the rating of a bridge. For Bridge SM-16, TxDOT used a yield strength of 36 ksi. The project calculations use a yield strength of 50 ksi because it was stated in the TxDOT bridge inventory record file that city records state that the bridge has 50 ksi steel. This increase in yield strength caused the calculations to produce a higher RF than the TxDOT value. If steel coupons from steel bridges were to be tested in tension, these tests could reveal yield strengths to be higher than the tabulated values that TxDOT is currently using for older bridges in which the actual yield strength is unknown. Increasing the steel yield strength would lead to an increased capacity and therefore an increased rating.

To examine the effects of applying improved material properties, the two bridges selected for further investigation were re-analyzed. Both were analyzed assuming an approximately 20 percent increase in yield strength from the assumed value of 33 ksi to 40 ksi. Table 5.27 shows the results of this procedure in the form of a ratio of revised RF to initially calculated RF. This procedure improved the RF of SM-5 by 33 percent of the originally calculated factor and increased the RF of SC-12 by 25 percent of the originally calculated value.

Table 5.27. The Effect of Yield Strength to Interior Girder Ratings

Bridge ID	Rating Method	TxDOT HS20 RF		HS20 RF for $f_y = 33$ ksi		HS20 RF for $f_y = 40$ ksi		RF with $f_y = 40$ ksi / RF with $f_y = 33$ ksi	
		Inv.	Oper.	Inv.	Oper.	Inv.	Oper.	Inv.	Oper.
SM-5	LFR	0.47	0.79	0.48	0.81	0.64	1.07	1.33	1.33
SC-12	LFR	0.55	0.93	0.55	0.92	0.71	1.18	1.29	1.28

5.3.4 Live Load Distribution Factors

The current LLDFs found in the *AASHTO Standard Specifications* and *AASHTO LRFD Specifications* are simple equations that are accurate for certain bridges of certain types and characteristics and less accurate and too conservative for others (Hueste et al. 2015). If a bridge was to be modeled using finite element software, it could more accurately show the distribution of the live load to the girders. These changes to the distribution factors could then possibly be applied to all bridges exhibiting certain characteristics. In a study conducted by researchers at Georgia Tech (Ellingwood et al. 2009c), this approach was taken, and the change in LLDFs was used to help increase the RF of bridges that were analyzed. Other states around the country, including Iowa, Oregon, and Pennsylvania, practice this approach, as well (IOWADOT 2013; ODOT 2015; PennDOT 2010).

To examine the effects of applying improved LLDFs, the two bridges selected for further investigation were re-analyzed. Both were analyzed assuming a new LLDF reduced to 75 percent of the initially calculated value. Table 5.28 shows the results of this procedure in the form of a ratio of revised RF to initially calculated RF. This procedure improved the RF of both SM-5 and SC-12 by 33 percent of the originally calculated RFs.

Table 5.28. The Effect of Live Load Distribution Factors to Interior Girder Ratings

Bridge ID	Rating Method	TxDOT HS20 RF		Calculated HS20 RF		Revised Calculated HS20 RF		Revised Calculated RF/ Calculated RF	
		Inv.	Oper.	Inv.	Oper.	Inv.	Oper.	Inv.	Oper.
SM-5	LFR	0.47	0.79	0.48	0.81	0.65	1.08	1.34	1.34
SC-12	LFR	0.55	0.93	0.55	0.92	0.73	1.22	1.33	1.33

5.3.5 Refined Analysis Models

Load ratings could be increased for some bridges through refined analysis methods, such as the use of the FEM. Current analysis methods given in AASHTO's manuals and standards are

simplified procedures that are meant to be fairly quick to complete and straightforward to understand. However, such methods can incorporate simplifying assumptions that may reduce the accuracy of the results and err on the side of conservatism. FEM models can be tailored to a particular bridge and reduce the need for simplifying assumptions because they are designed to be more representative of the actual in-situ condition of the bridge. As a result, the outcomes of loading a finite element model of a bridge should be more accurate in terms of how the bridge will actually react under the design loads. That being said, finite element models take a considerable amount of time to develop and can be inaccurate if incorrect modeling parameters are used. Analyzing every bridge in an inventory would be an onerous task. However, if a number of typical bridges were analyzed using finite element models, a practice could possibly be developed for bridges of one type based on representative models. If these changes increased capacity or decreased the load effect through refined LLDFs, RFs for a group of bridges could be increased.

The effect of using refined analysis models was not evaluated in the basic load rating task but will be examined in the next task of the project. As discussed for Bridges SC-15 and SC-16, the current method of analysis TxDOT uses is intended to be on the conservative side. An in-depth analysis of these bridges, perhaps by using computer models to examine load distribution or to determine the behavior of continuous bridges where some girders are not in contact with an interior support, would more than likely produce higher load ratings than TxDOT currently applies.

5.3.6 Rating Method

As shown in the results of the basic loading rating, for both simple-span and continuous steel bridges with concrete decks, the ASR method produced a higher average RF for the analysis limit states performed on the selected SM and SC bridges in most cases. However, the LFR method produced higher RFs for more individual bridges than the ASR and LRFR method. TxDOT has rated some of the selected bridges using LFR already, but many of them were rated using ASR, as well. Although the improvement of rating using LFR over rating using ASR was not very significant, rating using LFR instead of ASR could be a practice that is used to slightly increase ratings in situations where it is appropriate.

The effect of using a different rating method was not analyzed for the selected bridges for Tasks 4 and 5, because TxDOT already uses the LFR method for both bridges. Previously in this

report, it was observed that the LFR method produced higher flexure rating factors than the ASR and LRFR methods.

5.3.7 Partial Fixity at Supports

Consideration of partial fixity at supports may be appropriate for some bridges. Chajes et al. (1997) load tested an existing simply-supported, three-span, non-composite steel multi-girder and concrete slab bridge and found that although the bridge was a simple-span, it exhibited some degree of support restraint. This restraint was also included in the finite element model of the bridge. If a simple-span bridge can be load tested and demonstrated to show partial fixity at the supports, it would lower the maximum positive live load and dead load moments near the middle of the span and therefore increase the rating. This process could be established for bridges with certain characteristics and potentially applied to similar bridges exhibiting those characteristics. However, introducing end fixity means that a negative moment will then be induced at the support, and the negative moment region should then be checked when determining the controlling RF. The effect of partial support fixity will be examined in the refined analysis task of the project.

5.4 CONCRETE DECK RATING ANALYSIS

5.4.1 General Analysis Procedure and Results

Although the focus for the rating of the steel multi-girder bridges with concrete decks was on the steel girders, some of the concrete decks were also analyzed in the transverse direction. There were nine bridges for which the deck reinforcement information was available. Table 5.29 shows these bridges and their relevant characteristics for conducting ratings of the deck. Note that TxDOT currently standardizes its deck design. All new concrete decks built are 8.5 in. thick and have #4 bars at a maximum 9 in. spacing in both directions for the top mat and the same reinforcement layout for the bottom mat. This provides a steel area of 0.27 in²/ft in each direction for both the top and bottom mat (TxDOT 2018b).

Table 5.29. Geometric and Material Properties of the Analyzed Reinforced Concrete Bridge Decks

ID	Deck Condition	Steel Yield Strength (ksi)	Concrete Compressive Strength (ksi)	Deck Thickness (in.)	Transverse Reinforcement Area		Distance to Bottom Steel from Bottom of Section (in.)	Distance to Top Steel from Top of Section (in.)	Girder Spacing (ft)
					Bottom (in ² /ft)	Top (in ² /ft)			
SM-3*	6	33	2.5	8	0.200	0.200	1.250	1.50	7'-1"
SM-4*	6	33	2.5	8	0.200	0.200	1.250	1.50	7'-1"
SM-11	6	33	3.0	7.25	0.338	0.338	1.500	1.50	8'-0"
SM-15	7	40	3.0	7	0.338	0.338	1.500	1.50	9'-6"
SC-10	6	33	2.5	7.5	0.355	0.229	1.250	2.00	6'-11"
SC-12	6	40	2.5	6	0.304	0.304	1.500	1.50	6'-8"
SC-13	6	40	3.0	6.5	0.179	0.280	1.500	2.00	7'-0"

* For Bridges SM-3 and SM-4, the standard drawings provided by TxDOT show a 6.75 in. deck thickness; however, in TxDOT's load rating calculations for these bridges, it uses an 8 in. deck thickness. For that reason, an 8 in. deck thickness was used for the deck ratings.

The decks of these bridges were assumed to be 1 ft wide continuous section over the girders that act as supports. The moment capacities in the positive and negative moment regions of the deck were determined by adding the moments due to both the top and bottom steel forces to the concrete compressive forces about the neutral axis of the section. For the ASR method, the allowable steel and concrete stresses at both the inventory and operating level were used when finding the capacity. These stresses were obtained from Table 6B.5.2.3-1 and Table 6B.5.2.4.1-1 in the MBE (AASHTO 2018). For the LFR capacity, the yield strength of steel and the compressive strength of concrete were used from AASHTO MBE based on the age of the structure; however, the moment capacity was multiplied by a factor of 0.9 per Article 8.16.1.2.2 of the *AASHTO Standard Specifications* (AASHTO 2002). For the LRFR capacity, the moment capacity was multiplied by a reduction factor based on the steel strain, with an approach similar to what is shown in Figure C5.5.4.2.1-1 in the *AASHTO LRFD Specifications* (AASHTO 2014). The live load moment for the ASR and LFR methods were found using the equation given in Article 3.24.3.1 of the *AASHTO Standard Specifications* (AASHTO 2002). The live load moment for the LRFR method was found by modeling each one-way deck slab to determine the moment demands. This live load moment was divided by an effective width, shown in Equation (5.7), to obtain a moment on a per foot basis:

$$E = 26 + (6.6S) \quad (5.7)$$

where:

E = Effective width (in.).

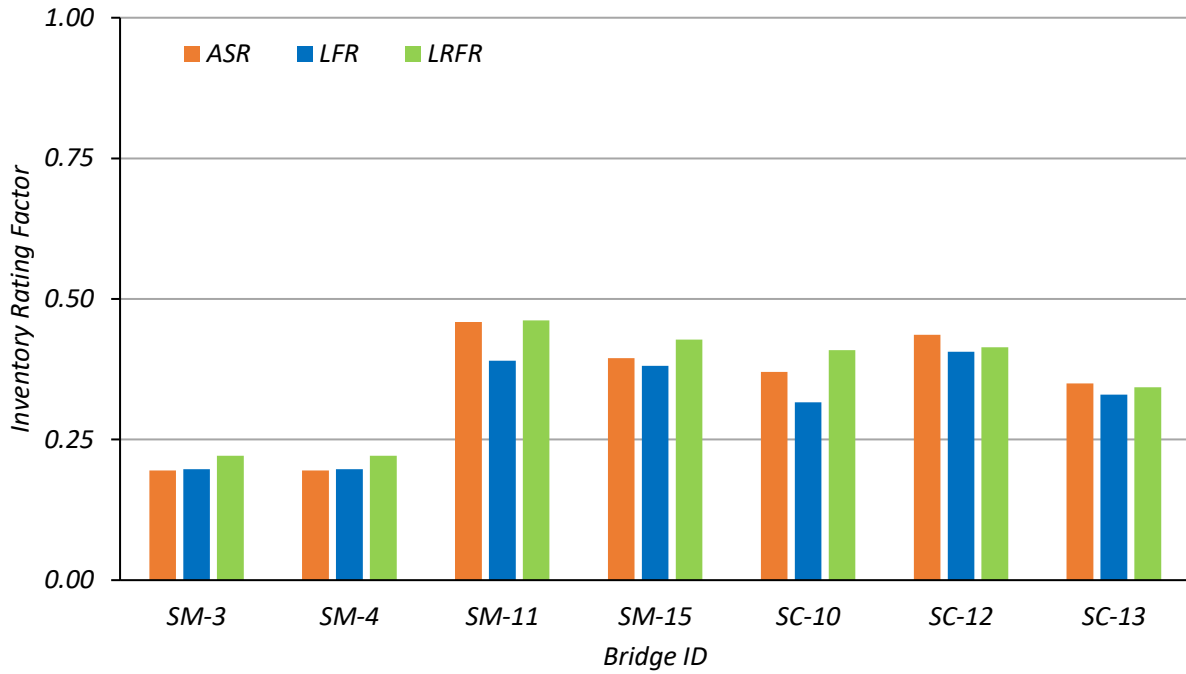
S = Girder spacing (ft).

Finally, the rating of each bridge was conducted for each method using the same general procedure as used for the steel girder sections. Table 5.30 and Figure 5.15 show the controlling RF found for each bridge deck in the transverse direction considering positive and negative moment regions.

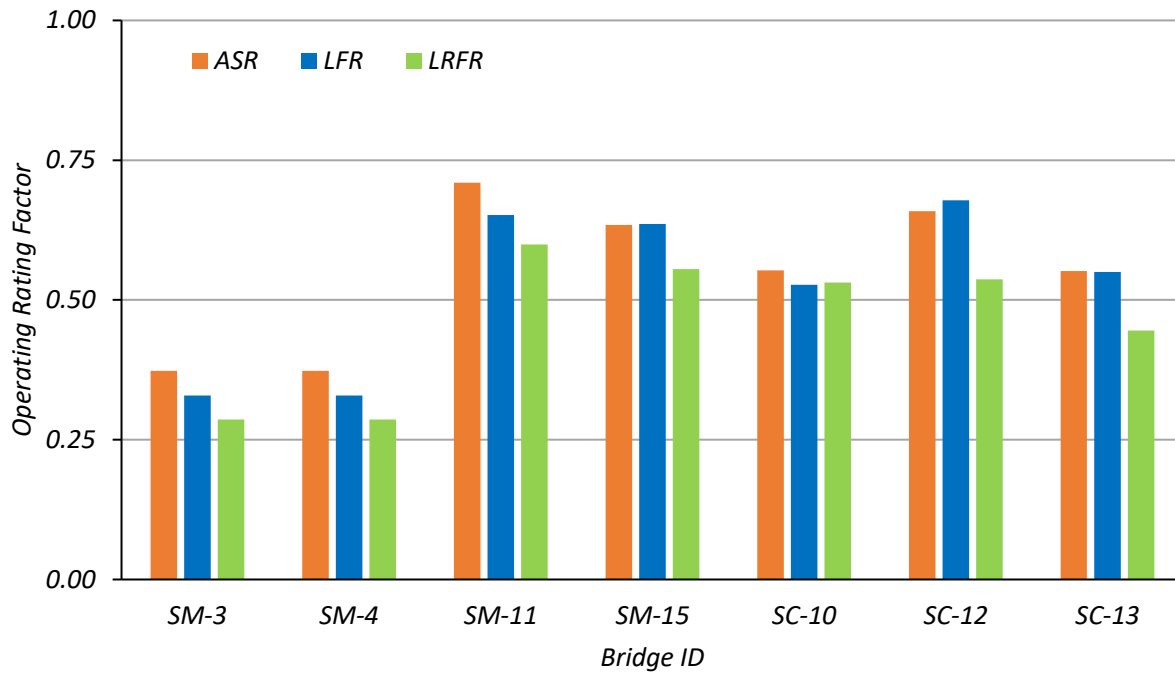
Table 5.30. Calculated Deck RFs for Bridges in the Transverse Direction with Deck Reinforcement Information

ID	Inventory HS20/HL93 RF			Operating HS20/HL93 RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
SM-3	0.20	0.20	0.22	0.37	0.33	0.29
SM-4	0.20	0.20	0.22	0.37	0.33	0.29
SM-11	0.46	0.39	0.46	0.71	0.65	0.60
SM-15	0.40	0.38	0.43	0.63	0.64	0.56
SC-10	0.32	0.28	0.32	0.50	0.47	0.41
SC-12	0.44	0.41	0.41	0.66	0.68	0.54
SC-13	0.35	0.33	0.34	0.55	0.55	0.45

For every bridge except one, SC-13, the ASR method produced higher operating RFs than the LFR method, and ASR produced higher operating RFs than the LRFR method for all bridges. The ASR method produced an operating RF 0.03 higher than the LFR method on average, with a standard deviation of 0.03. It also produced an operating RF 0.10 higher than the LRFR method, with a standard deviation of 0.02. The LFR method produced an operating RF 0.08 higher than the LRFR method on average, with a standard deviation of 0.03. None of the selected bridges had an inventory or operating RF greater than or equal to 1.0 for the decks. For every bridge, the deck RF corresponding to the same rating method used by TxDOT is lower than the current RF for the superstructure for that same bridge.



(a) Inventory RFs



(b) Operating RFs

Figure 5.15. Calculated Deck RFs for Selected Bridges in the Transverse Direction

5.4.2 Alternative Methods to Improve Deck Ratings

For all of the selected simple-span and continuous steel multi-girder bridges, TxDOT is not documenting load rating calculations for the deck within its inspection records. TxDOT is following a guideline in its *Bridge Inspection Manual*, which was in the second edition MBE and has recently been removed in the third edition MBE. This guideline essentially states that a concrete bridge or component can be rated for the HS20 load at the operating level as long as the bridge has been carrying unrestricted traffic for many years and shows no significant signs of distress. Therefore, TxDOT applies this guideline to the concrete decks of steel multi-girder bridges that have an HS20 operating RF of 1.0 and inventory RF of 0.75, which would not induce a posting. TxDOT should review the guidance provided in MBE to decide whether to retain the same practice.

To obtain RFs for the concrete decks, alternative methods of analysis can be considered. The current method of analysis of a concrete deck in the transverse direction simplifies the situation into a one-dimensional problem for simplified analysis. However, these decks are actually acting as two-way systems, and therefore, two-way slabs analysis methods could be used. Two-way slab analysis methods such as the yield line method could significantly improve the ratings and potentially show that the decks are adequate for the HS20 load. Plastic analysis methods such as moment redistribution or finite element analysis methods could also be used. Finally, the *AASHTO LRFD Specifications* discuss the empirical design method for concrete decks in Article 9.7.2 (AASHTO 2014). The empirical design method essentially allows one to design a concrete slab by only ensuring that minimum areas of reinforcement are met in each layer of steel. If an in-situ bridge contains these areas of reinforcement, then it is safe for the LRFD HL93 design load, and therefore the HS20 design load. There are certain requirements that a bridge must meet to be able to use the empirical design method, but, for example, the deck of Bridge SM-15 is found to be safely designed when using the empirical design method. In conclusion, the deck ratings can likely be increased using one or more alternative methods of analysis. This fact, along with consideration of the condition of the deck and its length of service, suggests that the decks of the selected steel multi-girder bridges will not control the rating in almost all cases.

6 SSLO CONCRETE BRIDGES

6.1 SIMPLE-SPAN CONCRETE SLAB BRIDGES

6.1.1 Introduction

Twenty-three concrete slab bridges were selected from the inventory of SSLO concrete slab bridges in Texas for basic load rating evaluation. Table 6.1 summarizes these bridges, along with their main characteristics.

The maximum span length corresponds to the simply supported span that governs the rating calculations. Half of the bearing length at each end of the span is deducted from the span length to determine the center-to-center bearing span length of the bridge. The deck width, slab thickness, and wearing surface thickness are obtained from the structural drawings, if available, or from field measurements recorded in the TxDOT inspection reports. The yield strength of steel and the compressive strength of concrete are properties extracted from the rating calculations provided in the inspection records of the bridges. The skew is defined as the angle between the line normal to the centerline of the roadway and the centerline of the pier cap or abutment, as shown in Figure 6.1. The skew angle affects the moment and shear demands.

There are two types of concrete slab bridges, one with structural curbs and one without structural curbs. According to the *TxDOT Rate Spreadsheet User Guide* (TxDOT 2001), the bridges denoted as “FS slabs” have structural curbs. These curbs contribute to the load carrying capacity of the bridge and thus are considered in the rating calculations. Illinois Bulletin 346 (IB346) (Jenson et al. 1943) documents the design procedure for bridge with integral curbs and/or beams. All other concrete slab bridges are considered to have non-structural slabs. It should be noted that complete information regarding all the bridges was not available, especially for the off-system bridges.

Table 6.1. Characteristics of the Selected SSLO Simple-Span Concrete Slab Bridges

ID	On/Off System	Year Built	Max. Span Length (ft)	Deck Width (ft)	Yield Strength (ksi)	Compressive Strength (ksi)	Slab Thickness (in.)	Wearing Surface Thickness (in.)	Skew Angle (deg.)
CS-1	Off	1920	18	26.8	33*	2.5	15.75	1.0	0
CS-2	Off	1930	19.5	21.4	30	2.5	19.0	0.5	0

CS-3	Off	1935	22	22.8	33	2.5	19.25	4.0	0
CS-4	On	1943	23	46.0	40*	3.0*	15.5	2.0	45
CS-5	On	1946	25	46.3	33	3.0*	12.0	1.25	0
CS-6	Off	1947	23	25.3	33	2.5	10.0	12.0	0
CS-7	Off	1947	25	25.3	33	3.0*	10.0	14.0	0
CS-8	On	1947	25	25.3	33	3.0*	12.0	6.0	0
CS-9	On	1948	25	21.3	33	2.5	11.0	3.0	0
CS-10	On	1949	25	44	33	3.0*	16.0	6.0	30
CS-11	On	1949	25	44	33	3.0*	16.0	6.0	30
CS-12	On	1950	25	21.3	33	3.0*	11.0	3.0	0
CS-13	On	1951	25	21.3	38*	2.5	11.0	2.5	0
CS-14	On	1951	25	25.4	40*	3.0*	11.0	0.5	0
CS-15	On	1951	25	25.3	40*	3.0*	11.0	0.5	0
CS-16	On	1951	25	25.3	40*	3.0*	11.0	0.5	0
CS-17	On	1952	25	21.3	33	3.0*	11.0	2.0	0
CS-18	On	1952	25	36.3	33	3.0*	11.0	1.5	45
CS-19	On	1953	25	35.3	33	2.5	11.0	0.5	0
CS-20	Off	1955	20	21.8	33	2.5	12.0	4.0	0
CS-21	Off	1960	25	25.3	33	3.0	14.0	-	45
CS-22	Off	1962	20	33.8	33	3.0	11.0	5.0	99
CS-23	Off	1970	25	31.0	36	3.0	15.0	5.0	30

Notes:

1. Steel yield strength is the value used by TxDOT for load rating calculations, which typically matches the value given in the *TxDOT Bridge Inspection Manual* (TxDOT 2018a) and the AASHTO MBE (AASHTO 2018) unless the mill certificate is available.
2. *Material strengths taken from the TxDOT load rating calculations.

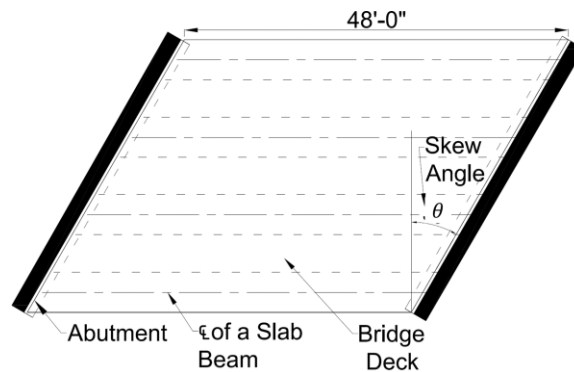


Figure 6.1. Skewed Bridge (Terzioglu et al. 2017)

6.1.2 Basic Load Rating Analysis

6.1.2.1 General

Each concrete slab bridge was rated using the three methods defined in MBE: ASR, LFR, and LRFR. The three rating methods follow a similar general procedure. For flexure rating factors, the following steps are used:

1. The live load moments, obtained from Table E6A-1 and Table C6B-1 of the MBE (AASHTO 2018), are distributed over an equivalent width of the slab to determine the maximum live load moment demands for the bridge based on the design load.
2. The dead load moments are also found by calculating an equivalent distributed load over the design width of the slab.
3. The bending moment capacities of the reinforced concrete section are calculated using the appropriate AASHTO capacity limits. The compression reinforcement is not considered in these calculations to adhere to TxDOT practice and for simplified calculations.
4. Load rating is conducted based on the three load rating procedures (ASR, LFR, and LRFR) at the inventory and operating levels.

Article 5.14.4.1 of the *AASHTO LRFD Specifications* (AASHTO 2017) states that slab bridges designed for flexure in accordance with Article 4.6.2.3 of the *AASHTO LRFD Specifications* may be satisfactory for shear. However, to provide a more complete overview of the rating process, shear rating information has been included in this report.

For shear rating factors, the following steps are used:

1. The live load shears, obtained from Table E6A-1 and Table C6B-1 of the MBE (AASHTO 2018), are distributed over an equivalent width of the slab to determine the maximum live load shear demands for the bridge based on the design load.
2. The dead load shears are also found by calculating an equivalent distributed load over the design width of the slab.
3. The shear capacities of the reinforced concrete section are calculated using the appropriate AASHTO capacity limits. The concrete slabs were noted to have no shear reinforcement. Consequently, the shear capacity of the reinforced concrete section comprises only the concrete capacity.

4. Load rating is conducted based on the three load rating procedures (ASR, LFR, and LRFR) at the inventory and operating levels.

6.1.2.2 Allowable Stress Rating and Load Factor Rating

The ASR and LFR calculations follow the requirements outlined in the *AASHTO Standard Specifications* (AASHTO 2002). The distribution width for the ASR and LFR methods is calculated according to Article 3.24.3.2 of the *Standard Specifications* as:

$$E = 4 + 0.06S \quad (6.1)$$

where:

S = Effective span length (ft).

The skew of the bridge is accounted for by reducing the effective span length of the bridge by a factor of $\cos\theta$, where θ is the skew angle. If the longitudinal tensile reinforcement for the skewed bridge runs parallel to the roadway, then the area of tensile reinforcement is reduced by a factor of $\cos^2\theta$. This practice is in accordance with Article 3.2.6 of the *AASHTO Standard Specifications*, which states that the loads must be resolved in the longitudinal and transverse directions.

In the ASR calculations, the stress limits for concrete and steel are defined. Table 6B.5.2.4.1-1 of MBE provides the maximum allowable bending unit stresses in concrete at inventory and operating levels, along with the modular ratio, for a given range of concrete ultimate strength. If the ultimate strength of concrete is unknown, the allowable unit stress based on the age of the structure can be obtained from Table 6B.5.2.4-1 of MBE. Similarly, the allowable unit stresses in tension for reinforcing steel at inventory and operating levels based on grade of steel and year of construction are provided in Table 6B.5.2.3-1 of MBE. The nominal moment capacity of the section is calculated by assuming linear elastic behavior at these stress limits and satisfying force equilibrium. Any compression steel provided within the section is neglected to abide by TxDOT practice and for simplified calculations. The flexural demand for the bridge is calculated at the midspan of the bridge, which is assumed to be the critical section for both dead load and live load. The load factor for dead load and live load demands at both inventory and operating levels is 1.0.

The concrete slab bridges were observed to have no shear reinforcement. As a result, the shear capacity of the reinforced concrete section is based on the concrete capacity. The allowable shear stress, v_c , is calculated as per Article 8.15.5.2.1 of the *AASHTO Standard Specifications* as:

$$v_c = 0.9\sqrt{f'_c} + 1,100\rho_w \left(\frac{Vd}{M} \right) \quad (6.2)$$

The shear demands for dead and live loads are calculated at the critical shear location, which is defined in Article 8.15.5.1.4 of the *AASHTO Standard Specifications* to be at a distance d from the face of the support. The load factors for dead and live demands in ASR are the same for both moment and shear calculations.

The flexural nominal capacity calculations for reinforced concrete sections in LFR are carried out based on Article 8.16.3.2 of the *Standard Specifications*. In these calculations, the compression reinforcement, if provided, is not considered in the capacity calculations to adhere to TxDOT practice and for simplified calculations. The calculations are carried out with the assumption that the tensile steel has yielded. This assumption is verified after determining the neutral axis depth of the section. The flexural demand for the bridge is calculated at the critical section for both dead load and live load. This section was assumed to be at the midspan of the bridge for the project calculations, following TxDOT practice. The load factor for dead load demands at both inventory and operating levels is 1.3, while that for live load demands is 2.17 at inventory level and 1.3 at operating level.

The shear capacity of the section is calculated according to Article 8.16.6.2.1 of the *AASHTO Standard Specifications*. It was observed from the structural drawings that shear reinforcements were not provided for concrete slab bridges. Therefore, only the concrete section contributes toward the shear capacity of the bridge. The shear demands for both dead and live loads are calculated at the critical section of the bridge, as defined in Article 8.16.6.1.2 of the *Standard Specifications*. The loads factors for dead and live demands in LFR are the same for both moment and shear calculations.

The RF for both LFR and ASR at the inventory and operating levels are calculated using Equation (4.1). The controlling RFs are multiplied by the weight of the design truck to determine the allowable gross truck weight.

There are two types of concrete slab bridges in Texas: one where the curb/parapet is not integral with the concrete slab, and the other, referred to as FS (Farm Service Road) bridges, with

integral curb/parapet and/or beams. These FS bridges were designed using the simplified procedures described in IB346 (Jenson et al. 1943). Therefore, TxDOT adopts the IB346 (Jenson et al. 1943) formulation to account for the contribution of structural curbs and beams to the bridge capacity. For these FS bridges, calculating the RFs of the slab section using the equivalent strip approach could be too conservative because the structural curbs and/or beams attract significant moment, which is not considered in the calculation of the effective width.

6.1.2.3 Illinois Bulletin 346 Method

Concrete slab bridges have been used across the United States since 1910 (Mead&Hunt 2009) due to their simple and economical design for short span bridges. TxDOT also refers to concrete slab bridges as FS bridges when they have integral curbs and/or beams. Concrete slab bridges were one of the main design and construction types between 1945 and 1965. A modified version of these constant depth slab bridges with monolithically poured curbs/parapets were called “Type FS” bridges in the TxDOT standard drawings. These integrated structural curbs/parapets were designed based on the simplified guidelines established from the findings of the research conducted at the University of Illinois (Jenson et al. 1943). It was found that by adopting integrated structural curbs (Type FS) that act as edge girders, the slab could be designed thinner than standard concrete slab bridges, making FS bridges more economical. High monolithic structural curbs in FS bridges also act as parapets and were found to be relatively simple to construct.

Several analytical and experimental research findings related to the design of FS bridges in Illinois were provided in a series of documents starting with Illinois Bulletin 346, and thus the method of analysis for FS bridges is termed IB346. The specialty of FS bridges is the additional load-carrying capacity offered by structural curbs. These structural curbs are usually 18 in. in height with #9 or #11 reinforcement (TxDOT 2001). In addition to slab bridges with structural curbs, slab bridges with reinforced edge beams were also rated using the IB346 method. TxDOT has adopted the IB346 method in lieu of the AASHTO strip method to load rate FS bridges. The *AASHTO Standard Specifications* (AASHTO 2002) also references the IB346 method in Section 3.24 for the distribution of loads and design of concrete slabs.

The structural curbs in FS bridges are large in size with significant reinforcement compared to non-structural curbs that may not be constructed monolithically with the bridge. The IB346 report (Jenson et al. 1943) provides a simplified design methodology based on experimental

evaluation of the actual behavior of several types of FS bridges. Experimental tests were conducted on a wide range of FS bridges, such as ones with no curbs, ordinary curbs, heavy curbs, and ordinary curbs with handrails, to study the sensitivity of relative stiffness of the curb and slab. These structures were eventually tested to failure under uniformly distributed loads, as well as concentrated loads, replicating the dead and vehicle live load patterns to produce maximum moments. IB346 experimental results and simplified design methodology for various FS bridges are based on the design conditions and theory developed in Illinois Bulletin 315 (Jenson 1939). The theory in IB315 considers vertical support offered by the edge beams, and the torsional stiffness of the edge curbs/beams is neglected. Moreover, the slab is assumed to be of constant thickness; any crowning on the roadway is neglected.

In the simplified analysis method, the cross-section of an FS bridge is divided into two parts: the slab and the edge curb/beam. FS bridges are assumed to be simply supported. The total moment resisted by the slab and the curb is obtained by static moments plus fractional increase in the live load moment. The total static bending moment resisted by the curb and slab in a FS simply supported bridge is given as:

$$M_{static} = m \frac{Pa}{4} + \frac{pba^2}{8} + 2 \frac{qa^2}{8} \quad (6.3)$$

where:

- m = Number of rear wheel loads (e.g., $m = 4$ for a two-lane bridge)
- P = Magnitude of rear wheel load including an allowance for impact
- p = Load per unit area between inside faces of curb
- b = Width of roadway between inside faces of curb
- a = Span of bridge from center-to-center of bearing areas
- q = Load per unit length along curb—includes dead load of curb and live load (if any)

The value of 2 in the third term of Equation (6.3) is used when the exact same curbs are present on both sides of the roadway. Based on the analysis in IB315, the moment in the curb is given as a fraction of each term in Equation (6.3) as:

$$M_{curb} = C_1 \frac{Pa}{4} + C_2 \frac{pba^2}{8} + C_3 \frac{qa^2}{8} \quad (6.4)$$

where:

$$C_1 = \left(\frac{12}{2.5 + G} \right) \frac{\left(4 - \frac{v}{a} \right)}{\left(4 + 28 \left(\frac{v}{a} \right) \right)}$$

$$C_2 = \frac{0.5 \left(\frac{a}{b} \right)}{0.47G + \sqrt[3]{1.15 + \left(\frac{a}{b} \right)^3}}$$

$$C_3 = \frac{\sqrt[3]{1.15 + \left(\frac{a}{b} \right)^3}}{0.47G + \sqrt[3]{1.15 + \left(\frac{a}{b} \right)^3}}$$

$$G = \frac{ah^3}{12I}$$

- G = Dimensionless stiffness factor, ratio of slab stiffness to curb stiffness
 I = Moment of inertia of curb gross section outside the roadway width, ft⁴
 h = Slab thickness, ft
 v = Axle width, center-to-center of truck tires (6 ft)

Therefore, the total average moment per unit width b resisted by the slab alone is the difference between Equation (6.3) and Equation (6.4)

$$M_{slab} = (m - 1.5C_1) \frac{Pa}{4b} + (1 - 2C_2) \frac{pa^2}{8} + (2 - 2C_3) \frac{qa^2}{4b} \quad (6.5)$$

The curb component is considered as an L-shaped section comprising the curb and adjoining slab of width $4h$. The bending moment resisted by this composite section is calculated as:

$$M_{L-curb} = M_{curb} + 4hM_{slab} \quad (6.6)$$

In summary, the bending moment resisted by each composite curb and slab component is calculated as:

$$M_{slab-DL} = (1 - C_{3A}) \frac{q_A a^2}{8b} + (1 - C_{2A} - C_{2B}) \frac{pa^2}{8} + (1 - C_{3B}) \frac{q_B a^2}{8b} \quad (6.7)$$

$$M_{slab-LL} = (1 - C_{3A}) \frac{(q_A)_{LL} a^2}{8b} + (m - \frac{3}{4} C_{1A}) \frac{Pa}{4b} + (1 - C_{3B}) \frac{(q_B)_{LL} a^2}{8b} \quad (6.8)$$

$$M_{L-curbA-DL} = C_{2A} \frac{pba^2}{4} + C_{3A} \frac{(q_A)_{DL} a^2}{8} + 4h(M_{slab})_{DL} \quad (6.9)$$

$$M_{L-curbA-LL} = C_{1A} \frac{Pa}{4} + C_{3A} \frac{(q_A)_{LL} a^2}{8} + 4h(M_{slab})_{LL} \quad (6.10)$$

In the above equations, the subscript *A* corresponds to the left curb, and subscript *B* corresponds to the right curb. Equations (6.9) and (6.10) can be rewritten by changing the subscript *A* to *B* for the other curb.

The capacity is calculated for each component (curb, beam, and slab portions) for bridges that have integral beams and curbs (such as FS bridges in Texas). Figure 6.2 shows typical cross-sections for each component in an FS bridge. The curb capacity is calculated by considering an effective slab width of $4h$ as part of the curb. The flexural capacity of this L-shaped section is due to the compression steel reinforcement, concrete compression stress block, and tension steel reinforcement. The slab capacity is calculated per foot width using the procedure outlined in Article 8.16.3.2 of the *AASHTO Standard Specifications*. The edge beam capacity is also calculated by considering an effective slab width of $4h$ as part of the beam. The compressive reinforcement in the beam is neglected.

Using the same rating procedure and coefficients as discussed for the ASR and LFR methods, each component is rated for inventory and operating levels. Inventory rating of the overall superstructure is the minimum of the inventory RFs of individual components. Operating rating of the overall superstructure is the weighted average, with the curb and slab rating considered proportionate to their tributary width (TxDOT 2018a).

6.1.2.4 Load and Resistance Factor Rating

The LRFR calculations follow the requirements outlined in the *AASHTO LRFD Specifications* (AASHTO 2017). The dead load moments and shears were calculated over a design strip width of

1 ft. The live load moments and shears are distributed over the equivalent strip width E (ft) defined in *AASHTO LRFD Specifications* Article 4.6.2.3, where Equation (6.11) corresponds to a single-lane loaded situation, while Equation (6.12) is for a multi-lane loaded condition.

$$E = 10.0 + 5.0\sqrt{L_1W_1} \quad (6.11)$$

$$E = 84.0 + 1.44\sqrt{L_1W_1} \leq \frac{12.0W}{N_L} \quad (6.12)$$

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

Article 4.6.2.3 of the *AASHTO LRFD Specifications* accounts for skew by reducing the longitudinal force effects by a factor of r , defined as:

$$r = 1.05 - 0.25\tan\theta \leq 1.00 \quad (6.13)$$

The flexural resistance of the rectangular section is calculated per Article 5.7.3.2.3 of the *AASHTO LRFD Specifications*, while the nominal shear capacity is calculated according to Article 5.8.3.3.

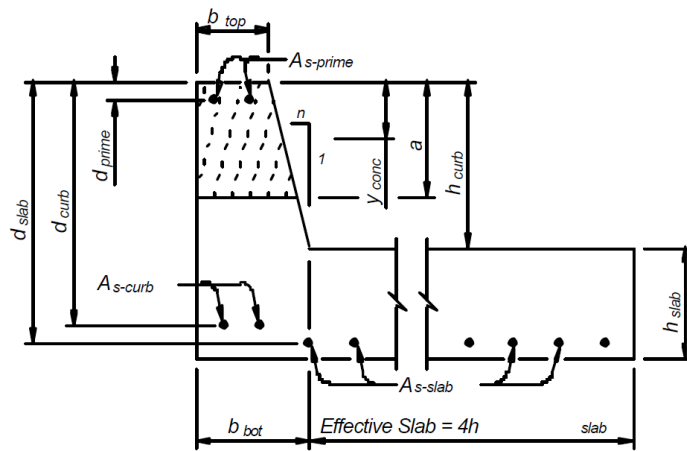
The AASHTO MBE Table 6A.4.2.2-1 defines the load factors for inventory and operating design loads for reinforced concrete bridges at the Strength I limit state. The RFs for LRFR are then calculated at the inventory and operating levels using Equation (4.2). The allowable gross truck weight is calculated by multiplying the governing RFs with the weight of the design truck.

6.1.3 Calculated Load Rating Results

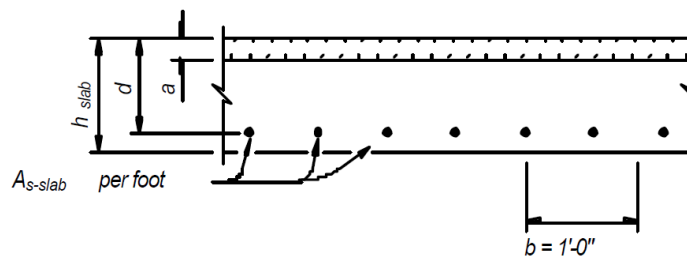
The main focus of the project basic load rating analysis is the rating of the superstructure. The 23 selected simple-span concrete slab bridges were rated using each of the three rating methods.

6.1.3.1 Flexure Analysis

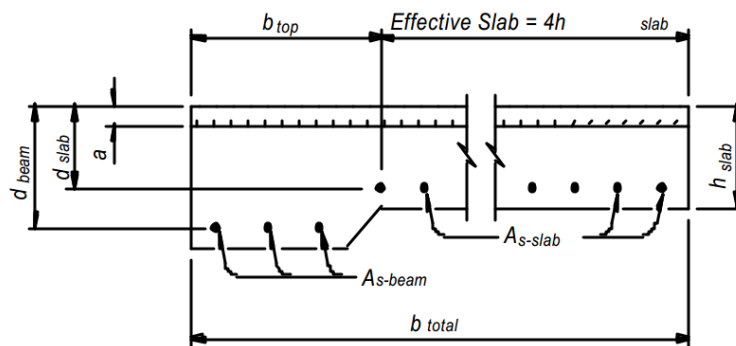
The flexure RFs for the design loads for each rating method are summarized in Table 6.2 and graphically presented in Figure 6.3. The results show that the RFs corresponding to the LRFR method are consistently lower than those RFs corresponding to the ASR and LFR methods. These results may be because the equivalent strip approach, which was deemed conservative for FS bridges, is used for the LRFR calculations. Two of the 23 bridges, CS-3 and CS-5, had both HS-20 inventory and operating RFs above 1.0, while 10 of the bridges have HS-20 operating RFs above 1.0. Bridges CS-2, CS-6, and CS-20 to CS-23 are off-system bridges with no structural drawings. TxDOT practice for rating such bridges is to use the flow charts provided in the *TxDOT Bridge Inspection Manual* (TxDOT 2018a). For on-system bridges, the flowchart shown in Figure 4.2 is followed, for off-system bridges the flowchart shown in Figure 4.3 is applied, and for concrete bridges with no plans, the flowchart shown in Figure 4.4 is used.



(a) Typical Curb Cross-Section for Capacity Calculations



(b) Typical Slab Cross-Section for Capacity Calculations



(c) Typical Beam Cross-Section for Capacity Calculations

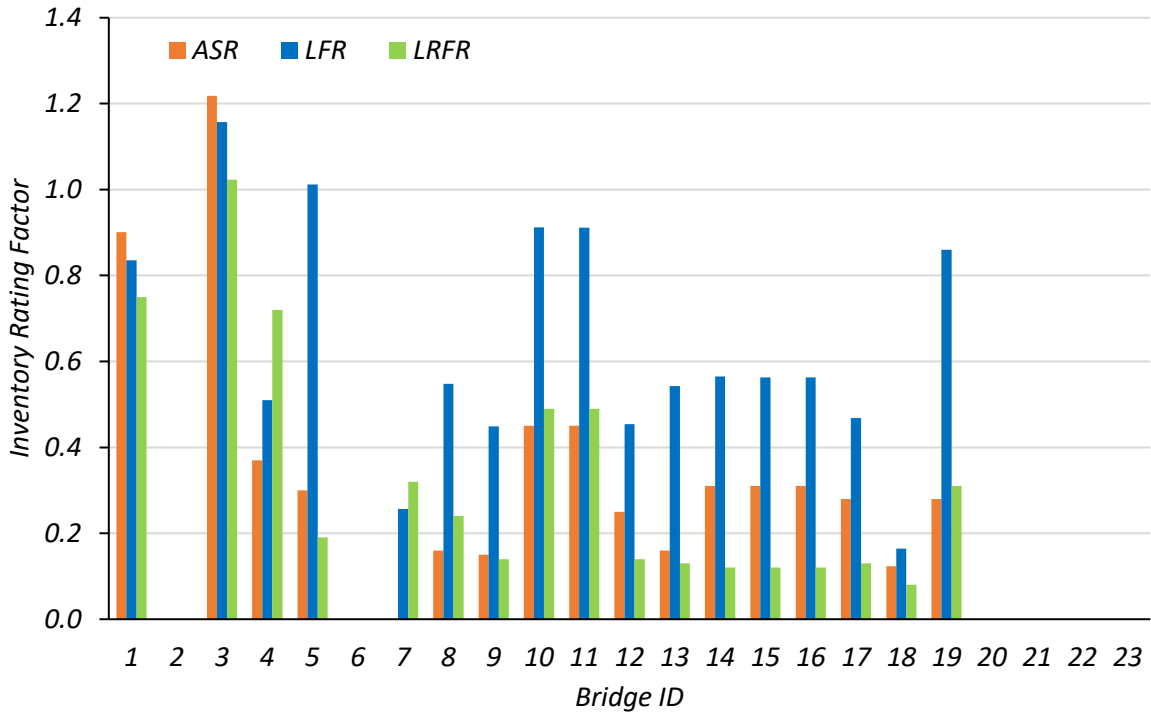
Figure 6.2. Typical Components for FS Bridges (TxDOT 2001)

Table 6.2. Flexure RFs for CS Bridges

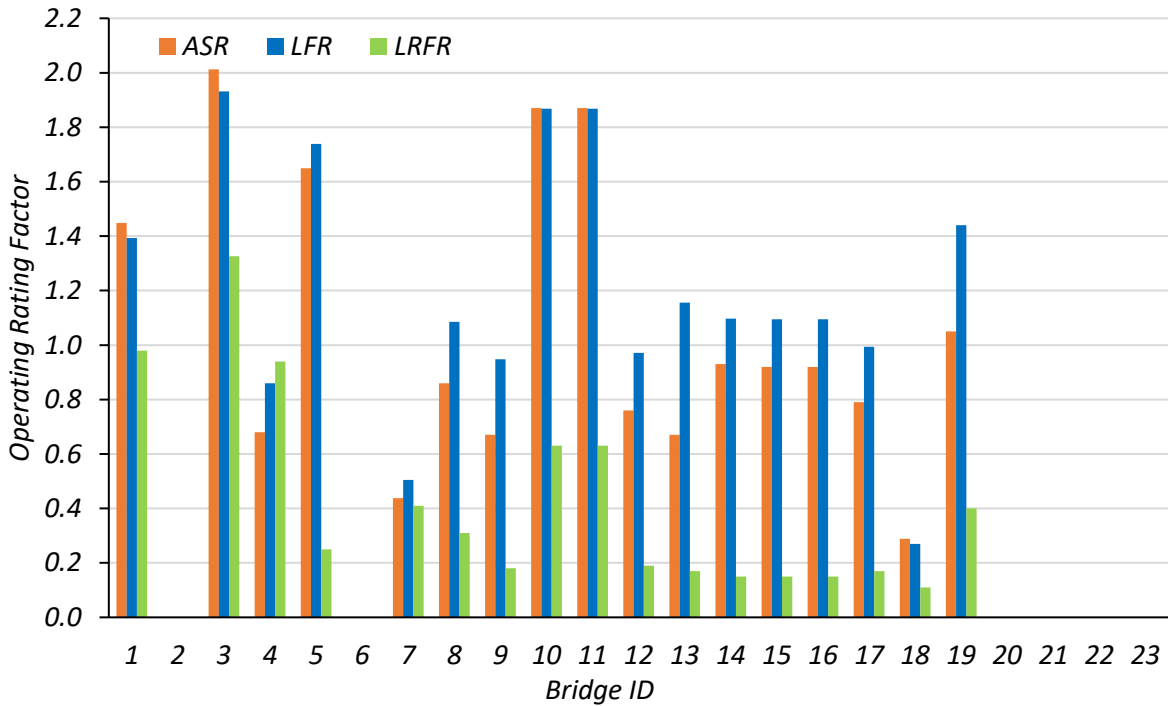
Bridge ID	FS Bridge	Inventory RF			Operating RF		
		ASR	LFR	LRFR	ASR	LFR	LRFR
CS-1		0.90	0.84	0.75	1.45	1.39	0.98
CS-2		-	-	-	-	-	-
CS-3		1.22	1.16	1.02	2.01	1.93	1.33
CS-4	x	0.37	0.51	0.72	0.68	0.86	0.94
CS-5	x	0.30	1.01	0.19	1.65	1.74	0.25
CS-6		-	-	-	-	-	-
CS-7	x	0.00*	0.26	0.32	0.44	0.51	0.41
CS-8	x	0.16	0.55	0.24	0.86	1.09	0.31
CS-9	x	0.15	0.45	0.14	0.67	0.95	0.18
CS-10	x	0.45	0.91	0.49	1.87	1.87	0.63
CS-11	x	0.45	0.91	0.49	1.87	1.87	0.63
CS-12	x	0.25	0.45	0.14	0.76	0.97	0.19
CS-13	x	0.16	0.54	0.13	0.67	1.16	0.17
CS-14	x	0.31	0.57	0.12	0.93	1.10	0.15
CS-15	x	0.31	0.56	0.12	0.92	1.10	0.15
CS-16	x	0.31	0.56	0.12	0.92	1.10	0.15
CS-17	x	0.28	0.47	0.13	0.79	0.99	0.17
CS-18	x	0.12	0.16	0.08	0.29	0.27	0.11
CS-19	x	0.28	0.86	0.31	1.05	1.44	0.40
CS-20		-	-	-	-	-	-
CS-21		-	-	-	-	-	-
CS-22		-	-	-	-	-	-
CS-23		-	-	-	-	-	-
Average		0.35	0.63	0.32	1.05	1.20	0.42
Std. Dev.		0.29	0.26	0.27	0.51	0.45	0.35

Note: A dash denotes bridges rated using procedure outlined in the *TxDOT Bridge Inspection Manual* (TxDOT 2018a) when structural drawings are not available.

* RF calculated to be negative due to high dead load demands.



(a) Inventory RFs



(b) Operating RFs

Figure 6.3. Flexure RFs for CS Bridges

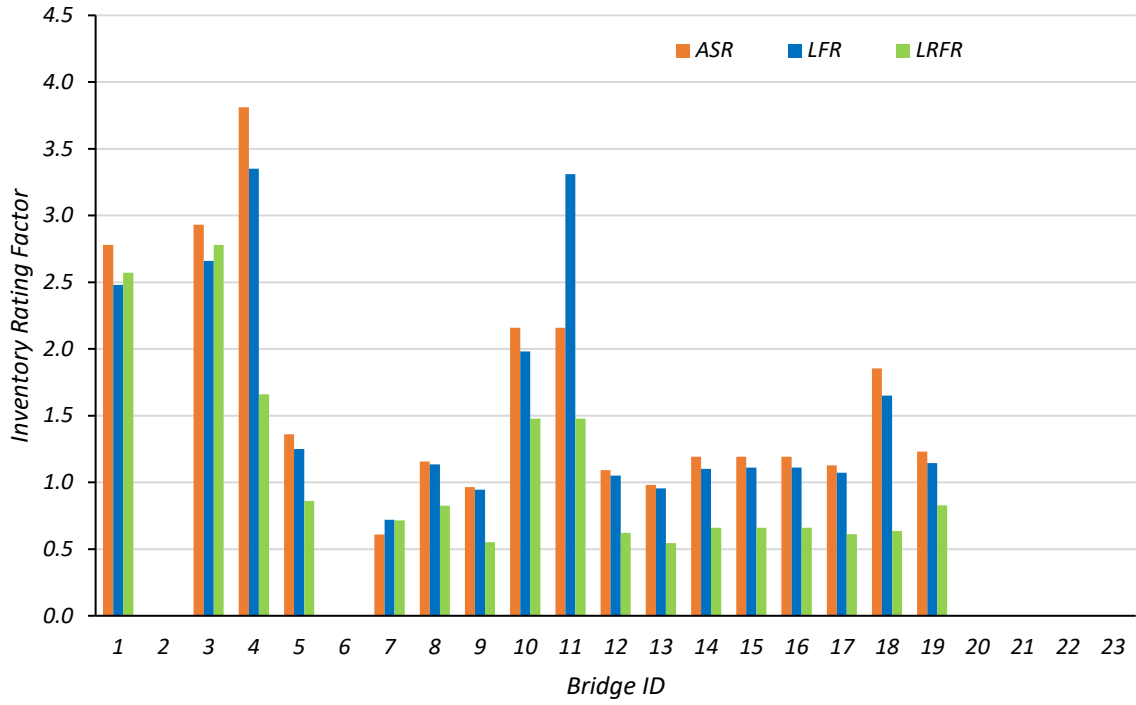
6.1.3.2 Shear Analysis

The shear RFs calculated for the selected CS bridges are shown in Table 6.3 and Figure 6.4. Again, some bridges could not be rated due to the lack of available drawings, particularly for off-system bridges. The LRFR method resulted in higher shear inventory and operating RFs for all the bridges.

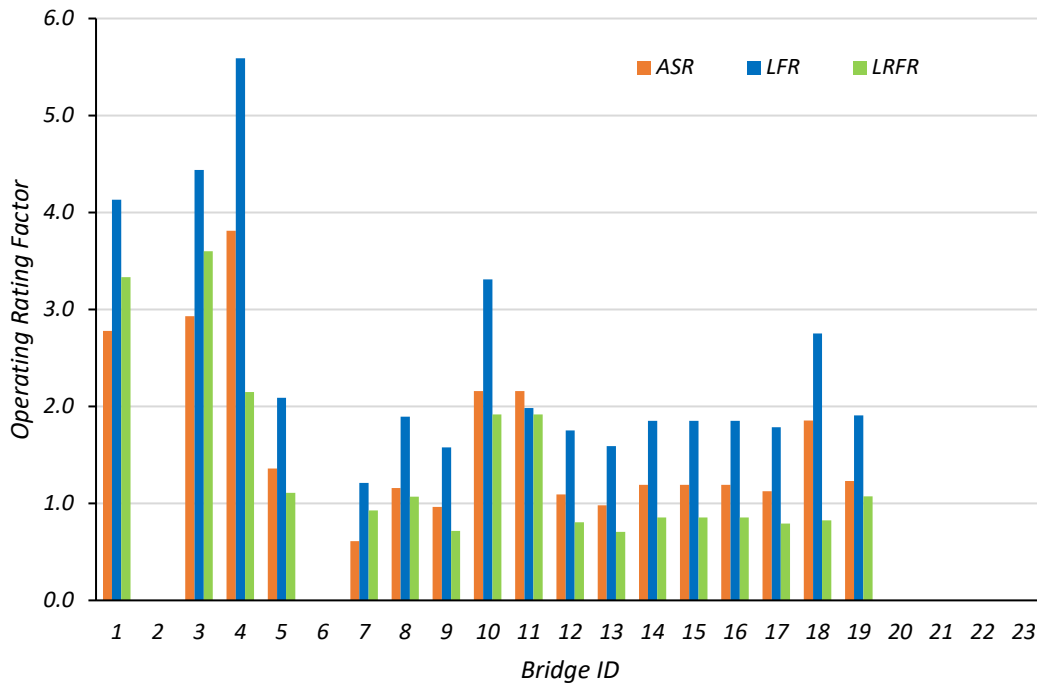
Table 6.3. Shear RFs for CS Bridges

Bridge ID	FS Bridge	Inventory RF			Operating RF		
		ASR	LFR	LRFR	ASR	LFR	LRFR
CS-1		2.78	2.48	2.57	2.78	4.13	3.33
CS-2		-	-	-	-	-	-
CS-3		2.93	2.66	2.78	2.93	4.44	3.60
CS-4	x	3.81	3.35	1.66	3.81	5.59	2.15
CS-5	x	1.36	1.25	0.86	1.36	2.09	1.11
CS-6		-	-	-	-	-	-
CS-7	x	0.61	0.72	0.72	0.61	1.21	0.93
CS-8	x	1.16	1.14	0.83	1.16	1.89	1.07
CS-9	x	0.96	0.94	0.55	0.96	1.58	0.72
CS-10	x	2.16	1.98	1.48	2.16	3.31	1.92
CS-11	x	2.16	3.31	1.48	2.16	1.98	1.92
CS-12	x	1.09	1.05	0.62	1.09	1.75	0.81
CS-13	x	0.98	0.95	0.54	0.98	1.59	0.71
CS-14	x	1.19	1.10	0.66	1.19	1.85	0.86
CS-15	x	1.19	1.11	0.66	1.19	1.85	0.85
CS-16	x	1.19	1.11	0.66	1.19	1.85	0.85
CS-17	x	1.13	1.07	0.61	1.13	1.79	0.79
CS-18	x	1.85	1.65	0.64	1.85	2.75	0.82
CS-19	x	1.23	1.14	0.83	1.23	1.91	1.07
CS-20		-	-	-	-	-	-
CS-21		-	-	-	-	-	-
CS-22		-	-	-	-	-	-
CS-23		-	-	-	-	-	-
Average		1.63	1.59	1.07	1.63	2.45	1.38
Std. Dev.		0.83	0.82	0.68	0.83	1.17	0.88

Note: A dash denotes bridges without structural drawings.



(a) Inventory RFs



(b) Operating RFs

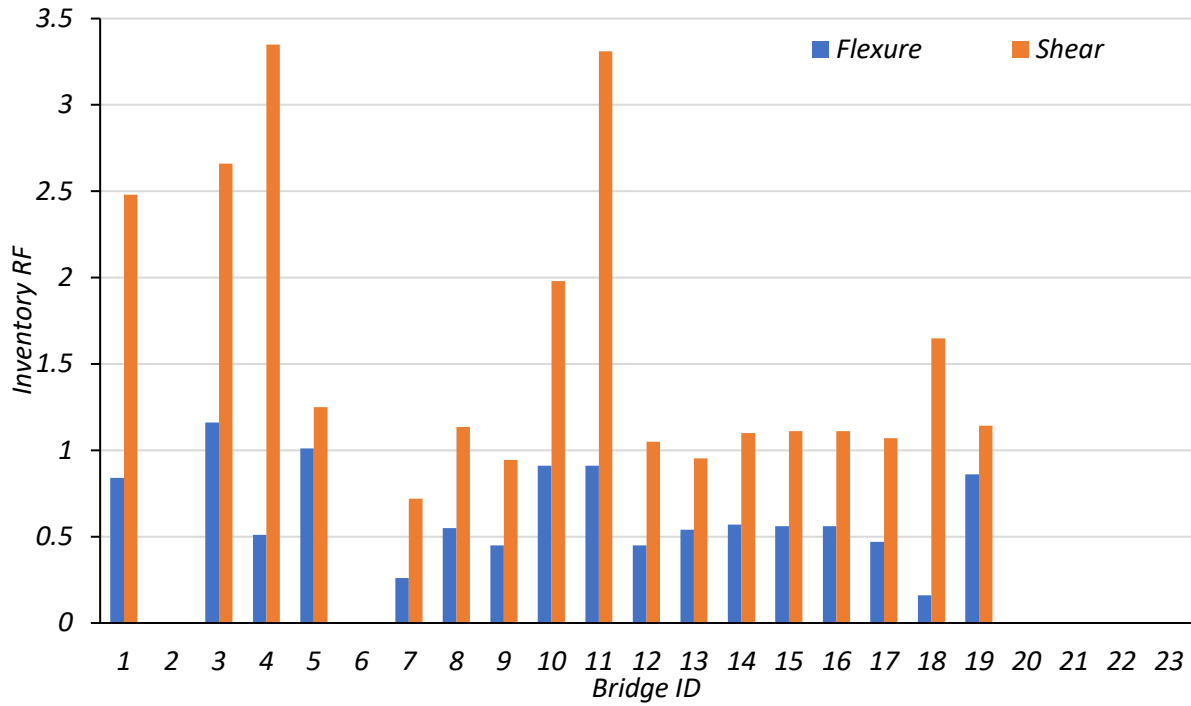
Figure 6.4. Shear RFs for CS Bridges

6.1.3.3 Comparison of Flexure and Shear RFs for CS Bridge

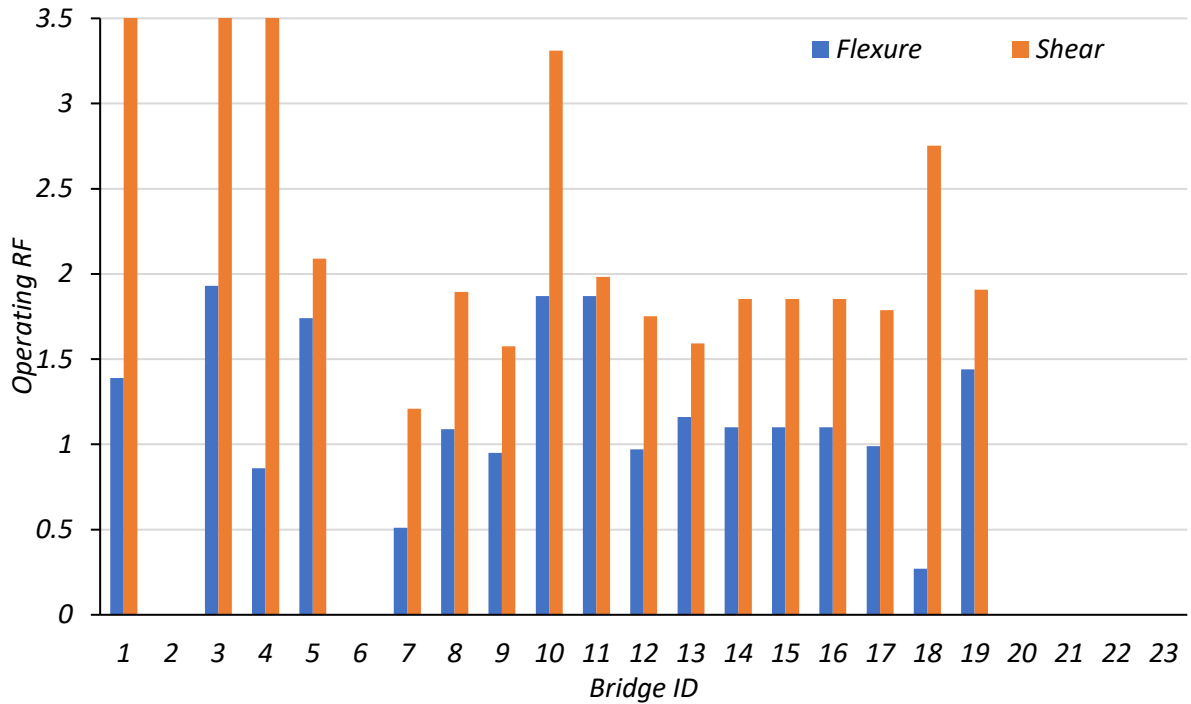
The calculated flexure RFs and shear RFs for the selected CS bridges are compared to determine the controlling RF for each bridge. Table 6.4 and Figure 6.5 show the inventory and operating flexure and shear RFs found using the same rating method TxDOT uses in analysis. For every CS bridge, the flexure RF controlled the rating at both the inventory and operating level.

Table 6.4. Comparison of Flexure and Shear RFs for CS Bridges

ID	TxDOT Rating Method	Inventory HS20 RF		Operating HS20 RF	
		Flexure	Shear	Flexure	Shear
CS-1	LFR	0.84	2.48	1.39	4.13
CS-2	-	-	-	-	-
CS-3	LFR	1.16	2.66	1.93	4.44
CS-4	LFR	0.51	3.35	0.86	5.59
CS-5	LFR	1.01	1.25	1.74	2.09
CS-6	-	-	-	-	-
CS-7	LFR	0.26	0.72	0.51	1.21
CS-8	LFR	0.55	1.14	1.09	1.89
CS-9	LFR	0.45	0.94	0.95	1.58
CS-10	LFR	0.91	1.98	1.87	3.31
CS-11	LFR	0.91	3.31	1.87	1.98
CS-12	LFR	0.45	1.05	0.97	1.75
CS-13	LFR	0.54	0.95	1.16	1.59
CS-14	LFR	0.57	1.10	1.1	1.85
CS-15	LFR	0.56	1.11	1.1	1.85
CS-16	LFR	0.56	1.11	1.1	1.85
CS-17	LFR	0.47	1.07	0.99	1.79
CS-18	LFR	0.16	1.65	0.27	2.75
CS-19	LFR	0.86	1.14	1.44	1.91
CS-20	-	-	-	-	-
CS-21	-	-	-	-	-
CS-22	-	-	-	-	-
CS-23	-	-	-	-	-
Avg.		0.63	1.59	1.20	2.45
Std. Dev.		0.26	0.82	0.45	1.17



(a) Inventory RFs



(b) Operating RFs

Figure 6.5. Comparison of Flexure and Shear RFs for CS Bridges

6.1.4 Check of CS Bridge Calculations

After conducting the rating analysis of each bridge, a check was conducted to confirm that the load rating process followed the AASHTO MBE and TxDOT practices. The calculated RFs for the CS bridges were compared to the TxDOT rating calculations by changing the input characteristics of each bridge to match the TxDOT inputs for the same rating method. A tolerance of 5 percent versus the TxDOT rating factor was adopted. Any difference within 5 percent may be attributed to slight rounding differences or other features. Table 6.5 provides the RFs TxDOT obtained for each selected bridge, the rating method used by TxDOT, and the ratio of the calculated RF to the TxDOT RF using the same rating method.

The inventory RF for Bridge CS-7 is 11 percent lower than that calculated by TxDOT because this RF corresponds to a single design lane, while the bridge operating RF corresponds to two design lanes. The calculated RF was determined for two design lanes to match the number of lanes reported in the NBI. Bridge CS-18 also has inventory and operating RFs greater than the 5 percent tolerance because the input characteristics used in the original rating calculations are not clear. However, the values match those used in the TxDOT rate spreadsheet that was provided along with the rating calculations. The TxDOT ratings for this bridge are not taken from TxDOT RATE spreadsheet calculations but are the values provided in the inspection report. Through this exercise, the calculated ratings were found to be close to the TxDOT ratings, thus confirming the process used in the rating calculations.

Table 6.5. RFs for CS Bridges with Adjusted Characteristics

Bridge ID	Rating Method	Adjusted Calculations for HS20 RF		Adjusted Calculations RF/ TxDOT RF	
		Inventory	Operating	Inventory	Operating
CS-1	LFR	0.88	1.47	1.00	1.00
CS-2	Flowchart	-	-	-	-
CS-3	LFR	1.21	2.01	1.00	1.00
CS-4	LFR	0.52	0.87	1.00	1.00
CS-5	LFR	0.43	0.72	1.03	1.03
CS-6	Flowchart	-	-	-	-
CS-7	LFR	0.27	0.51	0.79	1.01
CS-8	LFR	0.54	1.07	1.00	1.00
CS-9	LFR	0.45	0.94	1.00	1.00

CS-10	LFR	0.52	0.87	0.97	0.98
CS-11	LFR	0.52	0.87	0.98	0.98
CS-12	LFR	0.46	0.97	1.00	1.00
CS-13	LFR	0.54	1.14	1.00	1.00
CS-14	LFR	0.57	0.96	1.01	1.00
CS-15	LFR	0.57	0.96	1.00	1.00
CS-16	LFR	0.57	0.96	1.00	1.00
CS-17	LFR	0.47	0.98	1.00	1.00
CS-18	LFR	0.47	1.48	1.06	1.95
CS-19	LFR	0.51	0.98	1.00	1.01
CS-20	Flowchart	-	-	-	-
CS-21	Flowchart	-	-	-	-
CS-22	Flowchart	-	-	-	-
CS-23	Flowchart	-	-	-	-
Average		0.56	1.04	0.99	1.06
Std. Dev.		0.20	0.33	0.05	0.22

Note: Flowchart refers to the method of rating off-system bridges without structural drawings using the *TxDOT Bridge Inspection Manual* (TxDOT (2018a) (refer to Figure 4.4).

6.1.5 Comparison of Calculated Ratings with TxDOT Ratings

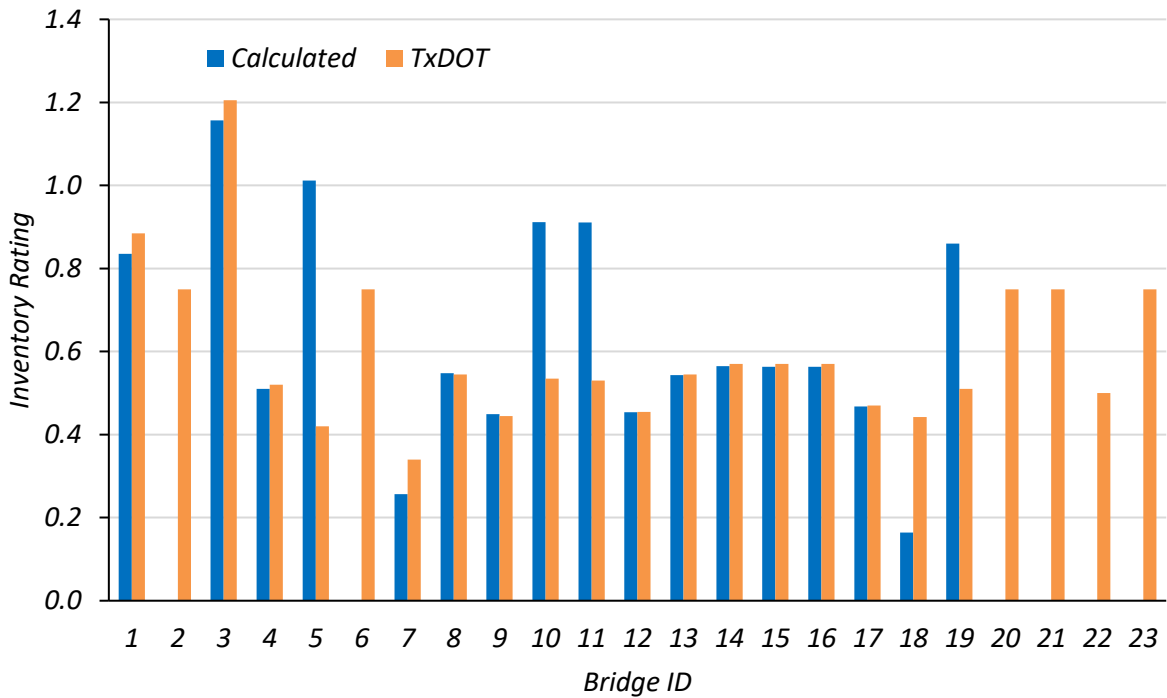
After confirming the rating process, the original calculated RFs were compared to the RFs that TxDOT documents in its inspection reports. Bridges CS-2, CS-6, and CS-20 through CS-23 were load rated following the procedure outlined in the *TxDOT Bridge Inspection Manual* (TxDOT 2018a), and results are shown in Figure 4.4. Table 6.6 and Figure 6.6 present the flexure RFs obtained by TxDOT, the rating method used, and the calculated RFs.

Fourteen out of the 23 selected CS bridges had operating ratings greater than or equal to TxDOT's rating, of which nine also had inventory ratings greater than or equal to that of TxDOT's rating. Three of the bridges with lower design-level ratings than TxDOT's had both inventory and operating ratings lower than TxDOT's, while five had inventory ratings lower than TxDOT's. Three of these bridges resulted in a ratio of 0.99 of the calculated RF to TxDOT's RF, indicating the ratings are almost the same.

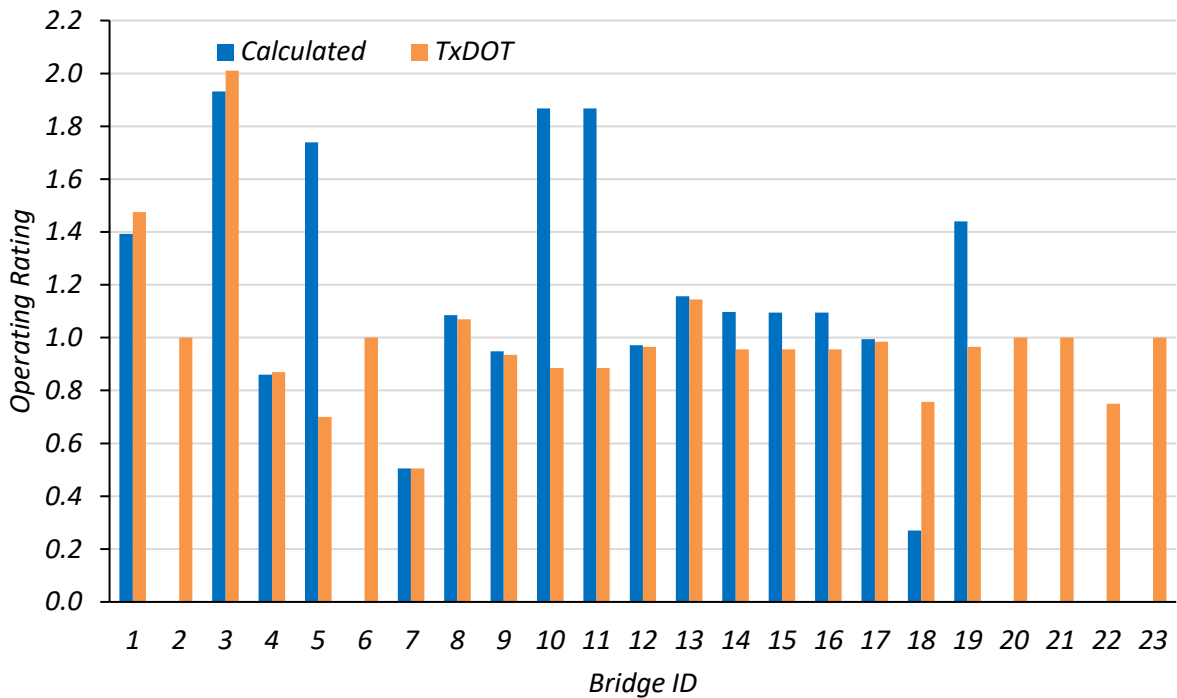
Table 6.6. Comparison of Flexure RFs for CS Bridges to TxDOT RFs

ID	Calculated HS20 RF		TxDOT HS20 RF		Rating Method	Calculated/TxDOT	
	Inventory	Operating	Inventory	Operating		Inventory	Operating
CS-1	0.84	1.39	0.89	1.48	LFR	0.94	0.94
CS-2	-	-	0.75	1.00	Flowchart	-	-
CS-3	1.16	1.93	1.21	2.01	LFR	0.96	0.96
CS-4	0.51	0.86	0.52	0.87	LFR	0.98	0.99
CS-5	1.01	1.74	0.42	0.70	LFR	2.41	2.48
CS-6	-	-	0.75	1.00	Flowchart	-	-
CS-7	0.26	0.51	0.34	0.51	LFR	0.76	1.00
CS-8	0.55	1.09	0.55	1.07	LFR	1.01	1.01
CS-9	0.45	0.95	0.45	0.94	LFR	1.01	1.01
CS-10	0.91	1.87	0.54	0.89	LFR	1.70	2.11
CS-11	0.91	1.87	0.53	0.89	LFR	1.72	2.11
CS-12	0.45	0.97	0.46	0.97	LFR	1.00	1.01
CS-13	0.54	1.16	0.55	1.15	LFR	1.00	1.01
CS-14	0.57	1.10	0.57	0.96	LFR	0.99	1.15
CS-15	0.56	1.10	0.57	0.96	LFR	0.99	1.15
CS-16	0.56	1.10	0.57	0.96	LFR	0.99	1.15
CS-17	0.47	0.99	0.47	0.99	LFR	1.00	1.01
CS-18	0.16	0.27	0.44	0.76	LFR	0.37	0.36
CS-19	0.86	1.44	0.51	0.97	LFR	1.69	1.49
CS-20	-	-	0.75	1.00	Flowchart	-	-
CS-21	-	-	0.75	1.00	Flowchart	-	-
CS-22	-	-	0.50	0.75	Flowchart	-	-
CS-23	-	-	0.75	1.00	Flowchart	-	-
Average	0.63	1.20	0.60	0.99		1.15	1.23
Std. Dev.	0.26	0.45	0.19	0.28		0.46	0.51

Note: Flowchart refers to the method of rating off-system bridges without structural drawings using the *TxDOT Bridge Inspection Manual* (TxDOT (2018a) (refer to Figure 4.4).



(a) Inventory RFs



(b) Operating RFs

Figure 6.6. Calculated RF vs. TxDOT RF for Flexure of CS Bridges

Table 6.7 summarizes the major reasons for differences between the calculated RFs and the TxDOT RFs. Additional details are provided in the following subsections.

Table 6.7. Reasons for Differences Between Calculated and TxDOT Flexure RFs for CS Bridges

Bridge ID	Reasons for Difference in Rating Factor					
	Dead Load Components	Skew Effect	Slab Tensile Reinforcement	Curb Reinforcement	Curb Reinforcement Cover	Illinois Bulletin 346
CS-1	x					
CS-2						
CS-3	x					
CS-4		x				
CS-5						x
CS-6						
CS-7	x		x	x	x	
CS-8			x	x	x	
CS-9			x	x	x	
CS-10		x	x			
CS-11		x	x			
CS-12			x		x	
CS-13			x		x	
CS-14			x	x	x	
CS-15			x	x	x	
CS-16			x	x	x	
CS-17			x	x	x	
CS-18		x				x
CS-19						x
CS-20						
CS-21						
CS-22						
CS-23						

6.1.5.1 Comparison of Ratings for Bridges CS-1 and CS-3

For Bridges CS-1 and CS-3, the calculated RFs account for non-structural curb self-weight that was not considered in TxDOT's calculations.

6.1.5.2 Comparison of Ratings for Bridge CS-4

Bridge CS-4 is a skewed bridge with non-structural curbs on either side that were demolished when widening work was done. In addition, to account for skew effects, the effective span (the span after deducting the total bearing length) is reduced by a factor of $\cos\theta$ to determine the calculated RFs. However, the TxDOT RATE spreadsheet multiplies the span length by the factor $\cos\theta$ and then subtracts the total bearing length from this value.

6.1.5.3 Comparison of Ratings for Bridge CS-5

Although Bridge CS-5 is classified as an FS bridge, the calculations did not use the IB346 approach. As a result, the TxDOT RFs are smaller than the project-calculated RFs, which are based on the bulletin.

6.1.5.4 Comparison of Ratings for Bridge CS-7

The calculated inventory RF for Bridge CS-7 was lower than that obtained by TxDOT because TxDOT reported the inventory rating for a one-lane loaded scenario, while the operating rating corresponds to two-lanes loaded. The calculated inventory RF, which was calculated to be negative due to the thick asphalt layer (14 in.), corresponds to a two-lane loaded scenario, as recorded in the NBI. The clear roadway width is 23.3 ft, and it can be seen from the photos that there is a line dividing the road into two lanes. Additionally, the area of tensile reinforcement for the slab, the areas of tensile and compressive reinforcement for the curb, and the cover to the compressive reinforcement in the curb in the TxDOT rating were different than the ones in the drawing. The cover to the curb compressive reinforcement was not detailed in the drawing. The project calculation was carried out with the assumption that this cover would be the same as the cover to the compressive reinforcement in the slab.

6.1.5.5 Comparison of Ratings for Bridges CS-8, CS-9 and CS-17

The area of tensile reinforcement for the slab, the areas of tensile and compressive reinforcement for the curb, and the cover to the compressive reinforcement in the curb in the TxDOT calculations were different than the ones called out in the drawing for these FS bridges. The cover to the curb compressive reinforcement was not called out in the drawing. The calculation was carried out with the assumption that this cover would be the same as the cover to the compressive reinforcement in the slab.

6.1.5.6 Comparison of Ratings for Bridges CS-10 and CS-11

Although these skewed bridges were identified as FS bridges, it was noted that the curbs were removed during the widening of the bridge. TxDOT rated the bridges using the area of tensile reinforcement of the widened section but still used the cover for the original bridge. The project

calculations rated the original bridge and the widened section with their respective reinforcement details and reported the minimum RFs, in this case corresponding to the original bridge. Moreover, to account for skew effects, the center-to-center of bearing length is reduced by a factor of $\cos\theta$. However, the TxDOT RATE spreadsheet multiplies the span length by the factor of $\cos\theta$ and then subtracts the bearing length from this value, which slightly changes the RFs.

6.1.5.7 Comparison of Ratings for Bridges CS-12 and CS-13

The area of tensile reinforcement for the slab and the cover to the compressive reinforcement in the curb in the TxDOT calculations were different than the ones called out in the drawing. The cover to the curb compressive reinforcement was not called out in the drawing. The calculation was carried out with the assumption that this cover is the same as the cover to the compressive reinforcement in the slab.

6.1.5.8 Comparison of Ratings for Bridges CS-14, CS-15 and CS-16

The area of tensile reinforcement for the slab, the areas of tensile and compressive reinforcement for the curb, and the cover to the compressive reinforcement in the curb in the TxDOT calculations were different than the ones called out in the drawing for these FS bridges. The cover to the curb compressive reinforcement was not provided in the drawing. The calculation was carried out with the assumption that this cover would be the same as the cover to the compressive reinforcement in the slab. Additionally, the slab thickness used in the TxDOT calculations was less than what was noted in the drawing.

6.1.5.9 Comparison of Ratings for Bridges CS-18

This skewed FS bridge was widened at both ends by demolishing the curbs and introducing a beam edge. The drawings do not call for any kind of continuity between the beam and the existing slab edge. However, TxDOT rating calculations considers the beams at either end to contribute to the rating of the existing bridge based on the IB346 approach. The project calculations provided in this report consider the original slab without curbs and beams. In addition, to account for skew effects, the effective span is reduced by a factor of $\cos\theta$. However, the TxDOT RATE spreadsheet multiplies the span length by the factor of $\cos\theta$ and then subtracts the total bearing length from this value.

6.1.5.10 Comparison of Ratings for Bridges CS-19

This FS bridge was widened on one side by demolishing the curb and introducing a beam edge. The drawings do not call for any continuity between the beam and the existing slab edge. However, the TxDOT rating calculations consider the beam along one edge to contribute to the rating of the existing bridge based on the IB346 approach. The project calculations in this report consider the original slab with a structural curb along one edge and no structural component along the second edge.

6.1.6 Conclusion

The selected simple-span concrete slab bridges were analyzed according to the *AASHTO MBE*, *AASHTO Standard Specifications*, and *AASHTO LRFD Specifications*. The LFR method resulted in higher flexure RFs, followed by the ASR and LRFR methods. The difference between the calculated RFs and the TxDOT RFs for each bridge was examined and explained. Based on the analysis performed on these bridges, possible areas for improvement to load posting are identified and elaborated upon in Section 6.3 of this report.

6.2 SIMPLE-SPAN CONCRETE MULTI-GIRDER BRIDGES

6.2.1 Introduction

Fourteen concrete multi-girder bridges were selected from the inventory of SSLO concrete multi-girder bridges in Texas. These bridges, along with their main characteristics, are summarized in Table 6.8. It should be noted that all selected bridges have two traffic lanes.

A concrete pan girder bridge consists of girders running along the longitudinal direction at specific spacings. Figure 6.7 shows a typical cross-section of such a bridge. The slab + beam depth is the total depth from the top of the slab to the bottom of the beam. This is either 24 in. or 33 in. for the selected bridges that are classified as concrete pan girder bridges. The girders for all the selected bridges were spaced at 36 in. center-to-center. All the information regarding the bridges was obtained from either the TxDOT inspection record or from the structural drawings when available. Similar to the concrete slab bridges, complete information regarding all the bridges could not be obtained, especially for the off-system bridges.

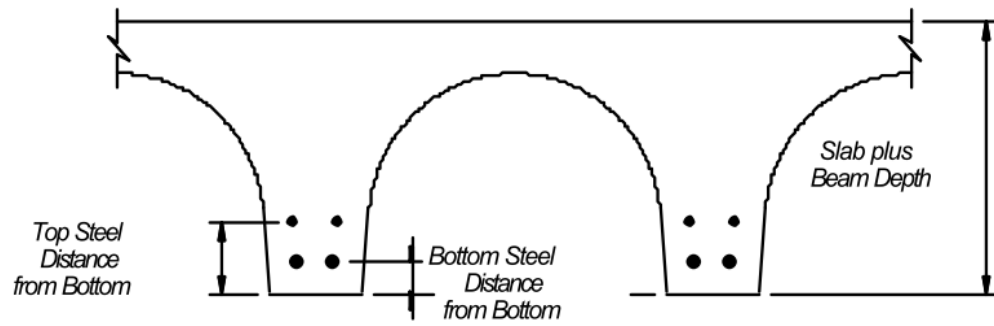


Figure 6.7. Typical Cross-Section of Concrete Pan Girder Bridge (TxDOT 2001)

Table 6.8. Selected SSLO Concrete Simple-Span Multi-girder Bridges and Characteristics

ID	On/Off System	Year Built	Max. Span Length	Deck Width	Yield Strength	Compressive Strength	Slab + Beam Depth	Spacing of Girders	Number of Girders	Wearing Surface Thickness
			(ft)	(ft)	(ksi)	(ksi)	(in.)	(in.)		(in.)
CM-1	On	1940	30	30.5	33	2.5	24	36	5	0.5
CM-2	Off	1942	30	35.5	33	2.5	24	36	12	0.5
CM-3	Off	1950	29	21.7	33	2.5	24	36	8	2
CM-4	Off	1950	29	21.7	33	2.5	24	36	8	1.5
CM-5	Off	1950	29	21.625	33	2.5	24	36	8	2
CM-6	On	1957	30	24.7	40*	3.0*	24	36	9	1.5
CM-7	Off	1957	40	31.2	33	2.5	33.5	36	11	3
CM-8	Off	1960	29	21.7	33	3.0	24	36	8	3
CM-9	Off	1960	30.3	24.7	33	3.0	-	36	9	-
CM-10	Off	1960	30.3	32.9	33	3.0	-	36	11	3
CM-11	Off	1960	40	48.1	33	3.0	33	36	17	-
CM-12	Off	1975	30.3	24.8	36	3.0	24	36	9	3
CM-13	Off	1982	40	28.3	40*	3.0	33	36	9	-
CM-14	Off	2000	30	32	36	3.0	-	36	11	3

Notes:

1. Steel yield strength is the value used by TxDOT for load rating calculations, which typically matches the value given in the *TxDOT Bridge Inspection Manual* (TxDOT 2018a) and the AASHTO MBE (AASHTO 2018) unless mill certificate is available.
2. *Material strengths taken from TxDOT's load rating calculations.

6.2.2 Basic Load Rating Analysis

6.2.2.1 General

Each of the selected concrete multi-girder bridges was rated using the three methods defined in MBE: ASR, LFR, and LRFR, provided sufficient information was available. The following steps are used:

1. The live load moments and shears are extracted from the AASHTO MBE table (MBE 2018 Tables E6A-1 and C6B-1) (AASHTO 2018).
2. The appropriate LLDFs are determined. The LLDFs, along with the appropriate impact factor, are applied to the live load moments and shears to calculate the maximum live load moments and shears applied to the bridge for the load being analyzed.

3. The dead load moments and shears from the pan girders and wearing surface are calculated over the corresponding effective widths.
4. The bending moment and shear capacities of the pan girder are calculated with the assumption of a tee beam. Compression reinforcement is not considered in these calculations to maintain consistency with TxDOT practice and for simplified calculations.
5. Load rating is carried out for the three load rating procedures (ASR, LFR, and LRFR) at the operating and inventory levels.

6.2.2.2 Allowable Stress Rating and Load Factor Rating

The procedure outlined in the *AASHTO Standard Specifications* (AASHTO 2002) is followed to carry out the ASR and LFR calculations. The effective width is defined according to Article 8.10.1.1 of the *AASHTO Standard Specifications*. The dead load moments and shears are calculated over this effective width. Table 3.23.1 of the *AASHTO Standard Specifications* provides the LLDF for multi-girder bridges considering concrete-on-concrete T-beam bridges.

The flexural demand is calculated at the midspan of the bridge, and the appropriate load factors are applied. The load factors are presented in Table 6.9. For the ASR method, the load factor for dead load and live load demands at both inventory and operating levels is 1.0. For the LRF method, the load factor for dead load demands at both inventory and operating levels is 1.3, while that for live load demands is 2.17 at the inventory level and 1.3 at the operating level.

The stress limits for concrete and steel are defined for the ASR method. The allowable bending unit stresses in concrete along with the modular ratio at inventory and operating levels for a given range of concrete ultimate strength are provided in Table 6B.5.2.4.1-1 of MBE. This information can also be obtained from Table 6B.5.2.4-1 of MBE based on year of construction when ultimate strength of concrete is unknown. The allowable unit stresses in tension for reinforcing steel at inventory and operating level based on grade of steel and year of construction are provided in Table 6B.5.2.3-1 of MBE. Linear elastic behavior at these stress limits is assumed, and force equilibrium is satisfied while calculating the moment capacity of the section at the allowable stress limit. In these calculations, the compression steel is not considered in order to remain consistent with TxDOT practice and to simplify the calculations.

The nominal flexural capacity calculations for concrete multi-girder bridges are carried out based on Article 8.16.3.3 of the *AASHTO Standard Specifications*. The pan girders in the selected

SM bridges generally have two layers of tensile reinforcement. The adjusted centroid of the reinforcement layers is used to calculate the nominal moment capacity for the LFR method. The calculations are carried out with the assumption that the neutral axis lies within the compression flange. Once this assumption is verified, the section is treated as a rectangular reinforced beam with width equal to the effective flange width.

The shear demands for dead and live loads are calculated at the critical shear location for the bridges, which is defined in Article 8.15.5.1.4 of the *AASHTO Standard Specifications* to be at a distance d from the face of the support. The load factors for dead and live demands for ASR ratings are the same for both moment and shear calculations (see Table 6.9).

The concrete pan girder bridges with structural drawings were noted to have shear reinforcement in the form of stirrups. Therefore, the shear capacity of the reinforced concrete section is calculated considering the contribution from both the concrete web and the stirrups, per Articles 8.15.5.2.1 and 8.15.5.3 of the *AASHTO Standard Specifications*, as:

$$v = v_c + v_s = 0.95\sqrt{f'_c} + \frac{A_v f_s}{b_w S_v} \quad (6.14)$$

where:

- A_v = Area of shear reinforcement (in²)
- b_w = Web width (in.)
- S_v = Spacing of shear reinforcement in direction parallel to the longitudinal reinforcement (in.)

Finally, the RFs for both LFR and ASR at the inventory and operating levels are calculated for both flexure and shear according to Equation (4.1). The controlling RFs are multiplied by the weight of the design truck to determine the allowable gross truck weight.

6.2.2.3 Load and Resistance Factor Rating

The requirements provided in the *AASHTO LRFD Specifications* (AASHTO 2017) are followed for the LRFR calculations. The effective width for pan girders bridges is defined in Article 4.6.2.6.1 of the *AASHTO LRFD Specifications* as the spacing between the girders. The flexural demand for both dead and live load is calculated at the midspan of the bridge. The live load demands are calculated for the HL-93 design truck. The live load flexure and shear distribution factors are obtained from Table 4.6.2.2.2b-1 and Table 4.6.2.2.3a-1, respectively, of

the *AASHTO LRFD Specifications*, considering a cast-in place concrete tee beam structure. These LLDFs, along with the impact factor, are applied to the live load demands. The moment capacity is calculated similar to the method used for a reinforced rectangular concrete section, with the assumption that the neutral axis lies within the compression flange (Article 5.7.3.2.2, *AASHTO LRFD Specifications*). This assumption is verified during the capacity analysis.

The load factor is 1.25 for dead load demands due to structural components and attachments, and 1.50 for dead load demands due to wearing surface and utilities. The load factor for live load demands is 1.75 at inventory level and 1.35 at operating level, as shown in Table 6.9. Finally, the RFs for the girders are calculated at the Strength I limit state for both inventory and operating levels. The governing RFs are multiplied with the weight of the design truck to determine the allowable gross weight for the bridge.

Table 6.9. Load Factors for Rating Methods

Rating Method	Dead Load Factor (A_1)		Live Load Factor (A_2)	
	Inventory	Operating	Inventory	Operating
	ASR	1.0	1.0	1.0
LFR	1.3	1.3	2.17	1.3
LRFR	1.25/1.50	1.25/1.50	1.75	1.35

6.2.3 Calculated Load Rating Results

6.2.3.1 Flexure Analysis

Only five of the selected CM bridges had sufficient information available to conduct load rating analysis. These bridges were load rated using each of the three rating methods, and the results are presented in Table 6.10 and Figure 6.8. Three out of the 14 CM bridges (CM-6, CM-13 and, CM-14) had drawings available, while Bridge CM-1 had some details available within TxDOT’s rating calculations. Load rating calculations for Bridge CM-5 were carried out based on information gathered from the standard drawing for this bridge type provided on the TxDOT website titled “CG 30'-4" Spans” (TxDOT 2005). It can be seen that when using the LFR method, Bridges CM-6 and CM-13 have operating RFs greater than 1.0, while Bridges CM-6 and CM-14 have both inventory and operating RFs greater than 1.0.

Table 6.10. Interior Girder Flexure RFs for CM Bridges

Bridge ID	Inventory RF			Operating RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
CM-1	0.46	0.50	0.28	0.87	0.83	0.36
CM-5	0.48	1.32	0.71	0.89	2.21	0.93
CM-6	0.52	0.99	0.55	0.98	1.65	0.71
CM-13	0.14	0.65	0.43	0.48	1.09	0.55
CM-14	0.46	1.46	0.80	0.90	2.44	1.04
Average	0.41	0.98	0.55	0.82	1.64	0.72
Std. Dev.	0.14	0.37	0.19	0.18	0.62	0.25

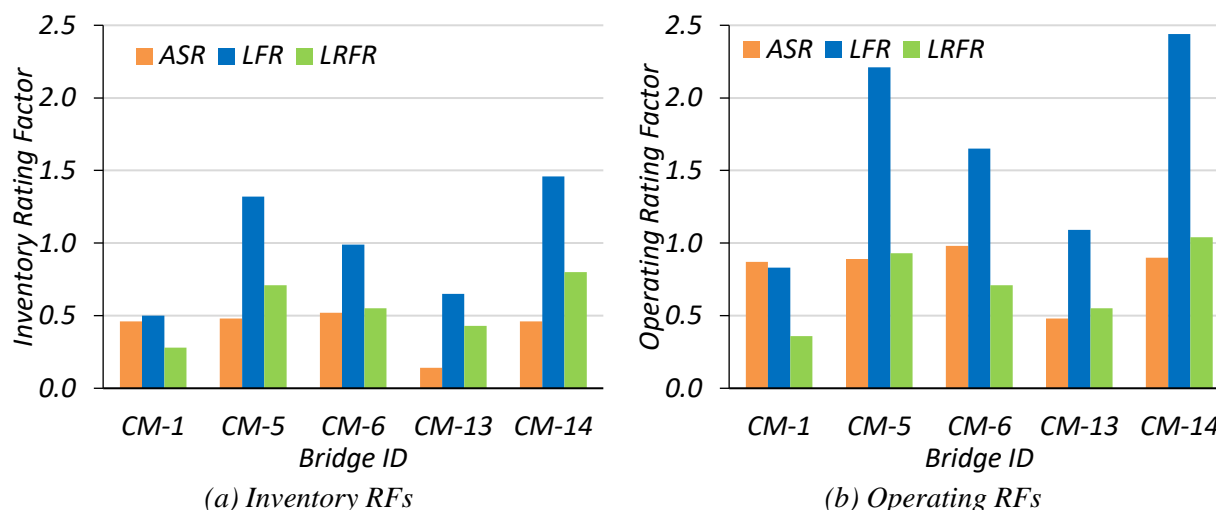


Figure 6.8. Interior Girder Flexure RFs for CM Bridges

6.2.3.2 Shear Analysis

Four out of the five CM bridges had sufficient information to carry out load rating using the three rating methods. Table 6.11 and Figure 6.9 show the shear RFs for these CM bridges.

Table 6.11. Shear RFs for CM Bridges

ID	Inventory RF			Operating RF		
	ASR	LFR	LRFR	ASR	LFR	LRFR
CM-5	1.49	1.27	0.6	1.84	2.12	0.78
CM-6	1.5	1.34	0.66	1.91	2.23	0.85
CM-13	1.58	1.43	0.59	2.01	2.39	0.76
CM-14	2.82	2.37	1.28	3.78	3.96	1.65
Average	1.85	1.60	0.78	2.39	2.68	1.01
Std. Dev.	0.56	0.45	0.29	0.81	0.75	0.37

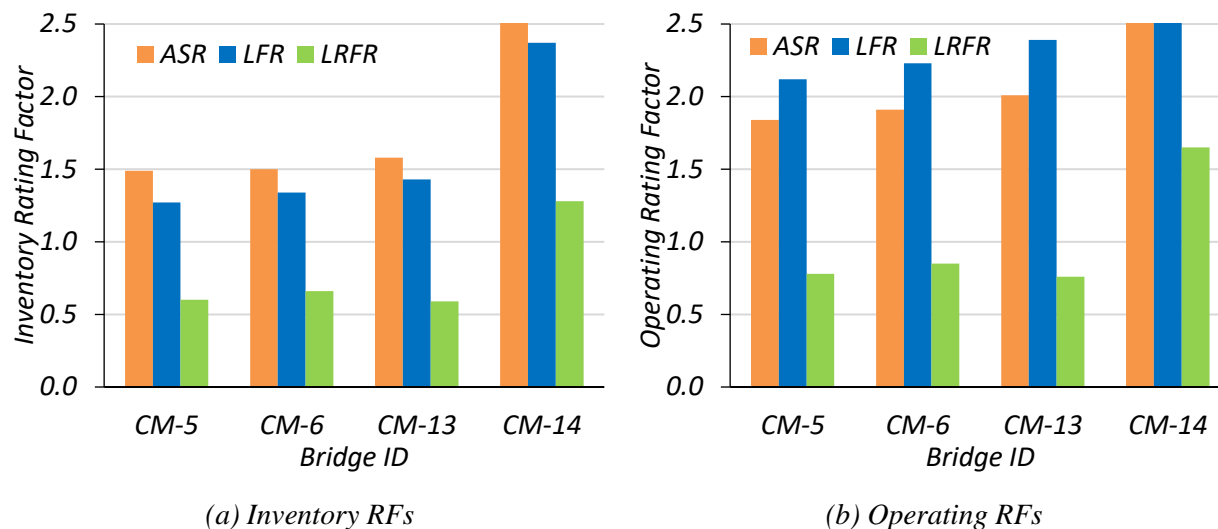


Figure 6.9. Interior Girder Shear RFs for CM Bridges

6.2.3.3 Comparison of Flexure and Shear RFs for CM Bridges

The calculated flexure RFs and shear RFs for the selected CM bridges are compared to determine the controlling RF for each bridge. The RFs for only four of the CM bridges could be compared. Table 6.12 and Figure 6.10 show the inventory and operating flexure and shear RFs found using the same rating method TxDOT uses in analysis. For every CM bridge, the flexure RF controlled the rating at both the inventory and operating level.

Table 6.12. Comparison of Flexure and Shear RFs for CM Bridges

ID	TxDOT Rating Method	Inventory HS20 RF		Operating HS20 RF	
		Flexure	Shear	Flexure	Shear
CM-5	LFR	1.32	1.27	2.21	2.12
CM-6	LFR	0.99	1.34	1.65	2.23
CM-13	LFR	0.65	1.43	1.09	2.39
CM-14	LFR	1.46	2.37	2.44	3.96
Avg.		1.03	1.71	1.73	2.86
Std. Dev.		0.33	0.47	0.55	0.78

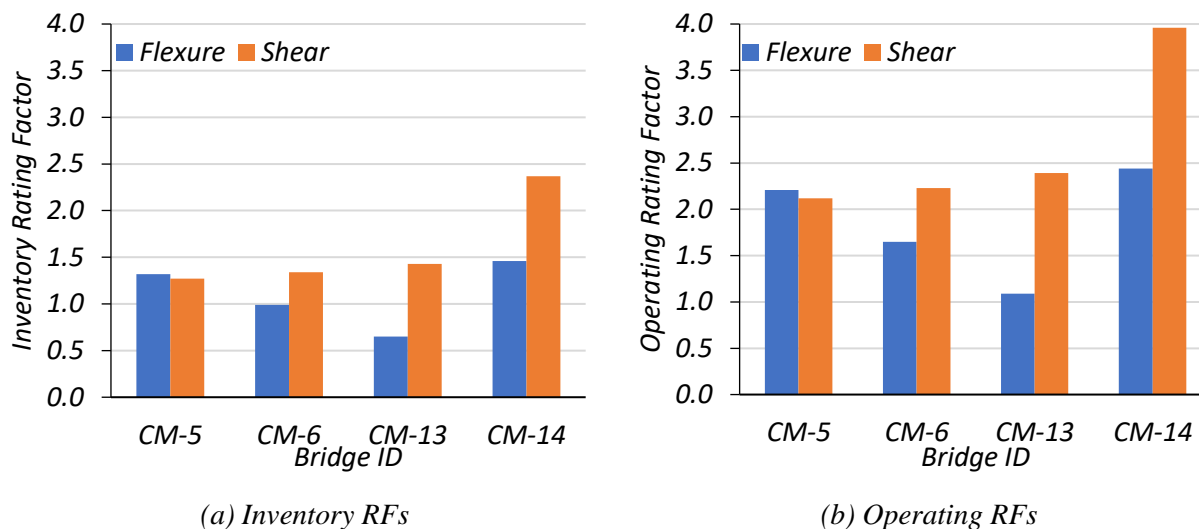


Figure 6.10. Interior Girder Flexure and Shear RFs for CM Bridges

6.2.4 Comparison of Calculated Ratings with TxDOT Ratings

The calculated flexure RFs for each concrete multi-girder bridge are compared against those provided by TxDOT in its inspection reports. Table 6.13 and Figure 6.11 present the flexure RFs obtained by TxDOT, the rating method used, and the ratio of calculated RF to TxDOT’s RF, and results show that the calculated flexure RFs closely match with the TxDOT RFs for Bridges CM-1, CM-6, and CM-13. The flexure RFs for Bridge CM-14 were higher than the ones determined by TxDOT.

Table 6.13. Comparison of Flexure RFs for CM Bridges to TxDOT RFs

ID	Calculated HS20 RF		HS20 RF (TxDOT)		TxDOT Rating Method	Calculated/TxDOT	
	Inventory	Operating	Inventory	Operating		Inventory	Operating
CM-1	0.50	0.83	0.50	0.83	LFR	0.99	1.00
CM-5	1.32	2.21	0.72	1.00	LFR	1.83	2.21
CM-6	0.99	1.65	1.00	1.66	LFR	1.00	1.00
CM-13	0.65	1.09	0.68	1.14	LFR	0.96	0.96
CM-14	1.46	2.44	1.00	1.35	LFR	1.46	1.81
Average	0.98	1.64	0.78	1.20		1.25	1.40
Std. Dev.	0.37	0.62	0.19	0.29		0.35	0.52

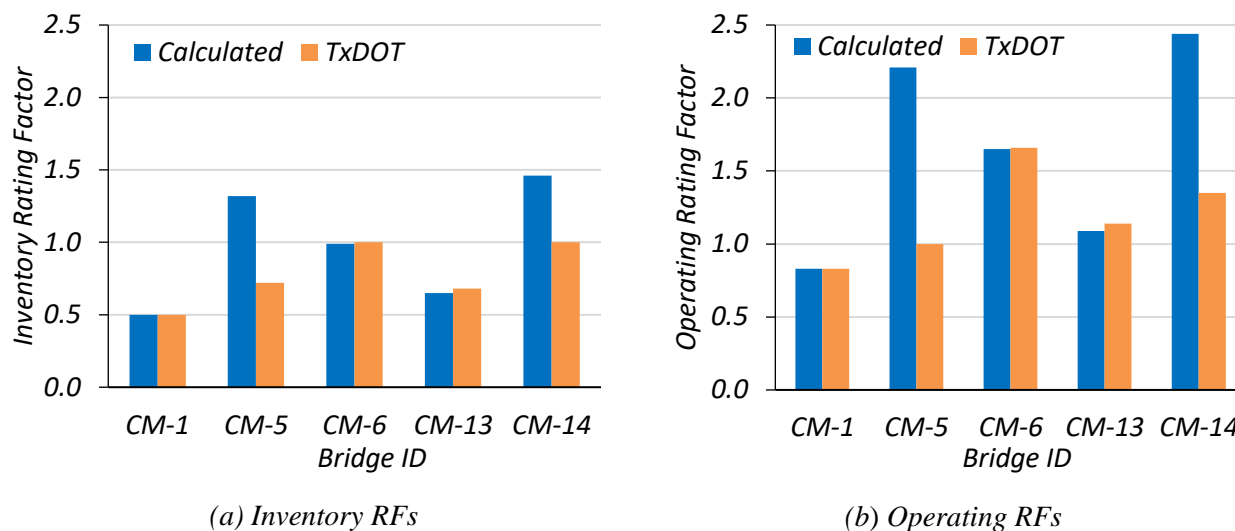


Figure 6.11. Calculated RF vs. TxDOT RF for Flexure of CM Bridges

6.2.5 Conclusion

The selected simple-span concrete multi-girder bridges were analyzed according to the *AASHTO MBE*, *AASHTO Standard Specifications*, and *AASHTO LRFD Specifications*. The LFR method resulted in higher flexure RFs. The difference between the calculated RFs and TxDOT's RFs for each bridge was examined and confirmed. Based on the analysis performed on these bridges, possible areas for improvement to load posting are identified and elaborated upon in Section 6.3.

6.3 AREAS OF POSSIBLE IMPROVEMENT FOR CONCRETE BRIDGES

6.3.1 Refined Analysis

The capacity of the concrete slab bridges was calculated assuming one-way slab action. However, the bridges meet the requirements of two-way analysis. Each bridge has a span-to-width ratio less than or equal to 1.5, and both longitudinal and transverse reinforcement are present. Analyzing concrete slab bridges as two-way slabs can provide more realistic capacity estimation.

Yield line analysis can also be used to evaluate the actual ultimate capacity of a concrete slab bridge. While linear elastic analysis can only predict the first yield at the section, yield line analysis provides more realistic ultimate capacity prediction for slabs. The only concern with yield line analysis is that it may estimate a higher or, at best, equal capacity to the true load-carrying

capacity. Therefore, it requires experience to be able to establish reasonable or valid yield line mechanisms. In addition, knowledge of reinforcement distribution is necessary at the start of the analysis. However, this method can be very useful for analyzing existing structures (Park and Gamble 2000). The full, distributed strength capacity of the concrete slab, and the global collapse of the slab instead of the failure of an effective width component is considered in yield line analysis (Middleton 2007). Pirayeh Gar et al. (2014) demonstrated using experimental tests that the capacity of bridge deck slabs may be satisfactorily estimated by yield line theory.

The load rating methods outlined in the AASHTO manuals are simplified, with assumptions resulting in conservative outcomes. Refined analysis using FEM could help increase the load rating of some of these bridges by accurately capturing the actual behavior of the bridge superstructure. However, developing finite element models takes a considerable amount of time, and special care is needed while determining the model parameters. Analyzing every load-posted bridge using FEM would be a time-consuming process. However, if a number of typical bridges were analyzed using finite element models, a practice could possibly be developed for similar bridges based on representative models. FEM analysis can eliminate inherent conservatism in approximate methods and provide higher RFs by simulating the actual bridge behavior in a more accurate manner.

Refined analysis was not explored in the basic load rating task, but will be examined in detail in the next task of the project.

6.3.2 Number of Lanes

For narrow two-lane bridges, reducing the number of design lanes can be an area of possible improvement. This approach has already been utilized by TxDOT in a number of cases for other bridge types and could be considered for concrete FS slab bridges. Calculating live load demands for one-lane loaded cases would result in a smaller live load moment and therefore higher RF values. The IB346 calculations utilize the number of lanes in determining the number of wheels over the design width. This process affects the live load moment corresponding to the design truck. In the LRFR calculations, the number of lanes limits the upper bound for the effective width of concrete slab bridges under multi-lane loaded conditions.

The effect of reducing the number of design lanes is shown in Table 6.14 for Bridges CS-14 and CM-6. A single live load lane is assumed for both bridges. The RF for CS-14 was improved

by 84 percent over the originally calculated factor, while the RF for CM-6 improved by 8 percent at the inventory level.

Table 6.14. Improvements to Ratings Based on Reduced Design Lanes

Bridge ID	Rating Method	Two-lanes loaded HS20 RF		One-lane loaded HS20 RF		One-lane loaded RF / Two-lanes loaded RF	
		Inventory	Operating	Inventory	Operating	Inventory	Operating
CS-14	LFR	0.57	1.10	1.05	3.80	1.84	3.45
CM-6	LFR	0.99	1.65	1.07	1.79	1.08	1.08

6.3.3 Accurate Material Information

The yield strength of steel is one of the key parameters in the calculation of RFs. This information can be obtained from mill test results available for steel based on the year of construction. For example, during the inspection conducted in 2017, Bridge CS-14 was noted to have mill test data available. Accordingly, the yield strength of reinforcement was taken to be 40 ksi. This increased the rating of the bridge significantly, thereby removing the need for load posting.

Another method for determining the yield strength of reinforcements in existing bridges is to conduct tensile tests on specimens obtained from the bridges. A trend could be studied by testing sufficient samples from bridges built during a certain period and, if applicable, applied to bridges constructed during that period. This procedure may help increase the rating of a specific bridge type with certain characteristics.

The effect of increased material strengths was studied on three sample bridges: CS-14, CS-16, and CM-6. The yield strength for CS-14 and CS-16 bridges was assumed to be 40 ksi, an approximate increase of 20 percent from 33 ksi. The yield strength for CM-6 was assumed to be 50 ksi, an approximate increase of 25 percent from 40 ksi. Table 6.15 shows the results in the form of a ratio of revised RF to initially calculated RF. This process improved the RF for CS-14 by 28 percent over the originally calculated RF and improved the RF for CS-16 by 29 percent of the originally calculated RF at the inventory level. The RF for CM-6 improved by 32 percent at the inventory level.

Table 6.15. Improvements to Ratings Based on Improved Yield Strength

Bridge ID	Rating Method	TxDOT HS20 RF		Calculated HS20 RF		Revised Calculated HS20 RF		Revised Calculated RF/ Calculated RF	
		Inven.	Oper.	Inven.	Oper.	Inven.	Oper.	Inven.	Oper.

CS-14	LFR	0.57	0.96	0.57	1.10	0.73	1.40	1.28	1.27
CS-16	LFR	0.57	0.96	0.56	1.10	0.72	1.40	1.29	1.27
CM-6	LFR	1.00	1.66	0.99	1.65	1.31	2.19	1.32	1.33

6.3.4 Live Load Distribution Factors

The *AASHTO Standard Specifications* and the *AASHTO LRFD Specifications* define the effective width for concrete slab bridges with empirical equations. The effective width of the slab affects the LLDF and therefore the RF.

In 2012, researchers at the University of Delaware conducted load tests for a selection of concrete slab bridges and determined their actual effective width, with the goal of developing new effective width formulas to be used in Delaware (Jones and Shenton 2012). In that study, a diagnostic load test was conducted on six slab bridges in Delaware, and equations for the effective slab width were developed. Table 6.16 compares the developed equations with the current AASHTO LRFD effective width equations.

The inverse of the effective width in feet is equal to the LLDF for a 1 ft longitudinal strip of a slab bridge (IDOT 2012; VDOT 2011; WisDOT 2017). Increasing the equivalent width, like the Delaware study suggested, will decrease the LLDF for the loaded strip of a slab bridge, which will decrease the live load effect on that strip, thereby increasing the RF for that strip and the controlling RF for the bridge. DelDOT has included these new effective width formulas in its rating software and as a result has removed or improved posting restrictions on over 50 bridges in Delaware (Arndt et al. 2017; Jones and Shenton 2012).

Table 6.16. Comparison of Effective Width Equations for Slab Bridges (Jones and Shenton 2012)

	Effective width (in.) per lane for one lane loaded	Effective width (in.) per lane for multiple lanes loaded
AASHTO LRFD	$E = 10.0 + 5.0\sqrt{L_1 W_1}$	$E = 84.0 + 1.44\sqrt{L_1 W_1} \leq \frac{12.0W_1}{N_L}$
Delaware study	$E = 10.0 + 5.8\sqrt{L_1 W_1}$	$E = 84.0 + 2.06\sqrt{L_1 W_1} \leq \frac{12.0W_1}{N_L}$

Note:

- E = Equivalent or effective width (in.)
- L_1 = Modified span length taken equal to the lesser of the actual span length or 60.0 ft
- W_1 = Modified edge-to-edge width of the bridge taken to be equal to the lesser of the actual width, or 60.0 ft for multilane loading, or 30.0 ft for single-lane loading
- W = Physical edge-to-edge width of the bridge (ft)
- N_L = Number of design lanes

6.3.5 Partial Fixity at Supports

Although designed as simply supported systems, unintended partial fixity may be achieved at the supports. This occurrence may be attributed to the bearing detail at the supports and/or friction between the bottom surface of the bridge and the bearing surface. The presence of partial fixity may be verified through field testing. Chajes et al. (1997) conducted a nondestructive experimental load test on a three-span, load-posted, slab-on-steel girder bridge. The 1940 bridge was designed as a non-composite structure with three simply supported spans. The test results indicated substantial restraint at the bearing supports.

The presence of partial fixity will induce negative moments at the supports, and these regions need to be checked when determining the controlling RFs. The effect of partial support fixity will be examined in the next task—the refined analysis—of the project.

7 SELECTED BRIDGES FOR FURTHER INVESTIGATION

Chapter 1 presents a summary of the TxDOT load-posted bridge inventory and provides selected bridges for four major bridge types: steel multi-girder, continuous steel multi-girder, concrete slab, and concrete multi-girder bridges. Chapters 5 and 6 present more detailed information and load rating analysis for these selected typical subsets of bridges that represent the overall SSLO bridge populations in Texas. All selected bridges were load rated based on guidance in the AASHTO MBE (AASHTO 2018). Conducting basic load ratings for representative bridges is necessary to identify the controlling limit states that likely lead to a reduced operating load for these bridges. These limit states can provide insight for determining potential refinements to load ratings for similar bridges, which could lead to increasing the posted load for a bridge or even removing the load posting.

Based on the detailed review and synthesis of the population of load-posted bridges in Texas and the basic load ratings of representative bridges, several areas of opportunity were identified for refined load rating. The controlling limit states identified through the basic load rating process provide insight into the most critical areas of concern by bridge type and allow strategies to be identified that show promise for achieving the greatest impacts. Details and possible impact of the areas of opportunities are provided in Chapters 3 and 4. Four typical load-posted bridge types, one bridge from each of the previously identified bridge types, were selected as representative real-world case studies to further investigate these areas of opportunity. Table 7.1 through Table 7.3 lists some of the key parameters for each selected bridge and for the average bridge of the corresponding bridge type. In addition, some consideration was given to proximity to College Station to help facilitate field testing by the research team.

7.1 SELECTED SIMPLE-SPAN STEEL MULTI-GIRDER BRIDGE

Table 7.1 lists several key parameters for the selected simple-span steel multi-girder bridge and the corresponding average values for steel multi-girder SSLO bridges in Texas. Bridge SM-5 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 41 ft 4 in., which is one simply supported span. With a roadway width of 23 ft 6 in., the effective span for load rating is 40 ft 2 in. and 24 ft 0 in. wide. The bridge carries

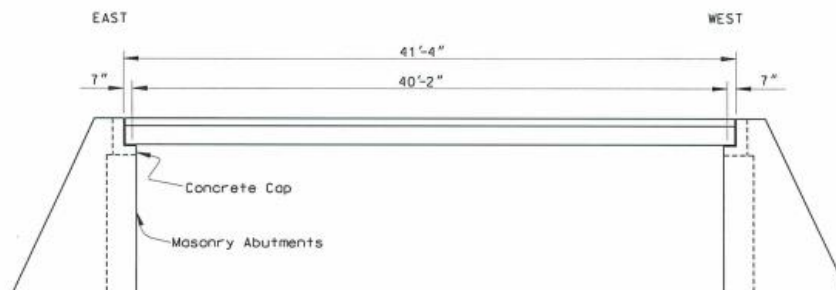
two lanes, one in each direction, and has an ADT of 300 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 17 US tons and an operating gross loading of 28 US tons. The bridge is posted for a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 47,000-lbs single vehicle, and a 74,000-lbs combination vehicle. Table 7.2 summarizes characteristics and basic load ratings, Figure 7.1 and Figure 7.2 shows the structural drawings and photos for the selected Bridge SM-5.

Table 7.1. Selected SSLO Steel Multi-Girder Bridges

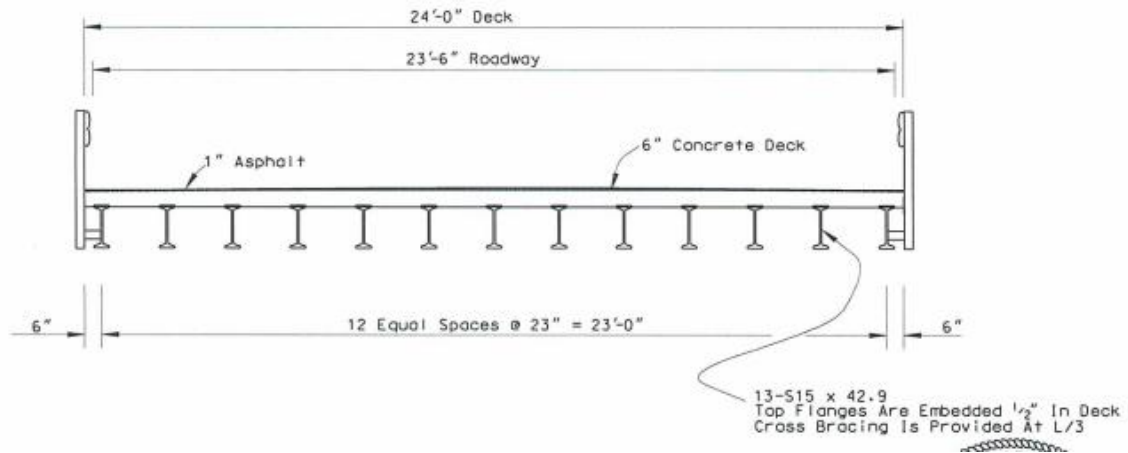
ID	Dist. to CS (mi)	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
							Deck	Super-structure	Sub-structure		
Avg.	–	–	1974	–	36	20	6	6	6	0.83	3
SM-5	61	3	1938	300	41	24	7	6	7	0.77	2



Figure 7.1. Bridge SM-5 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



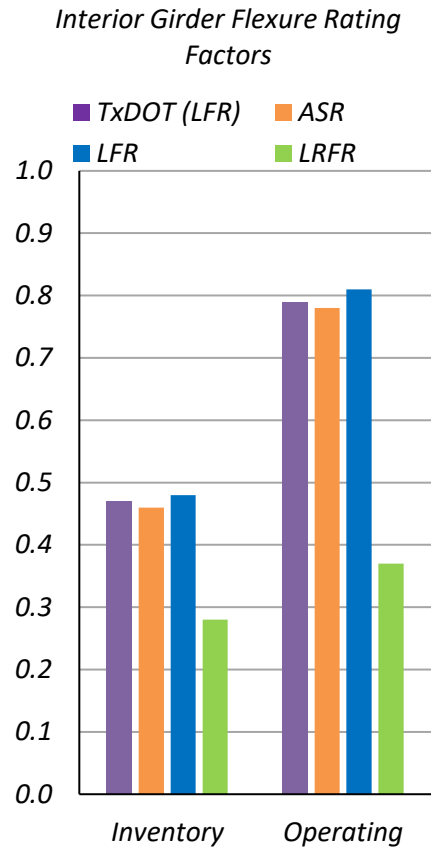
(c) Underside Superstructure View

Figure 7.2. Structural Drawings and a Photo of Bridge SM-5 (TxDOT 2018c)

Table 7.2. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-5

Bridge Characteristics	
Year Built	1938
On/Off-System	On
ADT	300
Controlling Span Length	40'-2"
Deck Width	24'-0"
Roadway Width	23'-6"
Interior Girder Section	S15x42.9

Exterior Girder Section	S15x42.9
Interior Girder Spacing	1'-11"
Slab Thickness (in.)	6.0
Wearing Surface Thickness (in.)	1.0
Composite	no
TxDOT Inventory RF	0.47
TxDOT Operating RF	0.79
TxDOT Inventory Rating (HS)	9.4
TxDOT Operating Rating (HS)	15.8
Posted Axle Limit (lbs)	see note 2



Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	130.7	C_{serv} (kip-ft)	130.7
C_{OP} (ksi)	24.75	C_n (kip-ft)	163.4	C_n (kip-ft)	190.3
f_{DL} (ksi)	8.8	M_{DL} (kip-ft)	43.5	M_{DL} (kip-ft)	43.5
$f_{LL-HS20}$ (ksi)	20.6	$M_{LL-HS20}$ (kip-ft)	101.9	$M_{LL-HL93}$ (kip-ft)	207.4
Inventory HS20 RF	0.46	Inventory HS20 RF	0.48	Inventory HL93 RF	0.28
Operating HS20 RF	0.78	Operating HS20 RF	0.81	Operating HL93 RF	0.37
Inventory Rating (HS)	9.2	Inventory Rating (HS)	9.6		
Operating Rating (HS)	15.6	Operating Rating (HS)	16.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE
2. Posting restrictions include a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 47,000-lbs single vehicle, and a 74,000-lbs combination vehicle.

7.2 SELECTED CONTINUOUS STEEL MULTI-GIRDER BRIDGE

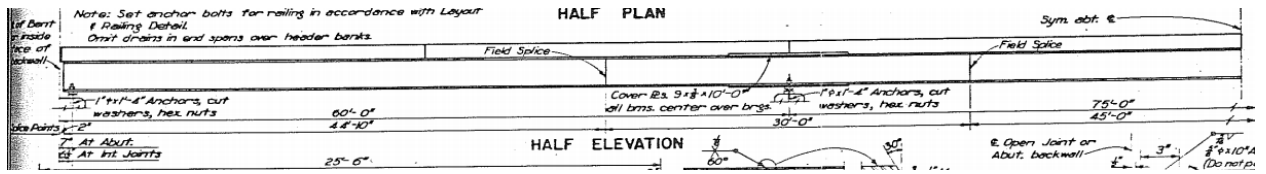
Table 7.3 lists several key parameters for the selected continuous steel multi-girder bridge and the corresponding average values for SSLO continuous steel multi-girder bridges in Texas. Bridge SC-12 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 7 (Good) without beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 195 ft, consisting of three continuous spans. The controlling span for load rating is 75 ft and is 25 ft 6 in. wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 260 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 19 US tons and an operating gross loading of 32 US tons. The bridge is posted for a 20,000-lbs single axle, 34,000-lbs tandem axle, 58,000-lbs single vehicle, or 75,000-lbs combination vehicle. Table 7.4 summarizes characteristics and basic load ratings, Figure 7.3 and Figure 7.4 shows the structural drawings and photos for the selected Bridge SC-12.

Table 7.3. Selected SSLO Continuous Steel Multi-Girder Bridges

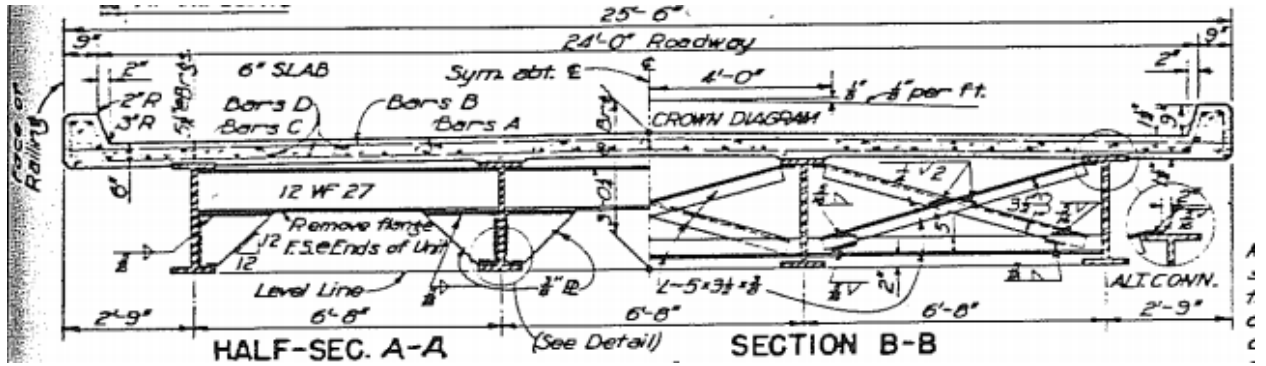
ID	Dist. to CS (mi)	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
							Deck	Super-structure	Sub-structure		
Avg.	–	–	1962	–	25	20	6	6	6	0.85	3
SC-12	155	3	1959	260	75	26	6	7	7	0.88	4



Figure 7.3. Bridge SC-12 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure 7.4. Structural Drawings and a Photo of Bridge SC-12 (TxDOT 2018c)

Table 7.4. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-12

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1959				
On/Off-System	On				
ADT	260				
Span Lengths	60' – 75' – 60'				
Deck Width	25'-6"				
Roadway Width	24'-0"				
Interior Girder Section	30WFx108				
Exterior Girder Section	30WFx108				
Interior Girder Spacing	6'-8"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	2.0				
Composite	no				
TxDOT Inventory RF	0.55				
TxDOT Operating RF	0.93				
TxDOT Inventory Rating (HS)	11.0				
TxDOT Operating Rating (HS)	18.5				
Posted Axle Limit (lbs)	see note 3				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	658.2	C_{serv} (kip-ft)	658.2
C_{OP} (ksi)	24.75	C_n (kip-ft)	822.8	C_n (kip-ft)	938.5
f_{DL} (ksi)	3.8	M_{DL} (kip-ft)	95.4	M_{DL} (kip-ft)	95.4
$f_{LL-HS20}$ (ksi)	20.2	$M_{LL-HS20}$ (kip-ft)	503.1	$M_{LL-HL93}$ (kip-ft)	625.8
Inventory HS20 RF	0.71	Inventory HS20 RF	0.64	Inventory HL93 RF	0.66
Operating HS20 RF	1.04	Operating HS20 RF	1.07	Operating HL93 RF	0.86
Inventory Rating (HS)	14.2	Inventory Rating (HS)	12.8		
Operating Rating (HS)	20.8	Operating Rating (HS)	21.4		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

1. Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.
3. Load restrictions include a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 58,000-lbs single vehicle, and a 75,000-lbs combination vehicle.

7.3 SELECTED SIMPLE-SPAN CONCRETE SLAB BRIDGE

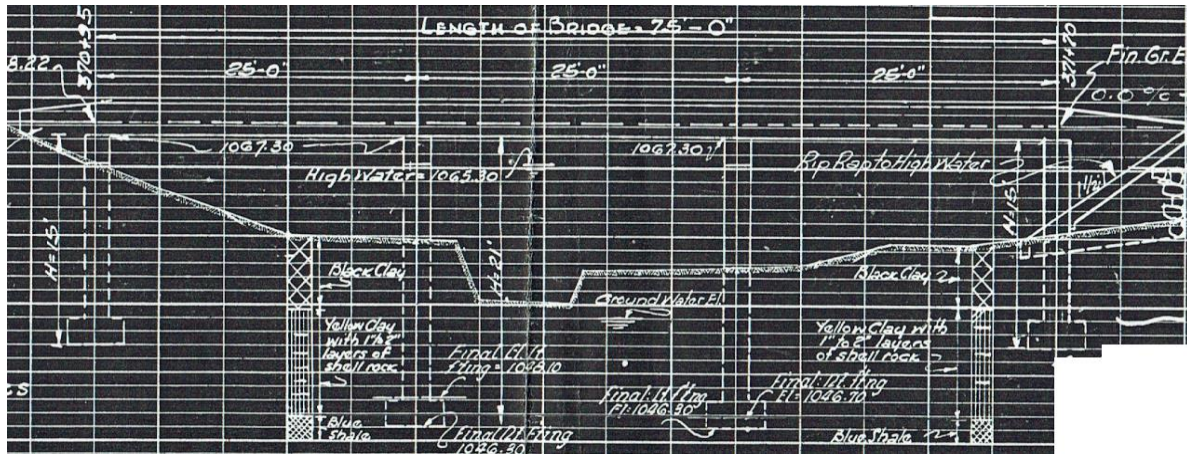
Table 7.5 lists several key parameters for the selected concrete slab bridge and the corresponding average values for SSLO concrete slab bridges in Texas. Bridge CS-9 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 75 ft and includes three simply supported spans. The controlling span for load rating is 25 ft and is 21 ft 4 in. wide, with a roadway width of 20 ft. The bridge carries two lanes, one in each direction, and has an ADT of 30 vehicles. The concrete slab controls the rating of the bridge, which has an inventory gross loading of 16 US tons and an operating gross loading of 33.7 US tons. The bridge is posted for a 28,000-lbs tandem axle. Table 7.6 summarizes characteristics and basic load ratings, Figure 7.5 and Figure 7.6 shows the structural drawings and photos for the selected Bridge CS-9.

Table 7.5. Selected SSLO Concrete Slab Bridges

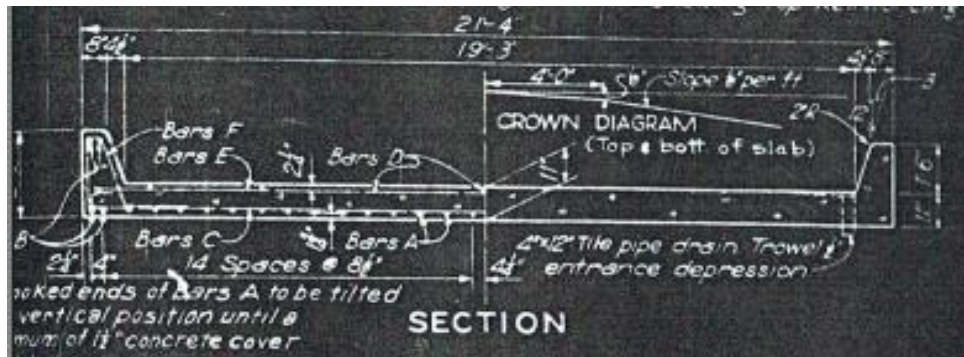
ID	Dist. to CS (mi)	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
							Deck	Super-structure	Sub-structure		
Avg.	–	–	1949	795	22	28	6	6	6	0.98	4
CS-9	157	3	1948	30	25	21	6	6	7	0.94	2



Figure 7.5. Bridge CS-9 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure 7.6. Structural Drawings and a Photo of Bridge CS-9 (TxDOT 2018c)

Table 7.6. Characteristics and Load Ratings of Bridge CS-9

Bridge Characteristics	
Year Built	1948
On/Off-System	On
ADT	30
Controlling Span Length	25'-0"
Deck Width	21'-4"
Roadway Width	20'-0"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	1 in. diam. @ 8.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 – 1.25 in. sq. bars T&B
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	0.45
TxDOT Operating RF	0.94
TxDOT Inventory Rating (HS)	8.9
TxDOT Operating Rating (HS)	18.7
Posted Axle Limit (lbs)	28,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.45	0.15	0.45	0.15
Operating	0.94	0.67	0.94	0.18

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.5	C_n (kip-ft/ft)	23.5	C_n (kip-ft/ft)	23.5
C_{OP} (kip-ft/ft)	18.7				
M_{DL} (kip-ft/ft)	14.6	M_{DL} (kip-ft/ft)	14.6	M_{DL} (kip-ft/ft)	14.6
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	50.6
Inventory HS20 RF	0.15	Inventory HS20 RF	0.45	Inventory HL93 RF	0.14
Operating HS20 RF	0.67	Operating HS20 RF	0.95	Operating HL93 RF	0.18
Inventory Rating (HS)	3.0	Inventory Rating (HS)	9		
Operating Rating (HS)	13.4	Operating Rating (HS)	19		

Notes:

1. $f'_c = 2.5$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

7.4 SELECTED SIMPLE-SPAN CONCRETE MULTI-GIRDER BRIDGE

Table 7.7 lists several key parameters for the selected concrete multi-girder bridge and the corresponding average values for SSLO concrete multi-girder bridges in Texas. Bridge CM-5 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a

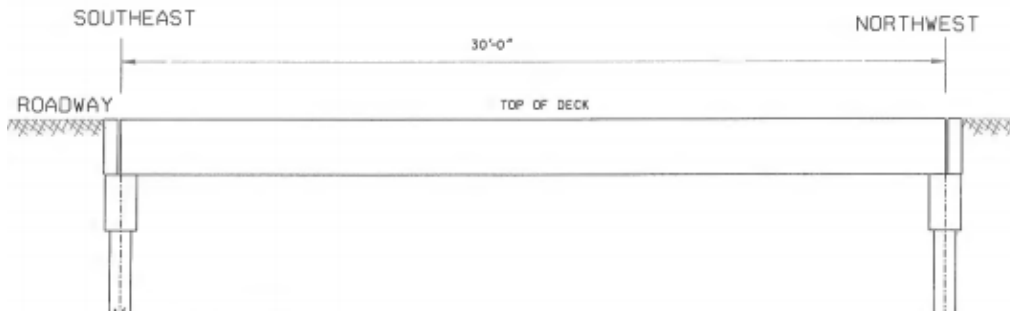
substructure condition rating of 5 (Fair). The bridge has a total length of 30 ft and is one simply supported span. The controlling span for load rating is 29 ft and is 21 ft 8 in. wide, with a roadway width of 21 ft. The bridge carries two lanes, one in each direction, and has an ADT of 150 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 26 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle. Table 7.8 summarizes characteristics and basic load ratings, Figure 7.7 and Figure 7.8 shows the structural drawings and photos for the selected Bridge CM-5.

Table 7.7. Selected SSLO Concrete Multi-Girder Bridges

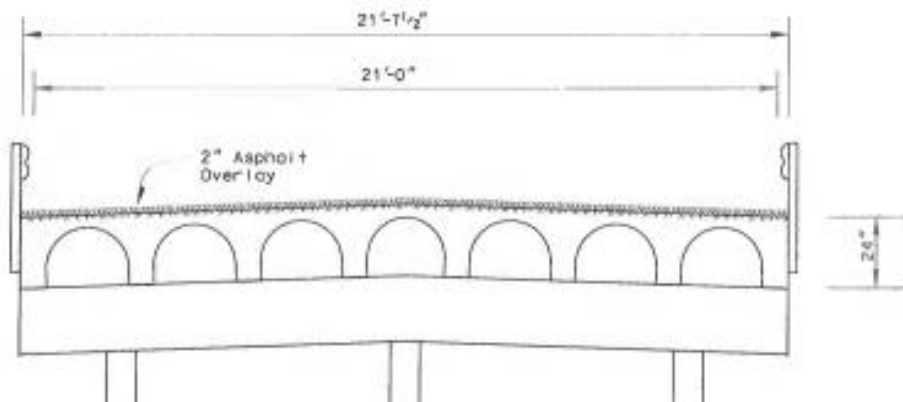
ID	Dist. to CS (mi)	Route Prefix	Year Built	ADT	Max. Span Length (ft)	Deck Width (ft)	Condition Rating			Operating HS20 Rating Factor	Posting Eval.
							Deck	Super-structure	Sub-structure		
Avg.	–	–	1964	–	34	28	7	7	6	0.99	5
CM-5	25	4	1950	150	29	22	7	7	5	0.99	5



Figure 7.7. Bridge CM-5 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure 7.8. Structural Drawings and a Photo of Bridge CM-5 (TxDOT 2018c)

Table 7.8. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-6

Bridge Characteristics		CM-5 Flexure Rating Factors			
Year Built	1950				
On/Off-System	Off				
ADT	150				
Max Span Length	29'-0"				
Deck Width	21'-8"				
Roadway Width	21'-0"				
Slab + Beam Depth (in.)	24.0				
Girder Spacing (in.)	36				
Slab Reinforcement	Unknown				
Beam Tension Reinforcement	Unknown				
Wearing Surface Thickness (in.)	2.0				
TxDOT Inventory RF	0.75				
TxDOT Operating RF	1.00				
TxDOT Inventory Rating (HS)	14.3				
TxDOT Operating Rating (HS)	20.0				
Posted Axle Limit (lbs)	24,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft)	–	C_n (kip-ft)	–	C_n (kip-ft)	–
C_{OP} (kip-ft)	–				
M_{DL} (kip-ft)	–	M_{DL} (kip-ft)	–	M_{DL} (kip-ft)	–
$M_{LL-HS20}$ (kip-ft)	–	$M_{LL-HS20}$ (kip-ft)	–	$M_{LL-HL93}$ (kip-ft)	–
Inventory HS20 RF	–	Inventory HS20 RF	–	Inventory HL93 RF	–
Operating HS20 RF	–	Operating HS20 RF	–	Operating HL93 RF	–
Inventory Rating (HS)	–	Inventory Rating (HS)	–		
Operating Rating (HS)	–	Operating Rating (HS)	–		

Notes:

1. $f'_c = 2.5$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

8 SUMMARY AND CONCLUSIONS

This research project quantifies and characterizes the population of load-posted bridges in Texas and identifies areas of opportunity, including more accurate material properties and information from bridge inspections, refined modeling, and proof testing for verification of acceptable load levels. The first phase of the project includes a detailed review and synthesis of the relevant literature, analysis of the load-posted bridge inventory in Texas, and conducting basic load rating analysis of selected typical bridge types to identify controlling limit states and areas of opportunities that could lead to an increase or removal of load posting. This Volume 1 Research Report fully documents the state-of-the-practice and state-of-the-art on load rating of existing bridges, the review and synthesis of the bridge characteristics of load-posted bridges in Texas, and the basic load rating for the selected representative bridges to identify the controlling limit states that likely lead to a reduced operating load for these bridges.

8.1 LITERATURE REVIEW

A detailed literature review has been compiled to fully document the state-of-the-practice and state-of-the-art on load rating of existing bridges. This comprehensive review provides relevant information regarding the benefits of load rating based on refined constitutive material information, field information, and analysis methods as well as for conducting basic load rating analysis following current state-of-the-practice. Several potential areas of refinement to the load rating process have been identified and are summarized below:

1. Multiple research projects have found, through field measurements and FEM analysis of existing bridges, that the AASHTO LLDFs can be conservative when compared to the measured LLDFs and LLDFs based on refined models of the bridge. If the live load moment effect on the controlling girders of a bridge can be reduced, the RF of that bridge increases as long as the girder is the controlling rating member.
2. If slab-on-steel girder bridges originally designed as non-composite structures or assumed to be non-composite due to unknown details or lack of shear studs could be shown to exhibit some degree of composite action and this behavior could be quantified, their load rating could likely be increased.

3. For concrete slab bridges, accounting for the two-way action of the slab could be used to improve the RFs of slab bridges when deemed necessary.
4. As shown by multiple reports and studies, many historic steel members exhibit higher yield and ultimate strength values than the nominal values used in design. Material testing of in-situ properties could also lead to new insights about the actual strength of steel and concrete members used in construction. Where these increases are permissible, it could provide a more accurate and potentially higher rating factor.
5. Current analysis methods given in AASHTO's manuals are simplified procedures and thus incorporate simplifying assumptions that may reduce the accuracy of the results, usually erring on the side of conservatism. Using refined analysis methods such as finite element models can be more representative of the actual in-situ condition of the bridge and can potentially lead to higher load ratings.

8.2 LOAD-POSTED BRIDGE INVENTORY

The population of load-posted bridges in Texas was reviewed to determine the characteristics that are most typical among these bridges. In particular, details for sub-standard for load only (SSLO) bridges are provided because these bridges are the primary candidates to benefit from increases in load rating because they are more likely to remain in service due to relatively good condition ratings. Key characteristics were reviewed and synthesized to provide an overall understanding of load-posted bridge parameters. These parameters include kind of material and/or design (concrete, steel, wood, etc.), type of design and/or construction (multi-girder, slab, tee beam, box girder, etc.), year built, maximum span length, total bridge length, operating rating, and ratio of operating rating to maximum legal load. As a result of this investigation, SSLO bridges that are load posted based on the AASHTO MBE and having typical parameters were identified from the following four bridge categories: (1) steel multi-girder, (2) concrete slab, (3) concrete multi-girder, and (4) continuous steel multi-girder bridges.

1. An initial review of the Texas load-posted bridge inventory revealed that the majority of the SSLO bridges are steel, concrete, wood, steel continuous, prestressed, and concrete continuous bridges.
2. Steel bridges are the largest bridge category by material kind; 74 percent of load-posted steel bridges and 79 percent of SSLO steel bridges are stringer/multi-girder bridges.

3. The majority of all simple-span load-posted or SSLO concrete bridges are slab, culvert, or stringer/multi-girder bridges. Approximately 45 percent of all load-posted concrete bridges are slab bridges, and 15 percent are multi-girder bridges.
4. While almost 30 percent of load-posted concrete bridges are culverts, this type was not considered further in this project because culvert bridges were the subject of a recently completed TxDOT project (Wood et al. 2010).
5. Four bridge types (steel multi-girder, concrete slab, concrete multi-girder, and steel continuous multi-girder SSLO bridges) were identified for additional consideration for the basic load rating, refined analysis, and field-testing tasks of the current project.

8.3 BASIC LOAD RATING AND AREAS OF OPPORTUNITY

Basic load rating calculations have been carried out for the selected representative set of bridges that correspond to the most populated bridges types, identified based on a review of the load-posted bridge inventory in Texas. Basic load ratings for the representative bridges were developed to identify the controlling limit states that likely lead to a reduced operating load for these bridges. These limit states provided insight for determining potential refinements to load ratings for similar bridges, which could lead to increasing the posted load for a bridge or even removing the load posting. The controlling limit states identified through the basic load rating process point toward the most critical areas of concern by bridge type and allow strategies to be developed that show promise for achieving the greatest impacts. The following areas have been identified as possible areas of opportunity for improving load ratings:

1. Slab-on-steel girder bridges are conservatively analyzed as non-composite when the interface details are unknown. However, as Chajes et al. (1997) showed, even non-composite bridges do exhibit some amount of partial composite behavior. If partial composite action can be quantified, it can be used to more accurately calculate the nominal resistance and increase the load rating.
2. If an engineering decision is made that the bridge is too narrow to carry two trucks, narrow two-lane bridges can be analyzed as a one-lane bridge if deemed appropriate. The reduced load demand can potentially help removing the posting.
3. Load rating calculations generally use the recommendations for the material properties specified in the AASHTO MBE (AASHTO 2018). These recommendations are generally

on the conservative side and underestimate the nominal resistance of the bridge components. Accurate measurement of material properties through standard laboratory testing on collected samples or estimations of material properties using in-situ nondestructive material tests can lead to an increase in the load ratings due to a more accurate, and potentially higher, strength-based capacity or stress limit.

4. Consideration of partial fixity at supports may be appropriate for some bridges. If a simple-span bridge can be load tested and demonstrated to show partial fixity at the supports, this end fixity would lower the maximum positive live load and dead load moments near the middle of the span and therefore increase the rating. This process could be established for bridges with certain characteristics and potentially applied to similar bridges exhibiting those characteristics.
5. Current analysis methods given in the AASHTO manuals and standards are simplified procedures erring on the side of conservatism that may reduce the accuracy of the results. FEM models can be tailored to a particular bridge and reduce the need for simplifying assumptions when they are properly developed to be representative of the actual in-situ condition of the bridge. Such a refined analysis can result in a decreased load effect through refined LLDFs and can be used to evaluate the effect of potential composite action between the girders and deck or partial fixity at the supports.
6. The current LLDFs found in the *AASHTO Standard Specifications* (AASHTO 2002) and *AASHTO LRFD Specifications* (AASHTO 2017) are approximate equations that are accurate for certain bridge types having certain characteristics but less accurate and too conservative for others (Hueste et al. 2015). The live load distribution between girders can be more accurately predicted using a properly developed three-dimensional FEM model.

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APPENDIX A
GEOGRAPHIC LOCATIONS OF SSLO BRIDGES IN TEXAS

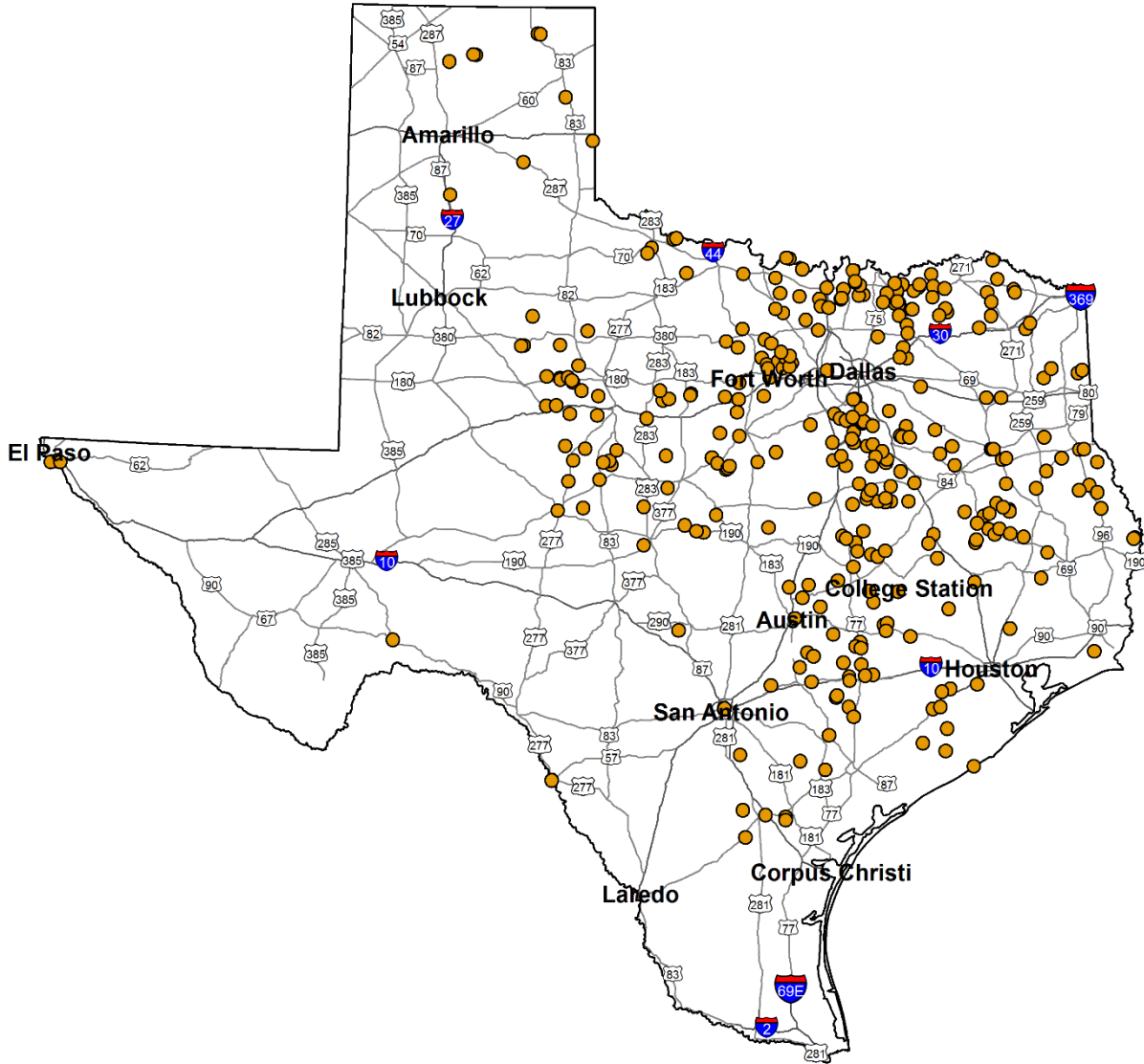


Figure A.1. Location Map of SSLO Steel Bridges

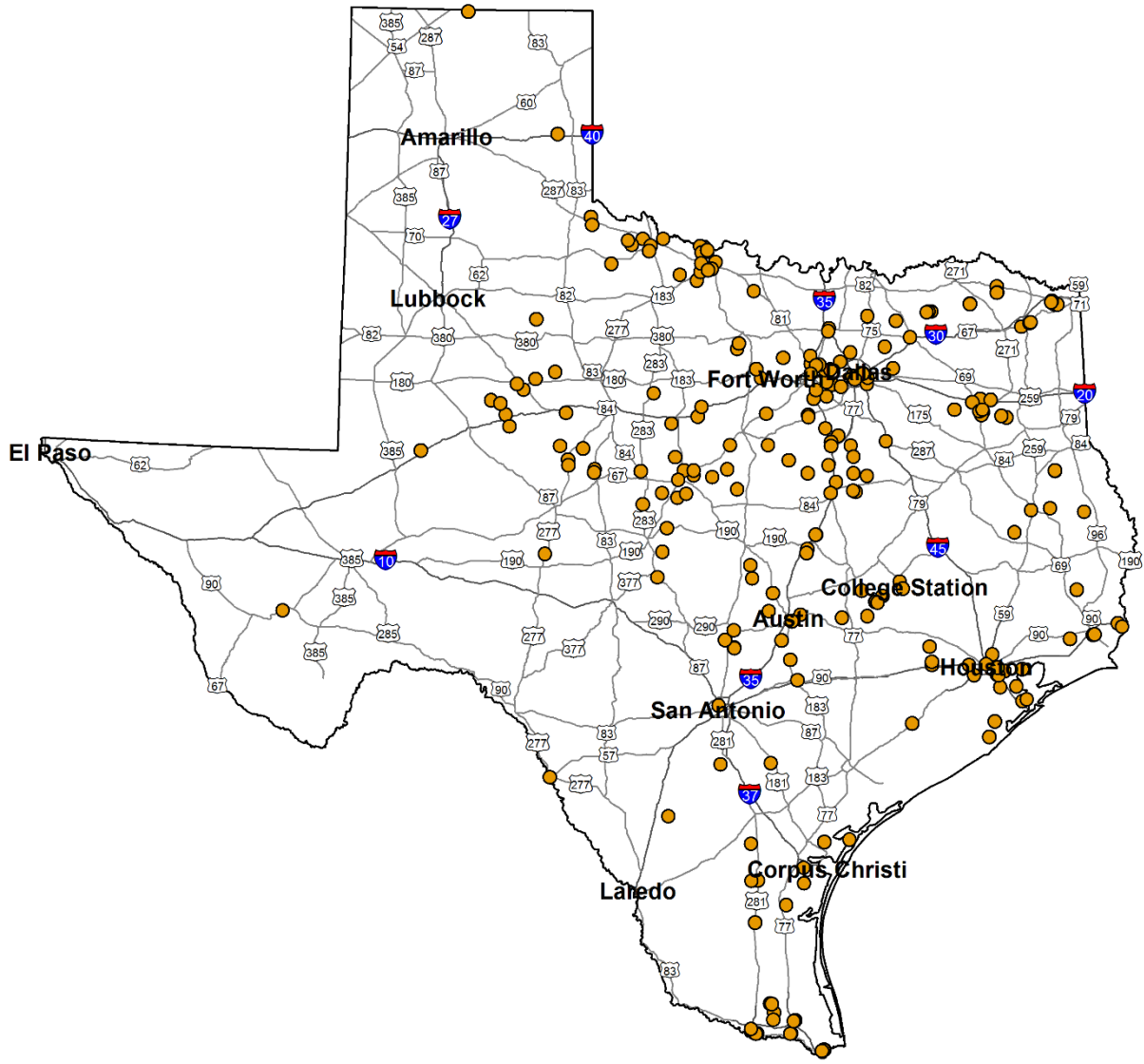


Figure A.2. Location Map of SSLO Concrete Bridges

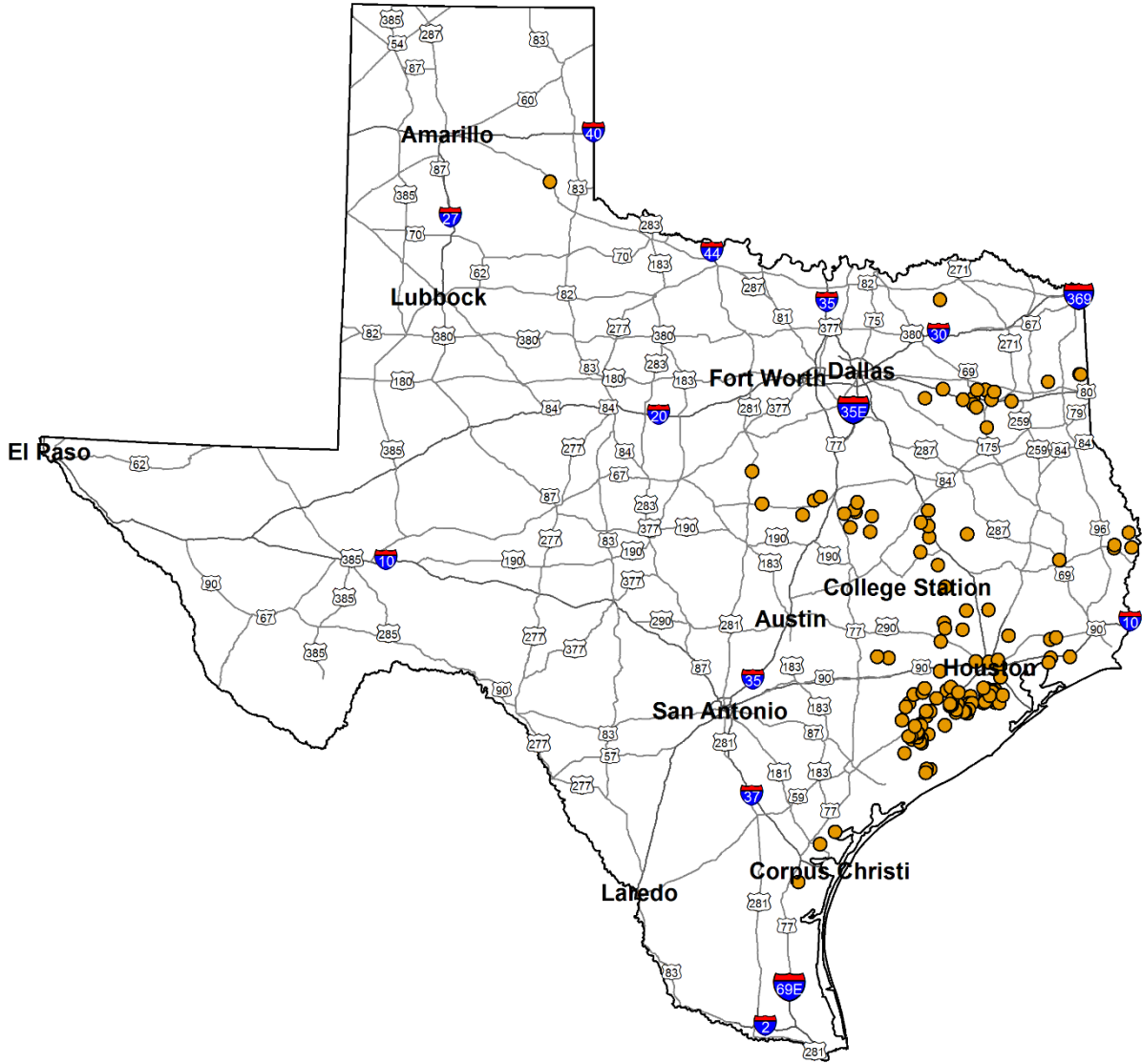


Figure A.3. Location Map of SSLO Wood Bridges

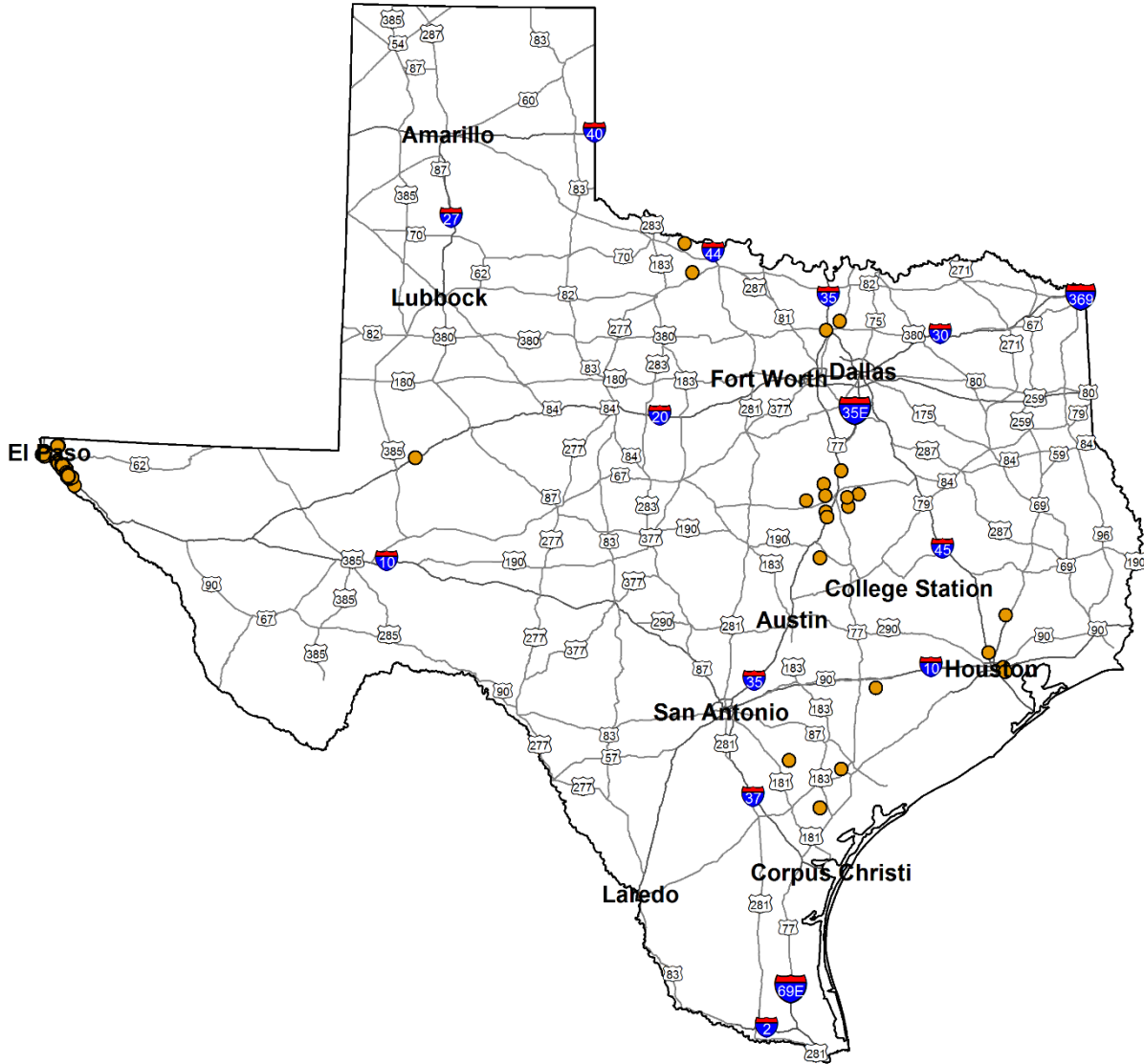


Figure A.5. Location Map of SSLO Prestressed Bridges

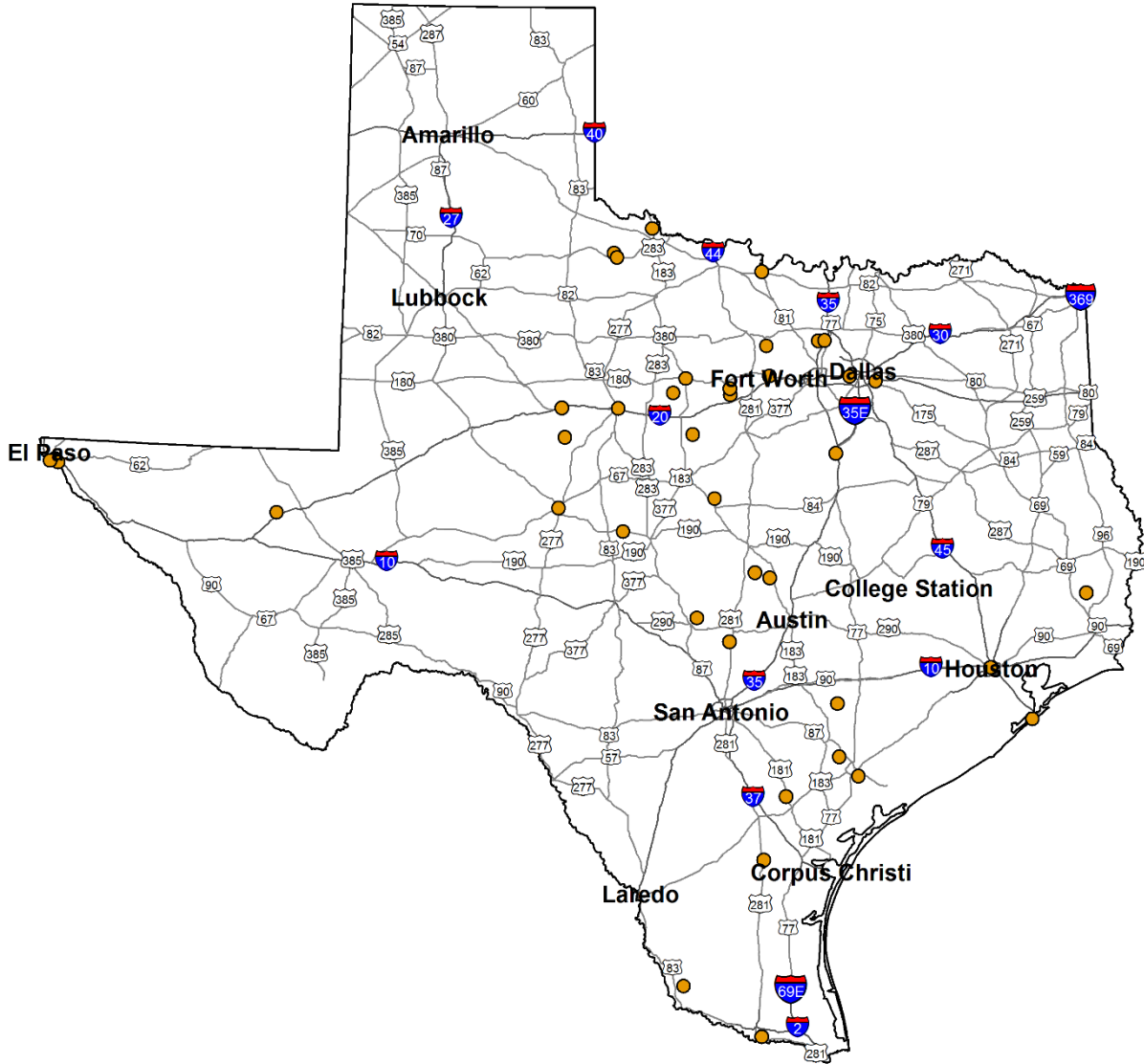


Figure A.6. Location Map of SSLO Concrete Continuous Bridges

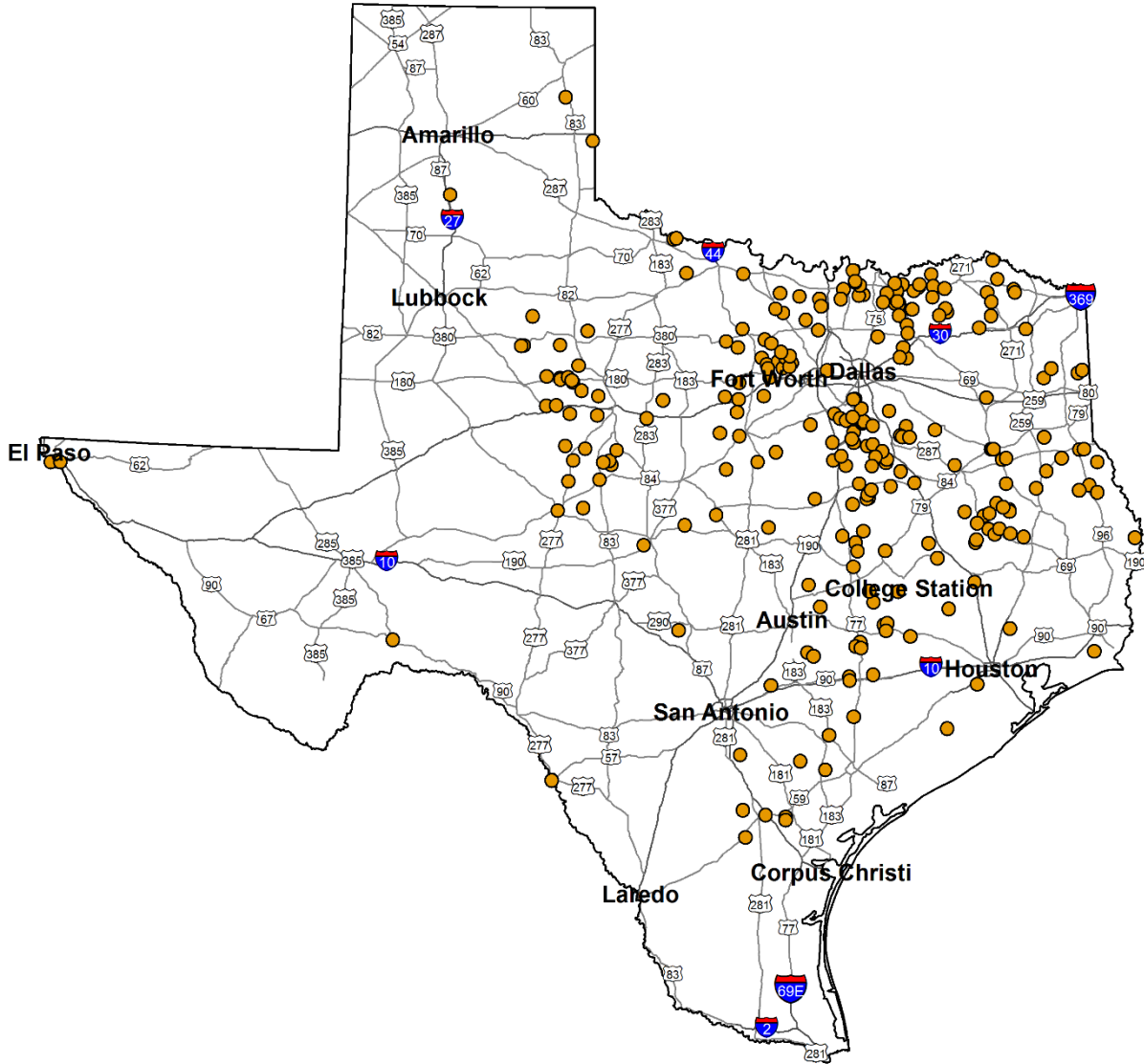


Figure A.7. Location Map of SSLO Steel Multi-Girder Bridges

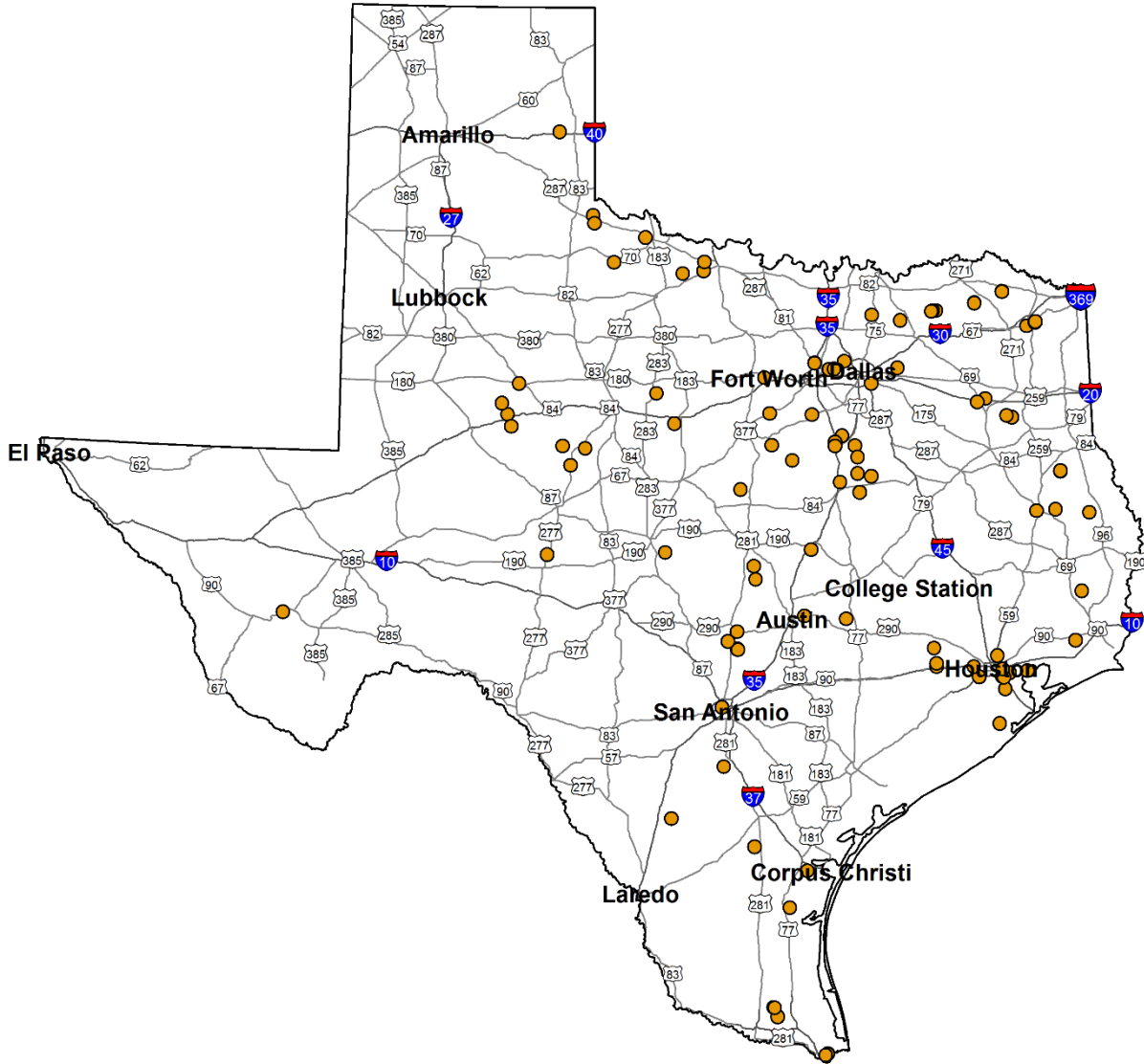


Figure A.8. Location Map of SSLO Concrete Slab Bridges

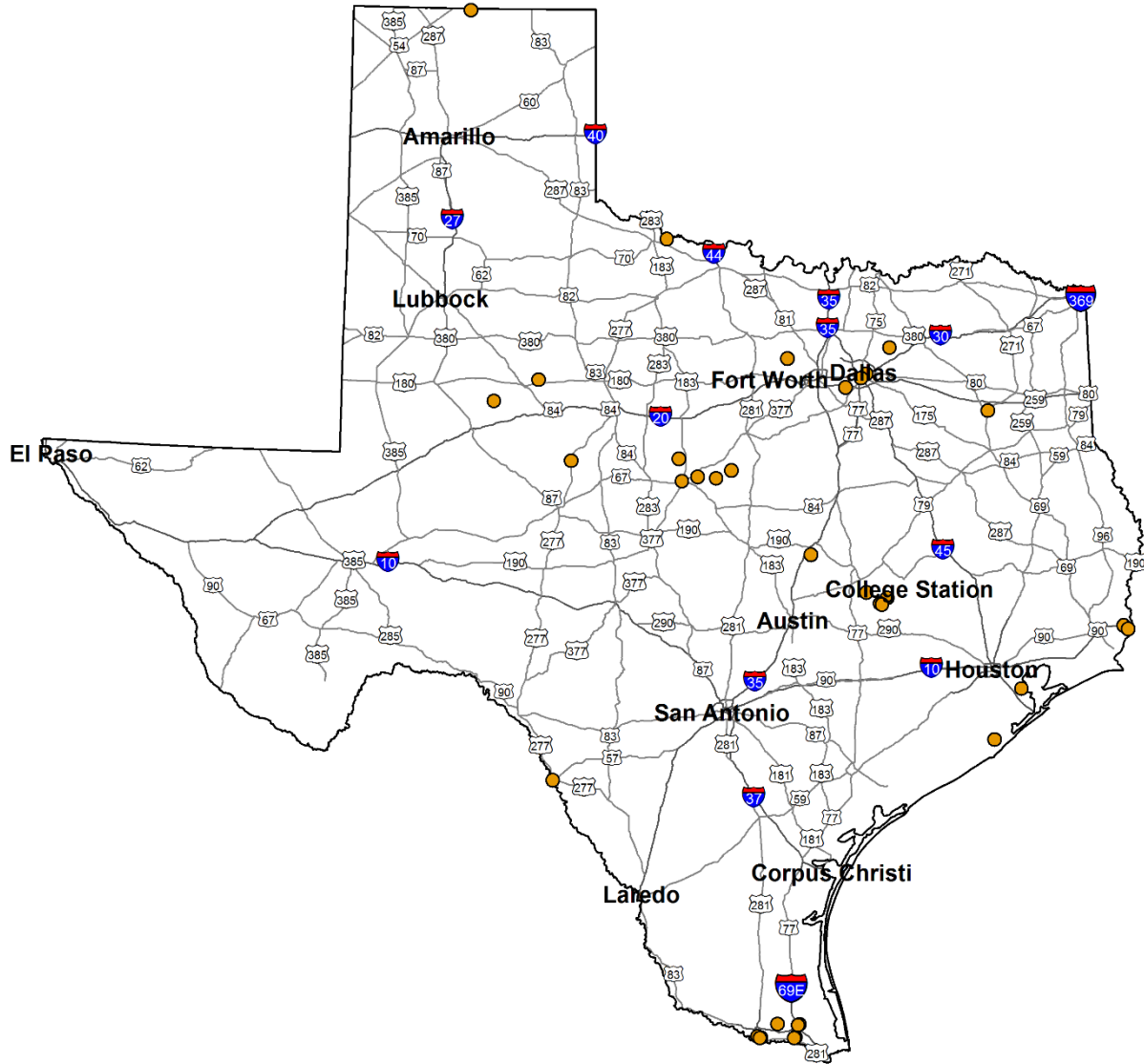


Figure A.9. Location Map of SSLO Concrete Multi-Girder Bridges

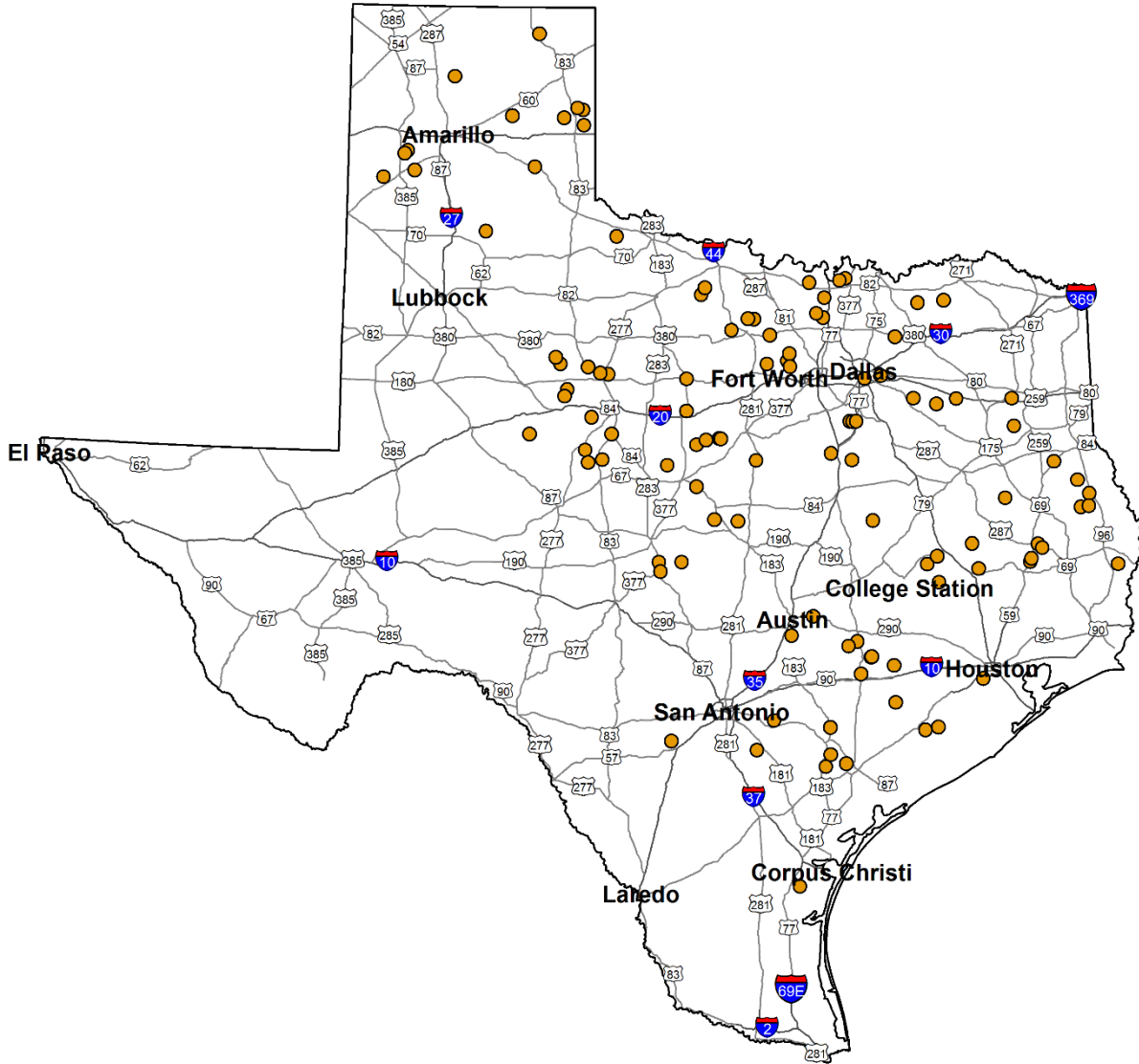


Figure A.10. Location Map of SSLO Steel Continuous Multi-Girder Bridges

APPENDIX B
SUMMARY OF STEEL BRIDGE DETAILS AND LOAD RATING

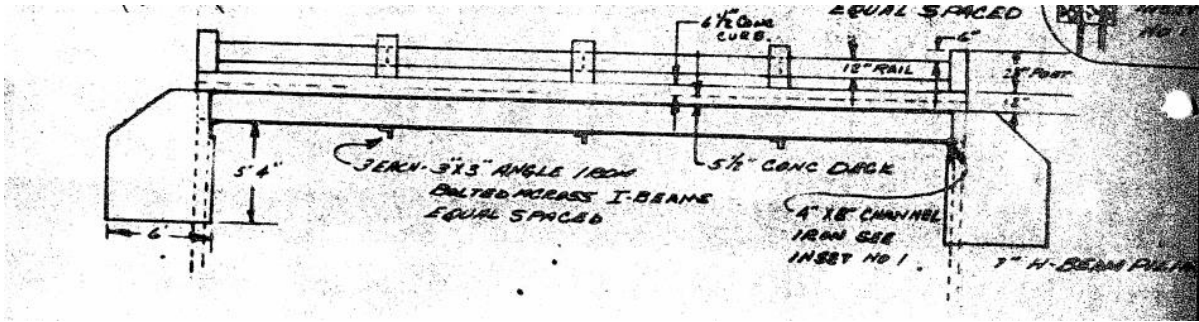
B.1 STEEL MULTI GIRDER BRIDGES

B.1.1 Bridge SM-1

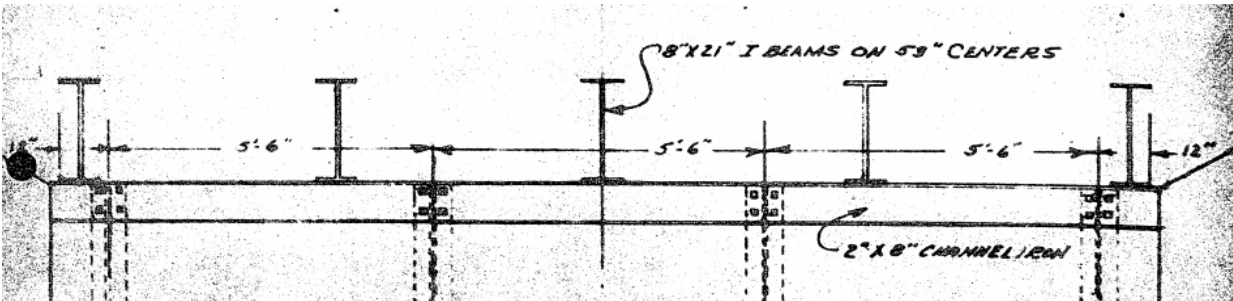
Bridge SM-1 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 40 ft, which is one simply supported span. The controlling span for load rating is 39 ft and is 24 ft 8 in., wide with a roadway width of 24 ft 2 in. The bridge carries two lanes, one in each direction, and has an ADT of 290 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 12 US tons and an operating gross loading of 20 US tons. The bridge is posted for a 15,000-lbs axle or tandem axle.



Figure B.1. Bridge SM-1 (TxDOT 2018c)



(a) Elevation View—does not match information in bridge inventory sheet



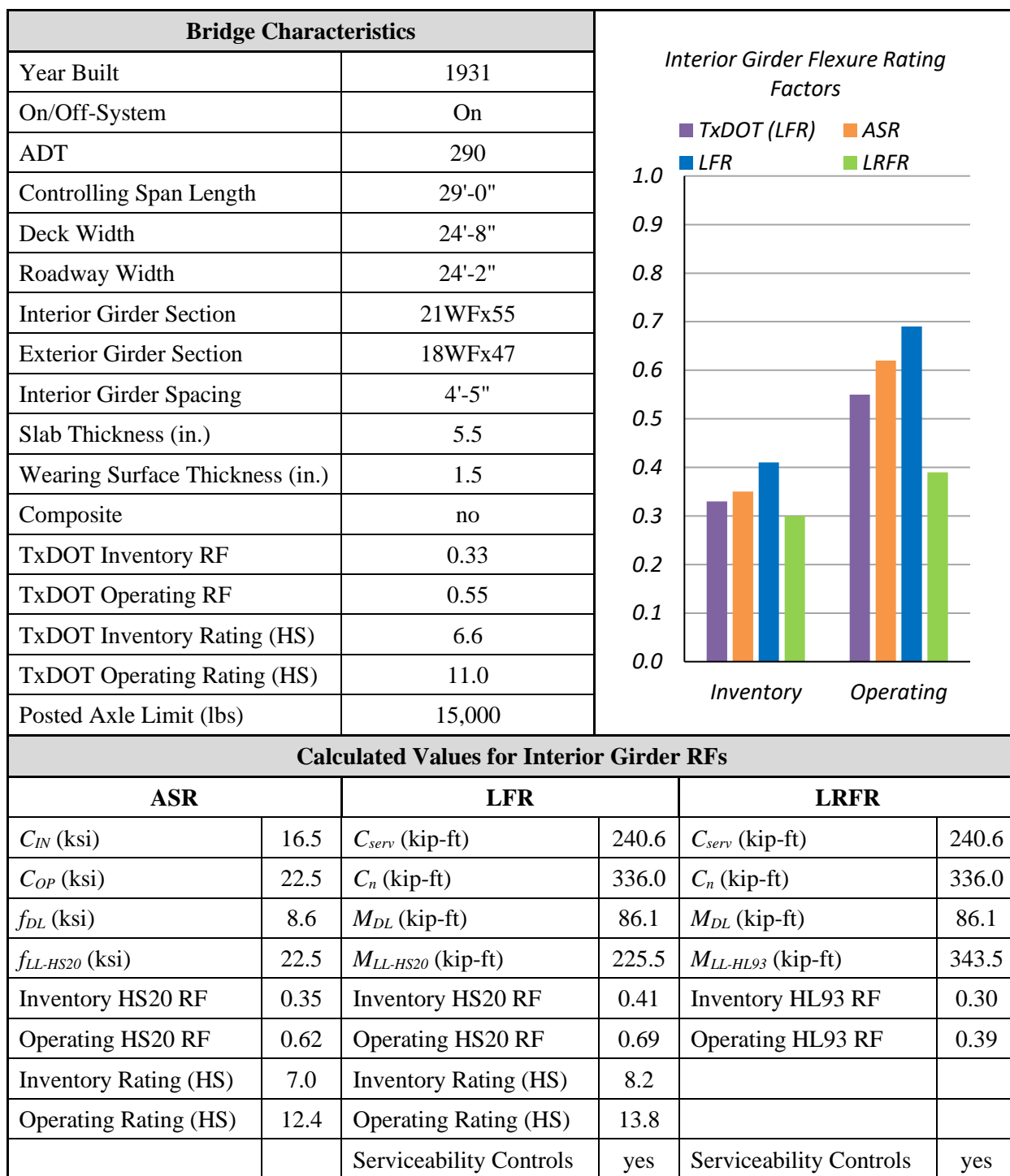
(b) Transverse Section—does not match information in bridge inventory sheet



(c) Underside Superstructure View

Figure B.2. Bridge Drawings and Photos (TxDOT 2018c)

Table B.1. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-1



Note: Steel yield strength taken as 30 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

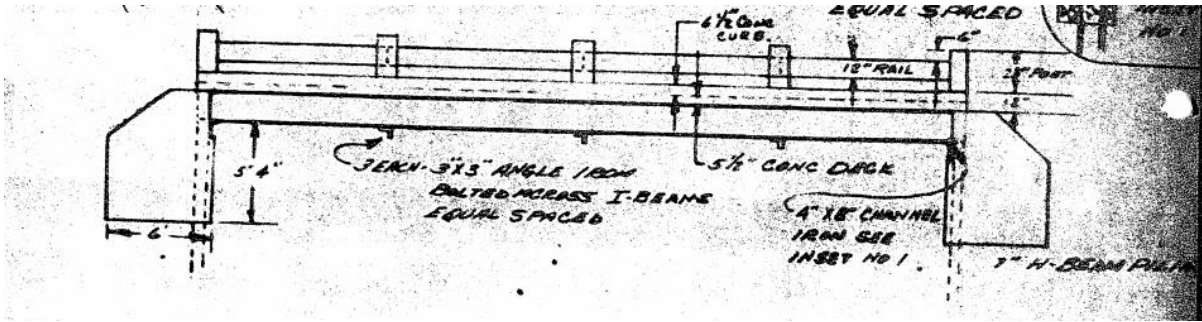
B.1.2 Bridge SM-2

Bridge SM-2 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 6

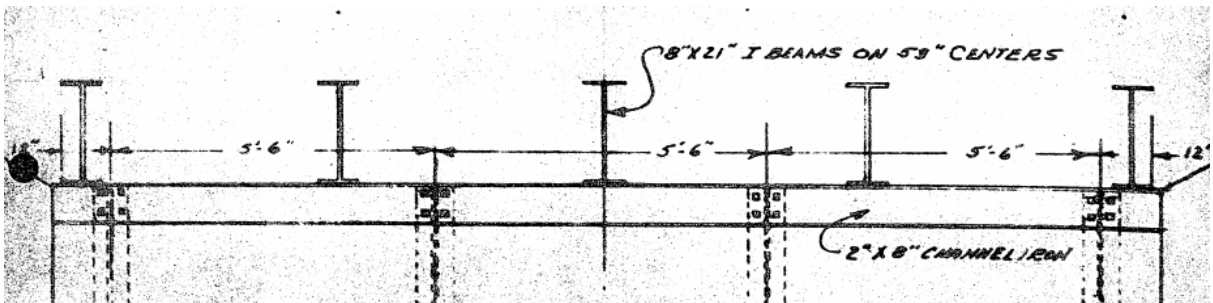
(Satisfactory). The bridge has a total length of 40 ft, which is one simply supported span. The controlling span for load rating is 38 ft 4 in. and is 25 ft 2 in. wide, with a roadway width of 24 ft 8 in. The bridge carries two lanes, one in each direction, and has an ADT of 290 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 12 US tons and an operating gross loading of 20 US tons. The bridge is posted for a 15,000-lbs axle or tandem axle.



Figure B.3. Bridge SM-2 (TxDOT 2018c)



(a) Elevation View—does not match information in bridge inventory sheet



(b) Transverse Section—does not match information in bridge inventory sheet



(c) Underside Superstructure View

Figure B.4. Bridge Drawings and Photos (TxDOT 2018c)

Table B.2. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-2

Bridge Characteristics	
Year Built	1931
On/Off-System	On
ADT	290
Controlling Span Length	28'-4"
Deck Width	25'-2"
Roadway Width	24'-8"
Interior Girder Section	21WFx55
Exterior Girder Section	18WFx47
Interior Girder Spacing	4'-5"
Slab Thickness (in.)	5.5
Wearing Surface Thickness (in.)	1.5
Composite	no
TxDOT Inventory RF	0.34
TxDOT Operating RF	0.56
TxDOT Inventory Rating (HS)	6.7
TxDOT Operating Rating (HS)	11.1
Posted Axle Limit (lbs)	15,000

Condition	TxDOT (LFR)	ASR	LFR	LRFR
Inventory	0.34	0.38	0.44	0.32
Operating	0.56	0.66	0.73	0.41

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	16.5	C_{serv} (kip-ft)	240.6	C_{serv} (kip-ft)	240.6
C_{OP} (ksi)	22.5	C_n (kip-ft)	336.0	C_n (kip-ft)	336.0
f_{DL} (ksi)	8.3	M_{DL} (kip-ft)	83.1	M_{DL} (kip-ft)	83.1
$f_{LL-HS20}$ (ksi)	21.6	$M_{LL-HS20}$ (kip-ft)	216.2	$M_{LL-HL93}$ (kip-ft)	332.7
Inventory HS20 RF	0.38	Inventory HS20 RF	0.44	Inventory HL93 RF	0.32
Operating HS20 RF	0.66	Operating HS20 RF	0.73	Operating HL93 RF	0.41
Inventory Rating (HS)	7.6	Inventory Rating (HS)	8.8		
Operating Rating (HS)	13.2	Operating Rating (HS)	14.6		
		Serviceability Controls	yes	Serviceability Controls	yes

Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.3 Bridge SM-3

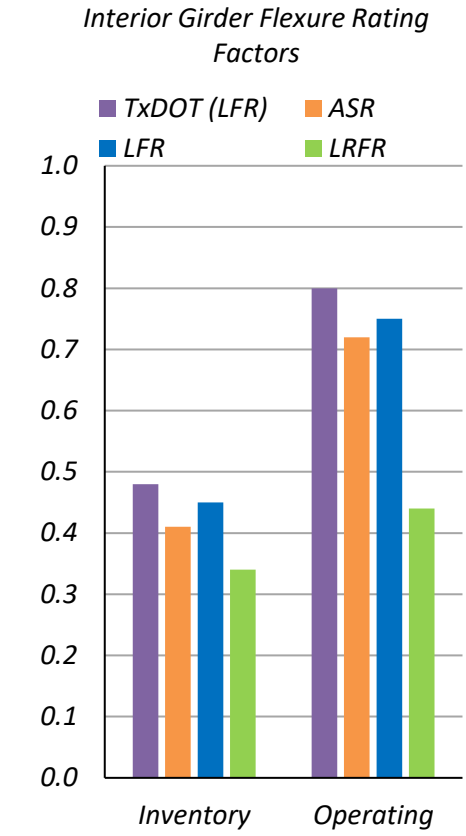
Bridge SM-3 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 270 ft and includes nine simply supported spans. The controlling span for load rating is 29 ft and is 25 ft 5 in. wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 450 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 17 US tons and an operating gross loading of 29 US tons. The bridge is posted for a 17,500-lbs axle or tandem axle.



Figure B.5. Bridge SM-3 (TxDOT 2018c)

Table B.3. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-3

Bridge Characteristics					
Year Built	1938				
On/Off-System	On				
ADT	450				
Controlling Span Length	29'-0"				
Deck Width	25'-5"				
Roadway Width	24'-0"				
Interior Girder Section	21WFx62				
Exterior Girder Section	21WFx62				
Interior Girder Spacing	7'-1"				
Slab Thickness (in.)	8.0				
Wearing Surface Thickness (in.)	1.0				
Composite	no				
TxDOT Inventory RF	0.48				
TxDOT Operating RF	0.80				
TxDOT Inventory Rating (HS)	9.5				
TxDOT Operating Rating (HS)	15.9				
Posted Axle Limit (lbs)	17,500				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	281.6	C_{serv} (kip-ft)	281.6
C_{OP} (ksi)	24.75	C_n (kip-ft)	352.0	C_n (kip-ft)	396.7
f_{DL} (ksi)	9.6	M_{DL} (kip-ft)	102.6	M_{DL} (kip-ft)	102.6
$f_{LL-HS20}$ (ksi)	21.0	$M_{LL-HS20}$ (kip-ft)	223.5	$M_{LL-HL93}$ (kip-ft)	345.1
Inventory HS20 RF	0.41	Inventory HS20 RF	0.45	Inventory HL93 RF	0.34
Operating HS20 RF	0.72	Operating HS20 RF	0.75	Operating HL93 RF	0.44
Inventory Rating (HS)	8.2	Inventory Rating (HS)	9		
Operating Rating (HS)	14.4	Operating Rating (HS)	15		
		Serviceability Controls	no	Serviceability Controls	yes



Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.4 Bridge SM-4

Bridge SM-4 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 90 ft and includes three simply supported spans. The controlling span for load rating is 30 ft and is 25 ft 5 in. wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 450 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 14 US tons and an operating gross loading of 20 US tons. The bridge is posted for a 17,500-lbs axle or tandem axle.



Figure B.7. Bridge SM-4 (TxDOT 2018c)

Table B.4. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-4

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1938				
On/Off-System	On				
ADT	450				
Controlling Span Length	30'-0"				
Deck Width	25'-5"				
Roadway Width	24'-0"				
Interior Girder Section	21WFx63				
Exterior Girder Section	21WFx63				
Interior Girder Spacing	7'-1"				
Slab Thickness (in.)	6.75				
Wearing Surface Thickness (in.)	2.0				
Composite	no				
TxDOT Inventory RF	0.39				
TxDOT Operating RF	0.56				
TxDOT Inventory Rating (HS)	7.7				
TxDOT Operating Rating (HS)	11.1				
Posted Axle Limit (lbs)	17,500				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	281.6	C_{serv} (kip-ft)	281.6
C_{OP} (ksi)	24.75	C_n (kip-ft)	352.0	C_n (kip-ft)	396.7
f_{DL} (ksi)	9.4	M_{DL} (kip-ft)	100.1	M_{DL} (kip-ft)	100.1
$f_{LL-HS20}$ (ksi)	21.0	$M_{LL-HS20}$ (kip-ft)	223.5	$M_{LL-HL93}$ (kip-ft)	345.1
Inventory HS20 RF	0.42	Inventory HS20 RF	0.46	Inventory HL93 RF	0.35
Operating HS20 RF	0.73	Operating HS20 RF	0.76	Operating HL93 RF	0.45
Inventory Rating (HS)	8.4	Inventory Rating (HS)	9.2		
Operating Rating (HS)	14.6	Operating Rating (HS)	15.2		
		Serviceability Controls	no	Serviceability Controls	yes

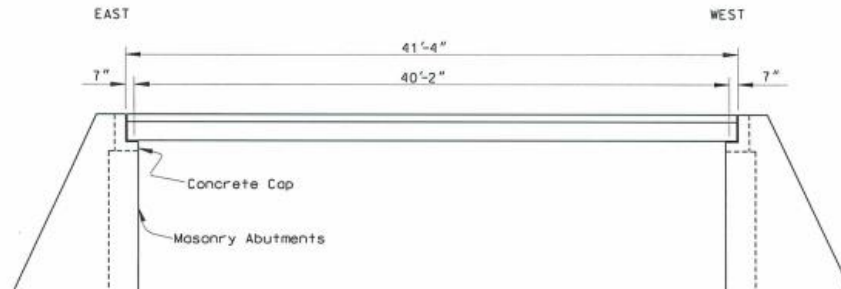
Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.5 Bridge SM-5

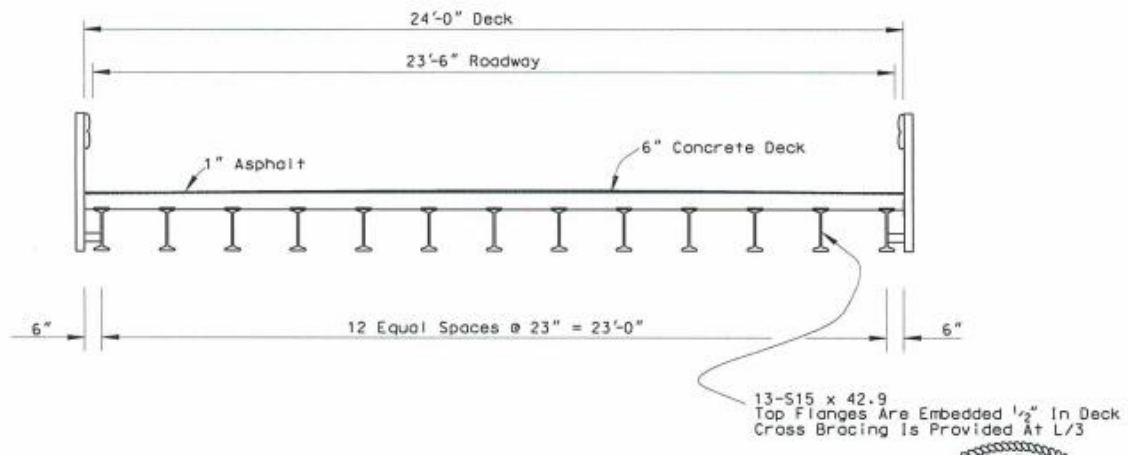
Bridge SM-5 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 41 ft 4 in., which is one simply supported span. The controlling span for load rating is 40 ft 2 in. and is 24 ft 0 in. wide with a roadway width of 23 ft 6 in. The bridge carries two lanes, one in each direction, and has an ADT of 300 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 17 US tons and an operating gross loading of 28 US tons. The bridge is posted for a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 47,000-lbs single vehicle, and a 74,000-lbs combination vehicle.



Figure B.9. Bridge SM-5 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.10. Bridge Drawings and Photos (TxDOT 2018c)

Table B.5. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-5

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1938				
On/Off-System	On				
ADT	300				
Controlling Span Length	40'-2"				
Deck Width	24'-0"				
Roadway Width	23'-6"				
Interior Girder Section	S15x42.9				
Exterior Girder Section	S15x42.9				
Interior Girder Spacing	1'-11"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	1.0				
Composite	no				
TxDOT Inventory RF	0.47				
TxDOT Operating RF	0.79				
TxDOT Inventory Rating (HS)	9.4				
TxDOT Operating Rating (HS)	15.8				
Posted Axle Limit (lbs)	see note 2				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	130.7	C_{serv} (kip-ft)	130.7
C_{OP} (ksi)	24.75	C_n (kip-ft)	163.4	C_n (kip-ft)	190.3
f_{DL} (ksi)	8.8	M_{DL} (kip-ft)	43.5	M_{DL} (kip-ft)	43.5
$f_{LL-HS20}$ (ksi)	20.6	$M_{LL-HS20}$ (kip-ft)	101.9	$M_{LL-HL93}$ (kip-ft)	207.4
Inventory HS20 RF	0.46	Inventory HS20 RF	0.48	Inventory HL93 RF	0.28
Operating HS20 RF	0.78	Operating HS20 RF	0.81	Operating HL93 RF	0.37
Inventory Rating (HS)	9.2	Inventory Rating (HS)	9.6		
Operating Rating (HS)	15.6	Operating Rating (HS)	16.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

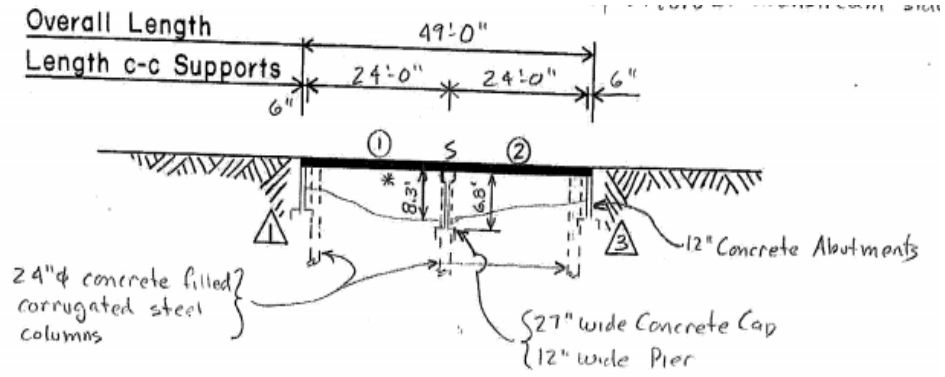
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 47,000-lbs single vehicle, and a 74,000-lbs combination vehicle.

B.1.6 Bridge SM-6

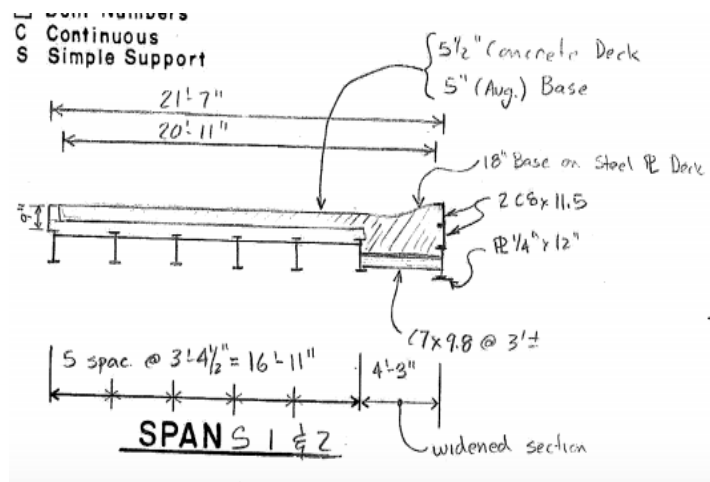
Bridge SM-6 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 49 ft and includes two simply supported spans. The controlling span for load rating is 24 ft 6 in. and is 21 ft 7 in. wide, with a roadway width of 20 ft 11 in. The bridge carries two lanes, one in each direction, and has an ADT of 50 vehicles. The assumed substructure rating controls the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 7,500-lbs axle or tandem axle.



Figure B.11. Bridge SM-6 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.12. Bridge Drawings and Photos (TxDOT 2018c)

Table B.6. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-6

Bridge Characteristics	
Year Built	1940
On/Off-System	Off
ADT	50
Controlling Span Length	24'-6"
Deck Width	21'-7"
Roadway Width	20'-11"
Interior Girder Section	S15x42.9
Exterior Girder Section	S15x42.9
Interior Girder Spacing	3'-5"
Slab Thickness (in.)	5.5
Wearing Surface Thickness (in.)	5.0
Composite	no
TxDOT Inventory RF	0.89
TxDOT Operating RF	1.39
TxDOT Inventory Rating (HS)	17.8
TxDOT Operating Rating (HS)	27.7
Posted Axle Limit (lbs)	7,500

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.89	0.78	0.75	0.43
Operating	1.39	1.21	1.25	0.56

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	130.7	C_{serv} (kip-ft)	130.7
C_{OP} (ksi)	24.75	C_n (kip-ft)	163.4	C_n (kip-ft)	190.3
f_{DL} (ksi)	6.0	M_{DL} (kip-ft)	29.6	M_{DL} (kip-ft)	29.6
$f_{LL-HS20}$ (ksi)	15.5	$M_{LL-HS20}$ (kip-ft)	76.8	$M_{LL-HL93}$ (kip-ft)	167.3
Inventory HS20 RF	0.78	Inventory HS20 RF	0.75	Inventory HL93 RF	0.43
Operating HS20 RF	1.21	Operating HS20 RF	1.25	Operating HL93 RF	0.56
Inventory Rating (HS)	15.6	Inventory Rating (HS)	15		
Operating Rating (HS)	24.2	Operating Rating (HS)	25		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

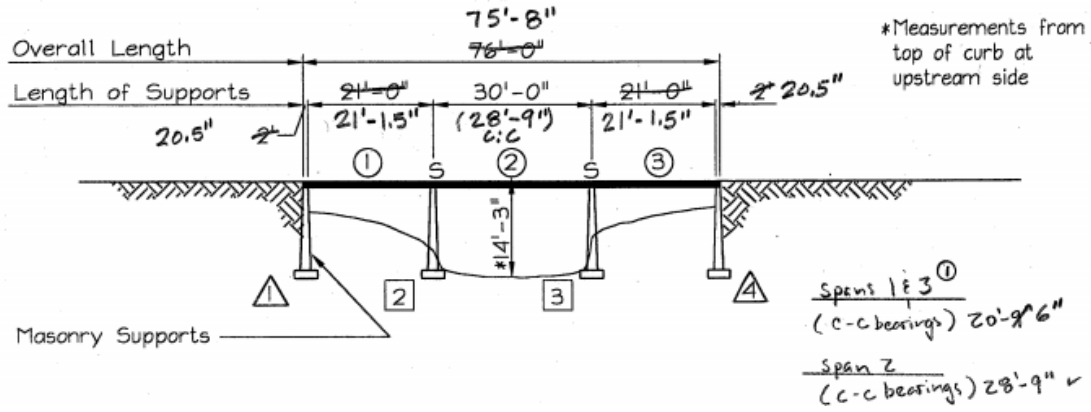
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Rating is controlled by assumed substructure RFs of 1.0 (Operating) and 0.75 (Inventory).
3. Posting is suggested to be removed as of 9/21/17 inspection.

B.1.7 Bridge SM-7

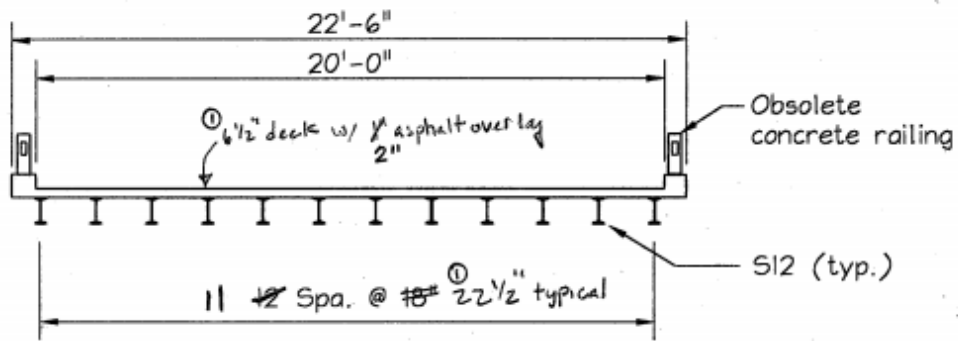
Bridge SM-7 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 4 (Poor) without beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 75 ft 8 in. and includes three simply supported spans. The controlling span for load rating is 28 ft 9 in. and is 22 ft 6 in. wide, with a roadway width of 20 ft. The bridge carries two lanes, one in each direction, and has an ADT of 340 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 21 US tons and an operating gross loading of 34 US tons. The bridge is posted for a 34,000-lbs axle or tandem axle.



Figure B.13. Bridge SM-7 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.14. Bridge Drawings and Photos (TxDOT 2018c)

Table B.7. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-7

Bridge Characteristics	
Year Built	1940
On/Off-System	On
ADT	340
Controlling Span Length	28'-9"
Deck Width	22'-6"
Roadway Width	20'-0"
Interior Girder Section	S12x31.8
Exterior Girder Section	S12x31.8
Interior Girder Spacing	1'-11"
Slab Thickness (in.)	6.5
Wearing Surface Thickness (in.)	2.0
Composite	no
TxDOT Inventory RF	0.57
TxDOT Operating RF	0.95
TxDOT Inventory Rating (HS)	11.4
TxDOT Operating Rating (HS)	19.0
Posted Axle Limit (lbs)	34,000

Condition	ASR	LFR	LRFR
Inventory	0.51	0.53	0.26
Operating	0.85	0.88	0.34

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	79.6	C_{serv} (kip-ft)	79.6
C_{OP} (ksi)	24.75	C_n (kip-ft)	99.6	C_n (kip-ft)	115.0
f_{DL} (ksi)	8.1	M_{DL} (kip-ft)	24.4	M_{DL} (kip-ft)	24.4
$f_{LL-HS20}$ (ksi)	19.6	$M_{LL-HS20}$ (kip-ft)	59.2	$M_{LL-HL93}$ (kip-ft)	143
Inventory HS20 RF	0.51	Inventory HS20 RF	0.53	Inventory HL93 RF	0.26
Operating HS20 RF	0.85	Operating HS20 RF	0.88	Operating HL93 RF	0.34
Inventory Rating (HS)	10.2	Inventory Rating (HS)	10.6		
Operating Rating (HS)	17.0	Operating Rating (HS)	17.6		
		Serviceability Controls	no	Serviceability Controls	yes

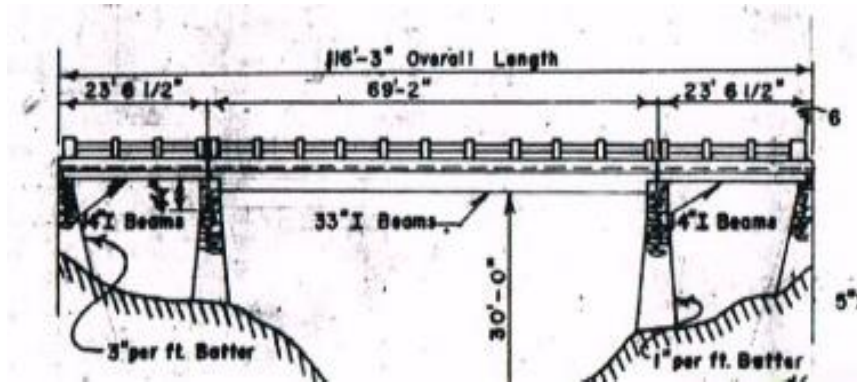
Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.8 Bridge SM-8

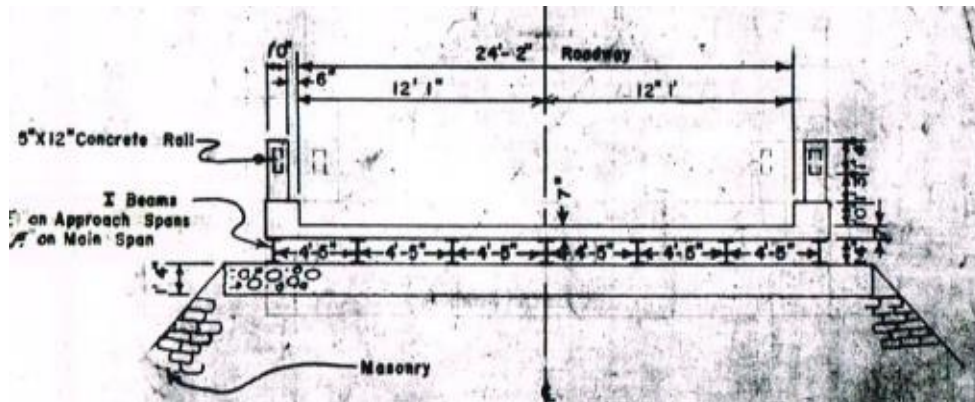
Bridge SM-8 has a deck condition rating of 5 (Fair), a superstructure condition rating of 5 (Fair) without beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 116 ft 3 in. and includes three simply supported spans. The controlling span for load rating is 21 ft 9 in. and is 27 ft 4 in. wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 170 vehicles. The steel girders control the rating of the bridge, which has an inventory gross loading of 15 US tons and an operating gross loading of 25 US tons. The bridge is posted for 28,000-lbs gross loading.



Figure B.15. Bridge SM-8 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.16. Bridge Drawings and Photos (TxDOT 2018c)

Table B.8. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-8

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1940				
On/Off-System	On				
ADT	170				
Controlling Span Length	21'-9"				
Deck Width	27'-4"				
Roadway Width	24'-0"				
Interior Girder Section	CB141X34				
Exterior Girder Section	CB141X34				
Interior Girder Spacing	4'-5"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	4.0				
Composite	no				
TxDOT Inventory RF	0.41				
TxDOT Operating RF	0.69				
TxDOT Inventory Rating (HS)	8.2				
TxDOT Operating Rating (HS)	13.7				
Posted Axle Limit (lbs)	28,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	106.7	C_{serv} (kip-ft)	106.7
C_{OP} (ksi)	24.75	C_n (kip-ft)	133.4	C_n (kip-ft)	147.7
f_{DL} (ksi)	10.5	M_{DL} (kip-ft)	42.5	M_{DL} (kip-ft)	42.5
$f_{LL-HS20}$ (ksi)	22.7	$M_{LL-HS20}$ (kip-ft)	91.9	$M_{LL-HL93}$ (kip-ft)	182.6
Inventory HS20 RF	0.34	Inventory HS20 RF	0.39	Inventory HL93 RF	0.23
Operating HS20 RF	0.63	Operating HS20 RF	0.65	Operating HL93 RF	0.29
Inventory Rating (HS)	6.8	Inventory Rating (HS)	7.8		
Operating Rating (HS)	12.6	Operating Rating (HS)	13		
		Serviceability Controls	no	Serviceability Controls	yes

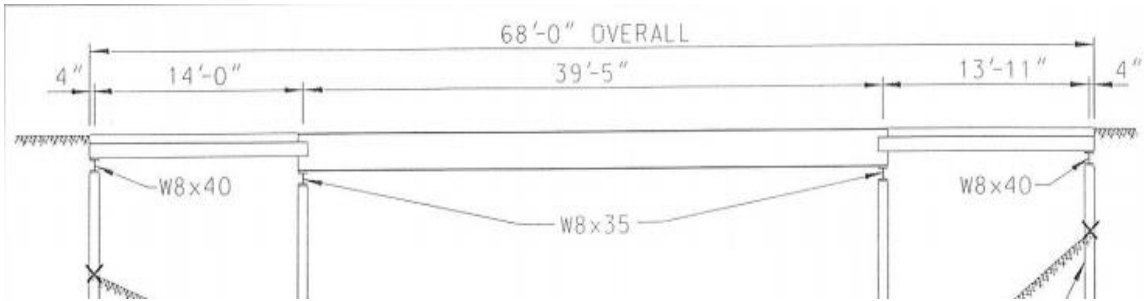
Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.9 Bridge SM-9

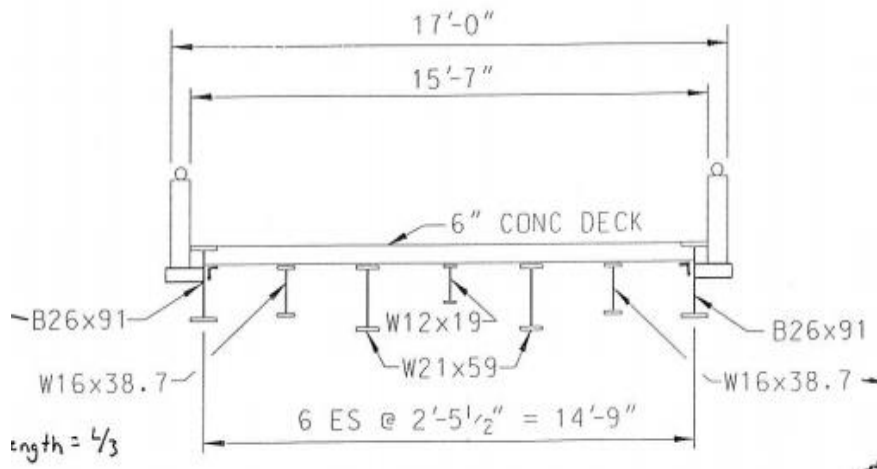
Bridge SM-9 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 68 ft and includes three simply supported spans. The controlling span for load rating is 39 ft 5 in. and is 17 ft wide, with a roadway width of 15 ft 6 in. The bridge carries one lane and has an ADT of 50 vehicles. The assumed deck rating controls the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for 48,000-lbs gross loading and a 24,000-lbs tandem axle.



Figure B.17. Bridge SM-9 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.18. Bridge Drawings and Photos (TxDOT 2018c)

Table B.9. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-9

Bridge Characteristics	
Year Built	1941
On/Off-System	Off
ADT	50
Controlling Span Length	39'-5"
Deck Width	17'-0"
Roadway Width	15'-6"
Interior Girder Section	21WFx59
Exterior Girder Section	B26x91
Interior Girder Spacing	4'-11"
Slab Thickness (in.)	6.0
Wearing Surface Thickness (in.)	0.0
Composite	no
TxDOT Inventory RF	0.82
TxDOT Operating RF	1.27
TxDOT Inventory Rating (HS)	16.3
TxDOT Operating Rating (HS)	25.3
Posted Axle Limit (lbs)	see note 2

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.82	0.88	0.89	0.71
Operating	1.27	1.37	1.49	0.93

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	385.1	C_{serv} (kip-ft)	385.1
C_{OP} (ksi)	24.75	C_n (kip-ft)	543.5	C_n (kip-ft)	543.5
f_{DL} (ksi)	6.2	M_{DL} (kip-ft)	90.86	M_{DL} (kip-ft)	90.86
$f_{LL-HS20}$ (ksi)	13.5	$M_{LL-HS20}$ (kip-ft)	197.2	$M_{LL-HL93}$ (kip-ft)	292.7
Inventory HS20 RF	0.88	Inventory HS20 RF	0.89	Inventory HL93 RF	0.71
Operating HS20 RF	1.37	Operating HS20 RF	1.49	Operating HL93 RF	0.93
Inventory Rating (HS)	17.6	Inventory Rating (HS)	17.8		
Operating Rating (HS)	27.4	Operating Rating (HS)	29.8		
		Serviceability Controls	yes	Serviceability Controls	yes

Notes:

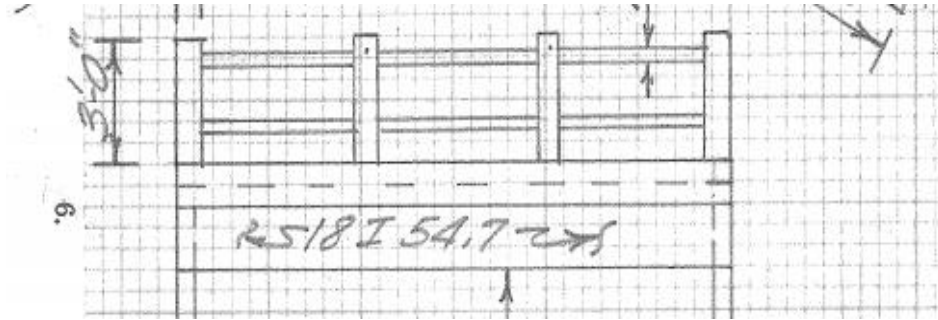
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 24,000-lbs tandem axle and a 48,000-lbs gross load.
3. Rating is controlled by assumed deck RFs of 1.0 (Operating) and 0.75 (Inventory).

B.1.10 Bridge SM-10

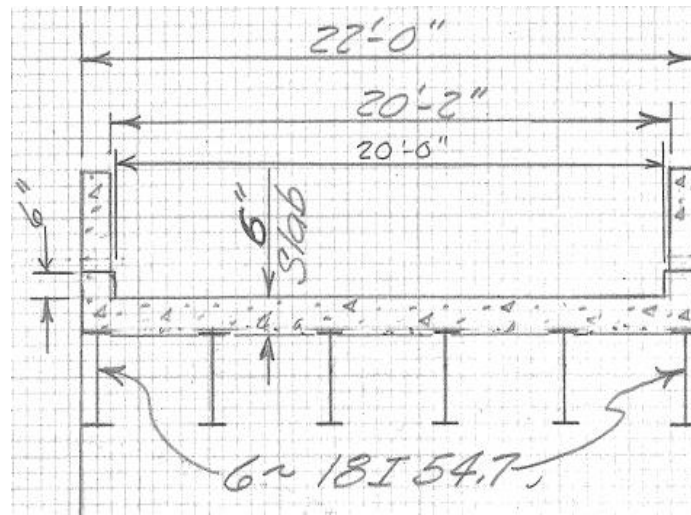
Bridge SM-10 has a deck condition rating of 5 (Fair), a superstructure condition rating of 5 (Fair) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 30 ft, which is one simply supported span. The controlling span for load rating is 30 ft and is 22 ft wide, with a roadway width of 20 ft. The bridge carries one lane and has an ADT of 100 vehicles. The assumed deck rating controls the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 43 US tons. The bridge is posted for a 15,000-lbs axle or tandem axle.



Figure B.19. Bridge SM-10 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.20. Bridge Drawings and Photos (TxDOT 2018c)

Table B.10. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-10

Bridge Characteristics	
Year Built	1945
On/Off-System	Off
ADT	100
Controlling Span Length	30'-0"
Deck Width	22'-0"
Roadway Width	20'-0"
Interior Girder Section	S18x54.7
Exterior Girder Section	S18x54.7
Interior Girder Spacing	4'-4"
Slab Thickness (in.)	6.0
Wearing Surface Thickness (in.)	1.0
Composite	no
TxDOT Inventory RF	0.77
TxDOT Operating RF	1.20
TxDOT Inventory Rating (HS)	15.4
TxDOT Operating Rating (HS)	24.0
Posted Axle Limit (lbs)	15,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.77	0.74	0.73	0.48
Operating	1.20	1.17	1.22	0.62

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	186	C_{serv} (kip-ft)	186
C_{OP} (ksi)	24.75	C_n (kip-ft)	232.5	C_n (kip-ft)	271.7
f_{DL} (ksi)	6.9	M_{DL} (kip-ft)	48.4	M_{DL} (kip-ft)	48.4
$f_{LL-HS20}$ (ksi)	15.2	$M_{LL-HS20}$ (kip-ft)	107.4	$M_{LL-HL93}$ (kip-ft)	201.5
Inventory HS20 RF	0.74	Inventory HS20 RF	0.73	Inventory HL93 RF	0.48
Operating HS20 RF	1.17	Operating HS20 RF	1.22	Operating HL93 RF	0.62
Inventory Rating (HS)	14.8	Inventory Rating (HS)	14.6		
Operating Rating (HS)	23.4	Operating Rating (HS)	24.4		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

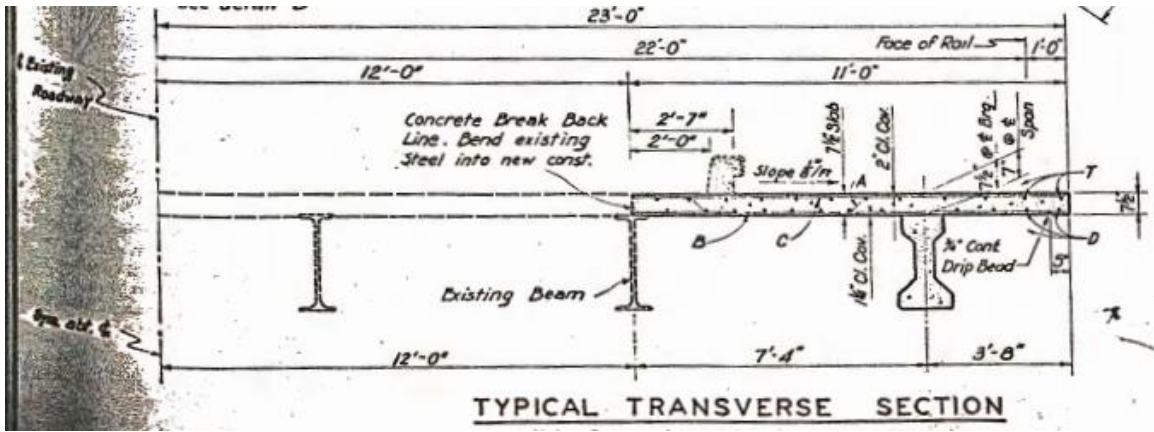
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Rating is controlled by assumed deck RFs of 1.0 (Operating) and 0.75 (Inventory).

B.1.11 Bridge SM-11

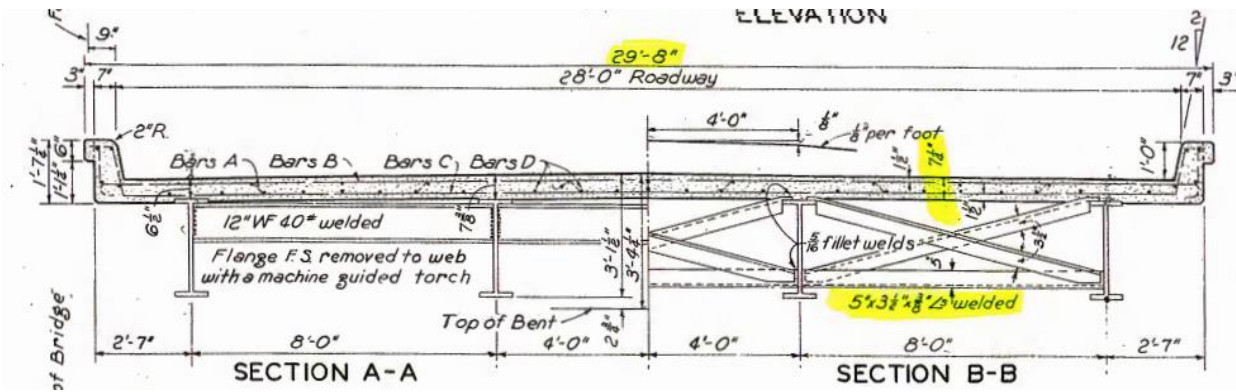
Bridge SM-11 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 521 ft 9 in. and includes thirteen simply supported spans. The controlling span for load rating is 40 ft and is 46 ft wide, with a roadway width of 43 ft 8 in. The bridge carries two lanes, one in each direction, and has an ADT of 1,230 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 26 US tons and an operating gross loading of 44 US tons. The bridge is posted for a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 69,000-lbs single vehicle, and a 80,000-lbs combination vehicle.



Figure B.21. Bridge SM-11 (TxDOT 2018c)



(a) Widening Details (1979)



(b) Transverse Section



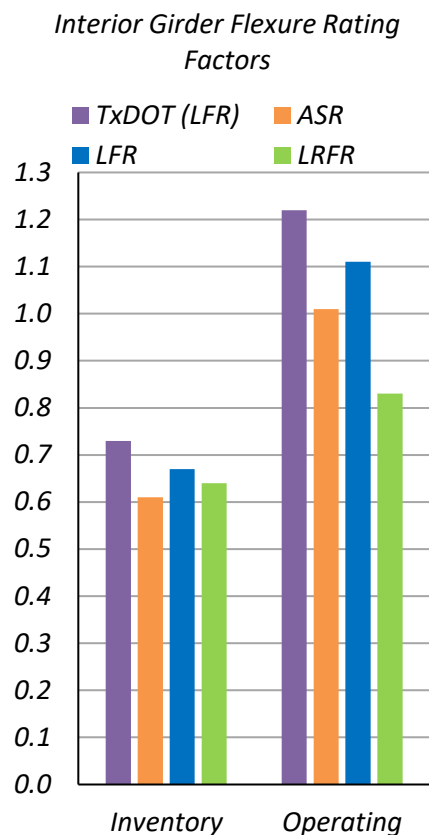
(c) Underside Superstructure View

Figure B.22. Bridge Drawings and Photos (TxDOT 2018c)

Table B.11. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-11

Bridge Characteristics	
Year Built	1947
On/Off-System	On
ADT	1230
Controlling Span Length	40'-0"
Deck Width	46'-0"
Roadway Width	43'-8"
Interior Girder Section	30WFX108
Exterior Girder Section	30WFX108

Interior Girder Spacing	7'-8" avg.
Slab Thickness (in.)	7.25
Wearing Surface Thickness (in.)	2.0
Composite	no
TxDOT Inventory RF	0.73
TxDOT Operating RF	1.22
TxDOT Inventory Rating (HS)	14.6
TxDOT Operating Rating (HS)	24.4
Posted Axle Limit (lbs)	see note 2



Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	658.2	C_{serv} (kip-ft)	658.2
C_{OP} (ksi)	24.75	C_n (kip-ft)	938.5	C_n (kip-ft)	822.8
f_{DL} (ksi)	8.2	M_{DL} (kip-ft)	205.3	M_{DL} (kip-ft)	205.3
$f_{LL-HS20}$ (ksi)	16.3	$M_{LL-HS20}$ (kip-ft)	407.6	$M_{LL-HL93}$ (kip-ft)	483.2
Inventory HS20 RF	0.61	Inventory HS20 RF	0.67	Inventory HL93 RF	0.64
Operating HS20 RF	1.01	Operating HS20 RF	1.11	Operating HL93 RF	0.83
Inventory Rating (HS)	12.2	Inventory Rating (HS)	13.4		
Operating Rating (HS)	20.2	Operating Rating (HS)	22.2		
		Serviceability Controls	yes	Serviceability Controls	yes

Notes:

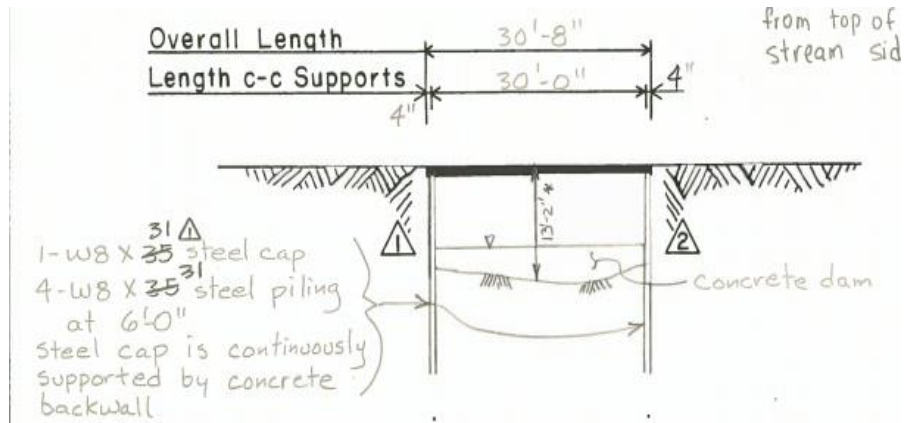
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 69,000-lbs single vehicle, and an 80,000-lbs combination vehicle.

B.1.12 Bridge SM-12

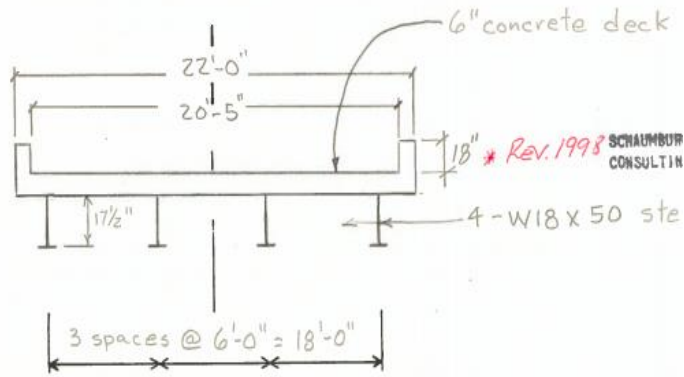
Bridge SM-12 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) with 3 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 30 ft 8 in., which is one simply supported span. The controlling span for load rating is 30 ft and is 22 ft wide, with a roadway width of 20 ft 5 in. The bridge carries one lane and has an ADT of 247 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 18 US tons and an operating gross loading of 31 US tons. The bridge is posted for a 24,000-lbs axle or tandem axle.



Figure B.23. Bridge SM-12 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.24. Bridge Drawings and Photos (TxDOT 2018c)

Table B.12. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-12

Bridge Characteristics					
Year Built	1955				
On/Off-System	Off				
ADT	247				
Controlling Span Length	30'-0"				
Deck Width	22'-0"				
Roadway Width	20'-5"				
Interior Girder Section	W18x50				
Exterior Girder Section	W18x50				
Interior Girder Spacing	6'-0"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.51				
TxDOT Operating RF	0.86				
TxDOT Inventory Rating (HS)	10.2				
TxDOT Operating Rating (HS)	17.1				
Posted Axle Limit (lbs)	24,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	189.7	C_{serv} (kip-ft)	189.7
C_{OP} (ksi)	24.75	C_n (kip-ft)	237.1	C_n (kip-ft)	269.4
f_{DL} (ksi)	9.3	M_{DL} (kip-ft)	66.7	M_{DL} (kip-ft)	66.7
$f_{LL-HS20}$ (ksi)	27.8	$M_{LL-HS20}$ (kip-ft)	200.0	$M_{LL-HL93}$ (kip-ft)	318.3
Inventory HS20 RF	0.32	Inventory HS20 RF	0.35	Inventory HL93 RF	0.26
Operating HS20 RF	0.56	Operating HS20 RF	0.58	Operating HL93 RF	0.33
Inventory Rating (HS)	6.4	Inventory Rating (HS)	7		
Operating Rating (HS)	11.2	Operating Rating (HS)	11.6		
		Serviceability Controls	no	Serviceability Controls	yes

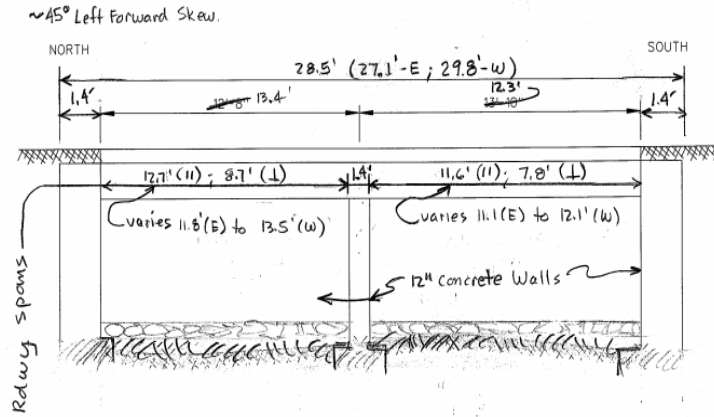
Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.13 Bridge SM-13

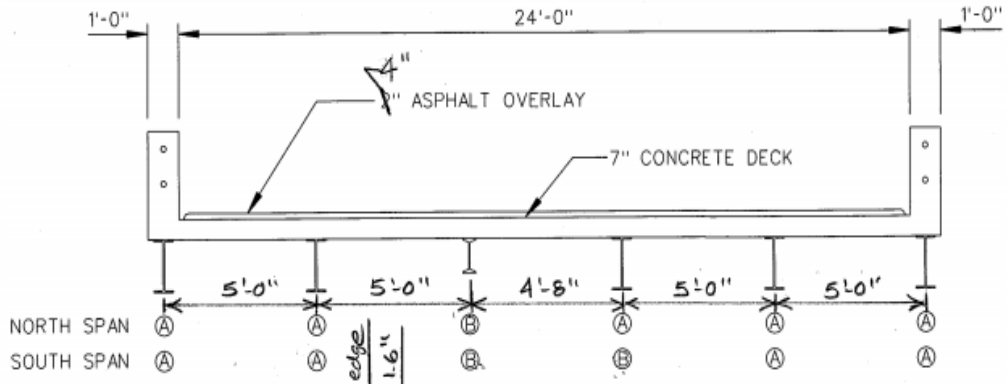
Bridge SM-13 has a deck condition rating of 5 (Fair), a superstructure condition rating of 5 (Fair) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 28 ft 6 in. and includes two simply supported spans. The controlling span for load rating is 13 ft 6 in. and is 26 ft wide, with a roadway width of 24 ft. The bridge has a 45 degree skew, carries two lanes, one in each direction, and has an ADT of 590 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 23 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure B.25. Bridge SM-13 (TxDOT 2018c)



(a) Elevation View



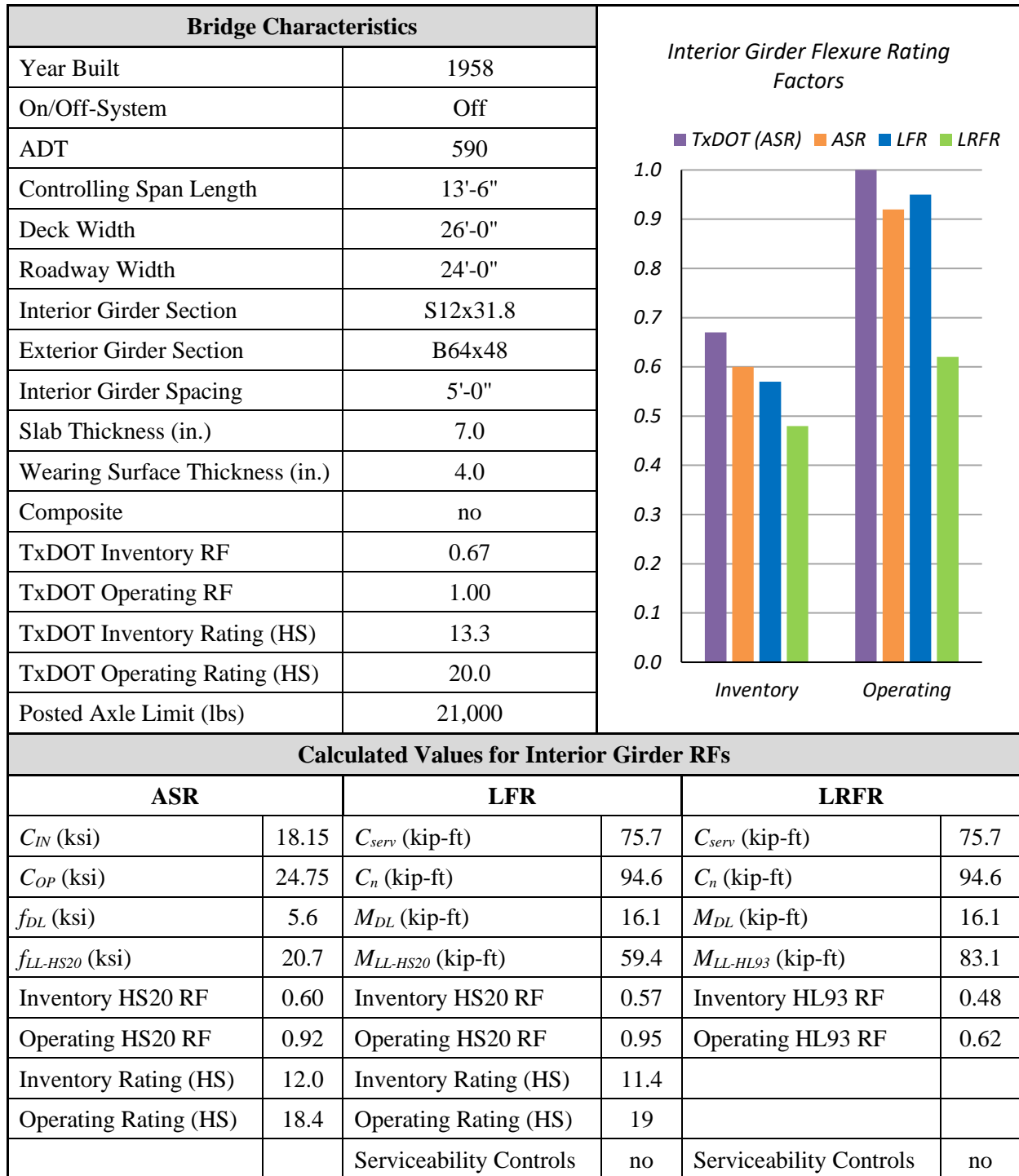
(b) Transverse Section



(c) Underside Superstructure View

Figure B.26. Bridge Drawings and Photos (TxDOT 2018c)

Table B.13. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-13



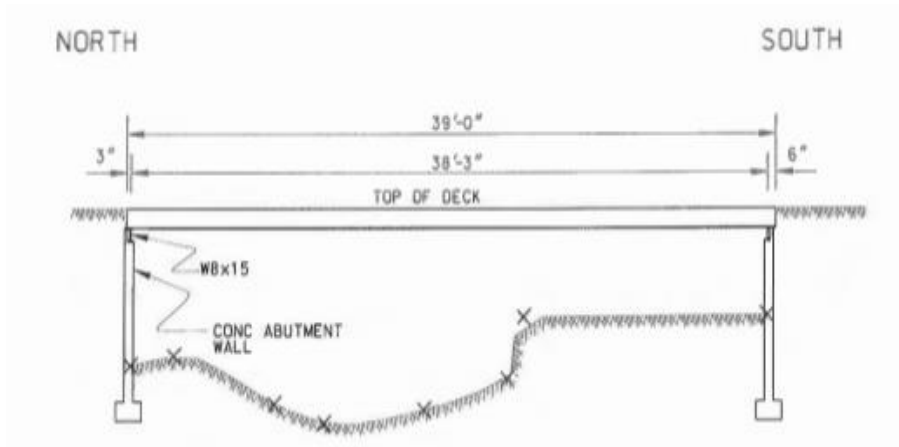
Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.14 Bridge SM-14

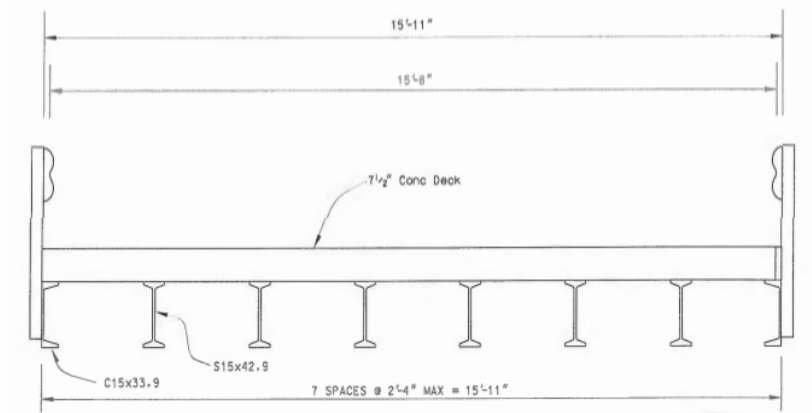
Bridge SM-14 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 39 ft, which is one simply supported span. The controlling span for load rating is 38 ft 3 in. and is 15 ft 11 in. wide, with a roadway width of 15 ft 8 in. The bridge carries one lane and has an ADT of 80 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 15 US tons and an operating gross loading of 27 US tons. The bridge is posted for a 15,000-lbs axle or tandem axle.



Figure B.27. Bridge SM-14 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.28. Bridge Drawings and Photos (TxDOT 2018c)

Table B.14. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-14

Bridge Characteristics	
Year Built	1960
On/Off-System	Off
ADT	80
Controlling Span Length	38'-3"
Deck Width	15'-11"
Roadway Width	15'-8"
Interior Girder Section	S15x42.9
Exterior Girder Section	C15x33.9
Interior Girder Spacing	2'-4"
Slab Thickness (in.)	7.5
Wearing Surface Thickness (in.)	0.0
Composite	no
TxDOT Inventory RF	0.41
TxDOT Operating RF	0.75
TxDOT Inventory Rating (HS)	8.2
TxDOT Operating Rating (HS)	15.0
Posted Axle Limit (lbs)	15,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.41	0.38	0.48	0.25
Operating	0.75	0.72	0.81	0.32

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	124.1	C_{serv} (kip-ft)	124.1
C_{OP} (ksi)	24.75	C_n (kip-ft)	180.8	C_n (kip-ft)	180.8
f_{DL} (ksi)	11.0	M_{DL} (kip-ft)	51.6	M_{DL} (kip-ft)	51.6
$f_{LL-HS20}$ (ksi)	19.1	$M_{LL-HS20}$ (kip-ft)	89.7	$M_{LL-HL93}$ (kip-ft)	185.7
Inventory HS20 RF	0.38	Inventory HS20 RF	0.48	Inventory HL93 RF	0.25
Operating HS20 RF	0.72	Operating HS20 RF	0.81	Operating HL93 RF	0.32
Inventory Rating (HS)	7.6	Inventory Rating (HS)	9.6		
Operating Rating (HS)	14.4	Operating Rating (HS)	16.2		
		Serviceability Controls	yes	Serviceability Controls	yes

Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.15 Bridge SM-15

Bridge Characteristics	
Year Built	1961
On/Off-System	On
ADT	1150
Controlling Span Length	42'-0"
Deck Width	25'-8"
Roadway Width	24'-0"
Interior Girder Section	30WFx116
Exterior Girder Section	30WFx116
Interior Girder Spacing	9'-6"
Slab Thickness (in.)	7.0
Wearing Surface Thickness (in.)	2.0
Composite	no
TxDOT Inventory RF	0.51
TxDOT Operating RF	0.85
TxDOT Inventory Rating (HS)	10.1
TxDOT Operating Rating (HS)	16.9
Posted Axle Limit (lbs)	28,000

Rating Type	TxDOT (LFR)	ASR	LFR	LRFR
Inventory	0.51	0.40	0.46	0.44
Operating	0.85	0.73	0.76	0.57

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	721.4	C_{serv} (kip-ft)	721.4
C_{OP} (ksi)	24.75	C_n (kip-ft)	901.7	C_n (kip-ft)	1026.0
f_{DL} (ksi)	10.2	M_{DL} (kip-ft)	279.4	M_{DL} (kip-ft)	279.4
$f_{LL-HS20}$ (ksi)	19.9	$M_{LL-HS20}$ (kip-ft)	544.7	$M_{LL-HL93}$ (kip-ft)	650.5
Inventory HS20 RF	0.40	Inventory HS20 RF	0.46	Inventory HL93 RF	0.44
Operating HS20 RF	0.73	Operating HS20 RF	0.76	Operating HL93 RF	0.57
Inventory Rating (HS)	8.0	Inventory Rating (HS)	9.2		
Operating Rating (HS)	14.6	Operating Rating (HS)	15.2		
		Serviceability Controls	no	Serviceability Controls	yes

Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

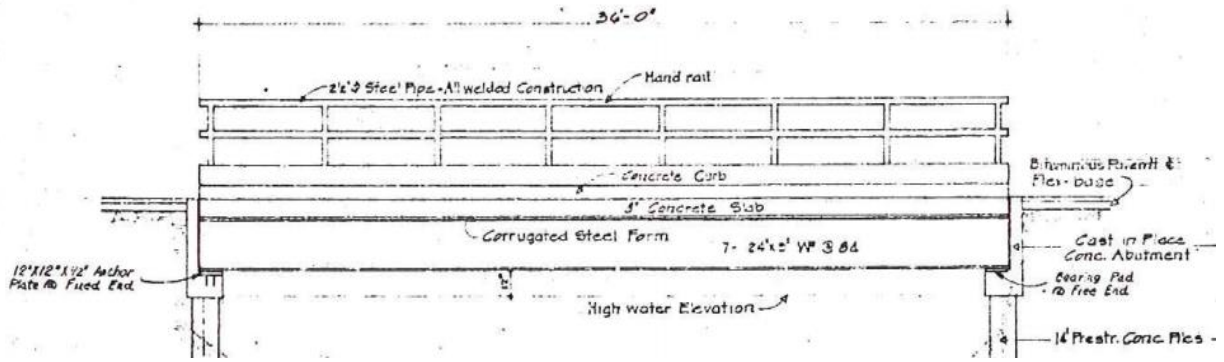
B.1.16 Bridge SM-16

Bridge SM-16 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 7

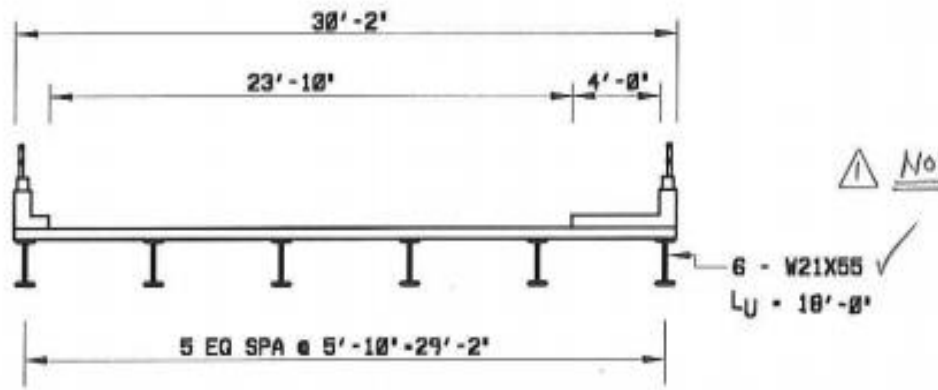
(Good). The bridge has a total length of 37 ft 4 in., which is one simply supported span. The controlling span for load rating is 36 ft and is 30 ft 2 in. wide, with a roadway width of 23 ft 10 in. The bridge carries two lanes, one in each direction, and has an ADT of 730 vehicles. The assumed substructure rating controls the rating of the bridge, which has an inventory gross loading of 29 US tons and an operating gross loading of 44 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure B.31. Bridge SM-16 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.32. Bridge Drawings and Photos (TxDOT 2018c)

Table B.16. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-16

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1965				
On/Off-System	Off				
ADT	730				
Controlling Span Length	26'-0"				
Deck Width	30'-2"				
Roadway Width	23'-10"				
Interior Girder Section	W21x55				
Exterior Girder Section	W21x55				
Interior Girder Spacing	5'-11"				
Slab Thickness (in.)	7.0				
Wearing Surface Thickness (in.)	0.0				
Composite	yes				
TxDOT Inventory RF	0.82				
TxDOT Operating RF	1.24				
TxDOT Inventory Rating (HS)	16.3				
TxDOT Operating Rating (HS)	24.7				
Posted Axle Limit (lbs)	28,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	27.5	C_{serv} (kip-ft)	47.5	C_{serv} (kip-ft)	47.5
C_{OP} (ksi)	37.5	C_n (kip-ft)	986.2	C_n (kip-ft)	924.2
f_{DL} (ksi)	8.1	M_{DL} (kip-ft)	118.9	M_{DL} (kip-ft)	118.9
$f_{LL-HS20}$ (ksi)	17.7	$M_{LL-HS20}$ (kip-ft)	261.1	$M_{LL-HL93}$ (kip-ft)	380.8
Inventory HS20 RF	0.86	Inventory HS20 RF	1.20	Inventory HL93 RF	0.96
Operating HS20 RF	1.43	Operating HS20 RF	1.99	Operating HL93 RF	1.25
Inventory Rating (HS)	17.2	Inventory Rating (HS)	24		
Operating Rating (HS)	28.6	Operating Rating (HS)	39.8		
$S_{x,n=n}$ (in ³)	176.9	Serviceability Controls	yes	Serviceability Controls	yes

Notes:

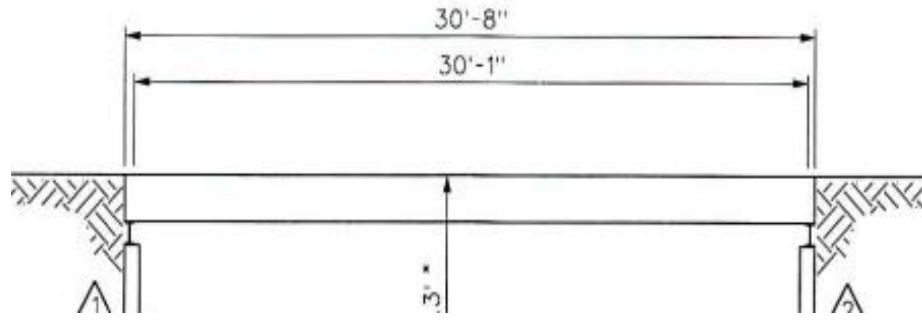
1. Steel yield strength taken as 50 ksi per city records in TxDOT’s Bridge Inventory Sheet.
2. Composite section calculations follow the procedure shown in MBE APPENDIX A Example A1.

B.1.17 Bridge SM-17

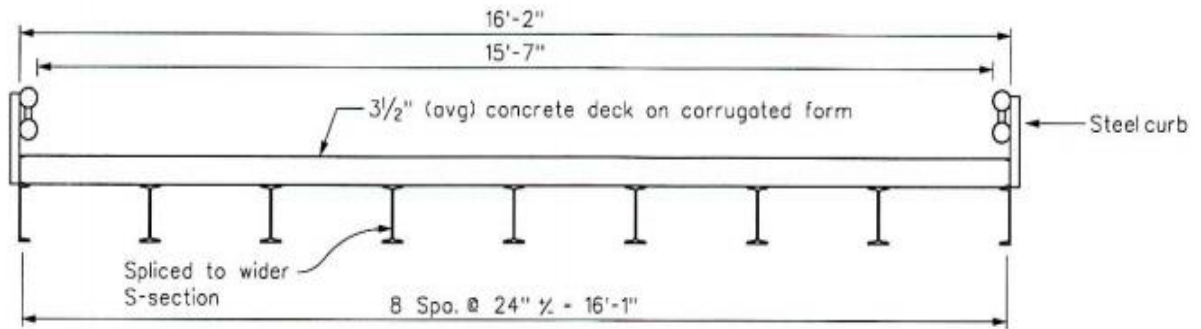
Bridge SM-17 has a deck condition rating of 7 (Good), a superstructure condition rating of 5 (Fair) with 10 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 31 ft, which is one simply supported span. The controlling span for load rating is 30 ft 1 in. and is 16 ft 2 in. wide, with a roadway width of 15 ft 7 in. The bridge carries one lane and has an ADT of 70 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 20 US tons and an operating gross loading of 32 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure B.33. Bridge SM-17 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.34. Bridge Drawings and Photos (TxDOT 2018c)

Table B.17. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-17

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1970				
On/Off-System	Off				
ADT	70				
Controlling Span Length	30'-1"				
Deck Width	16'-2"				
Roadway Width	15'-7"				
Interior Girder Section	S12x31.8				
Exterior Girder Section	C12x20.7				
Interior Girder Spacing	2'-0"				
Slab Thickness (in.)	3.5				
Wearing Surface Thickness (in.)	3.0				
Composite	no				
TxDOT Inventory RF	0.55				
TxDOT Operating RF	0.89				
TxDOT Inventory Rating (HS)	11.0				
TxDOT Operating Rating (HS)	17.7				
Posted Axle Limit (lbs)	24,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	71.7	C_{serv} (kip-ft)	71.7
C_{OP} (ksi)	24.75	C_n (kip-ft)	103.5	C_n (kip-ft)	103.5
f_{DL} (ksi)	7.8	M_{DL} (kip-ft)	21.2	M_{DL} (kip-ft)	21.2
$f_{LL-HS20}$ (ksi)	19.3	$M_{LL-HS20}$ (kip-ft)	52.4	$M_{LL-HL93}$ (kip-ft)	133.9
Inventory HS20 RF	0.54	Inventory HS20 RF	0.58	Inventory HL93 RF	0.26
Operating HS20 RF	0.88	Operating HS20 RF	0.97	Operating HL93 RF	0.34
Inventory Rating (HS)	10.8	Inventory Rating (HS)	11.6		
Operating Rating (HS)	17.6	Operating Rating (HS)	19.4		
		Serviceability Controls	yes	Serviceability Controls	yes

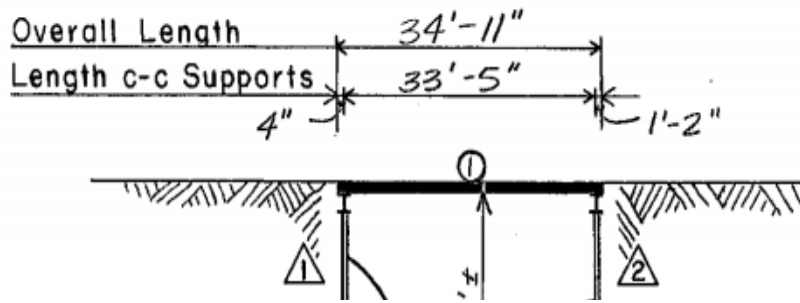
Note: Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.18 Bridge SM-18

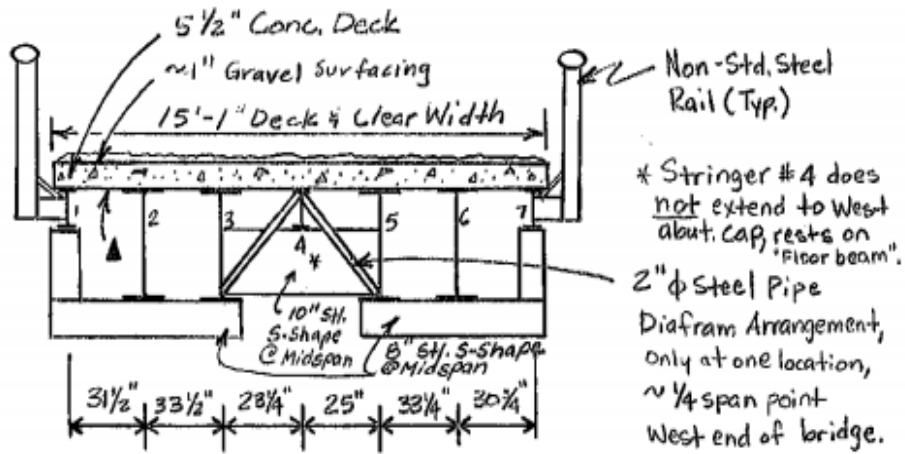
Bridge SM-18 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 5 (Fair) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 35 ft, which is one simply supported span. The controlling span for load rating is 33 ft 5 in. and is 15 ft 1 in. wide, with a roadway width of 15 ft 1 in. The bridge carries one lane and has an ADT of 100 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 21 US tons and an operating gross loading of 34 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure B.35. Bridge SM-18 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.36. Bridge Drawings and Photos (TxDOT 2018c)

Table B.18. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-18

Bridge Characteristics	
Year Built	1977
On/Off-System	Off
ADT	100
Controlling Span Length	33'-5"
Deck Width	15'-1"
Roadway Width	15'-1"
Interior Girder Section	S18x54.7
Exterior Girder Section	S18x54.7
Interior Girder Spacing	2'-10"
Slab Thickness (in.)	5.5
Wearing Surface Thickness (in.)	1.0
Composite	no
TxDOT Inventory RF	0.59
TxDOT Operating RF	0.95
TxDOT Inventory Rating (HS)	11.7
TxDOT Operating Rating (HS)	19.0
Posted Axle Limit (lbs)	21,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.59	0.75	0.75	0.45
Operating	0.95	1.20	1.25	0.58

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	14.85	C_{serv} (kip-ft)	157.0	C_{serv} (kip-ft)	157.0
C_{OP} (ksi)	20.25	C_n (kip-ft)	196.2	C_n (kip-ft)	229.3
f_{DL} (ksi)	5.5	M_{DL} (kip-ft)	40.0	M_{DL} (kip-ft)	40.0
$f_{LL-HS20}$ (ksi)	12.3	$M_{LL-HS20}$ (kip-ft)	89.1	$M_{LL-HL93}$ (kip-ft)	184.4
Inventory HS20 RF	0.76	Inventory HS20 RF	0.75	Inventory HL93 RF	0.45
Operating HS20 RF	1.20	Operating HS20 RF	1.25	Operating HL93 RF	0.58
Inventory Rating (HS)	15.2	Inventory Rating (HS)	15		
Operating Rating (HS)	24.0	Operating Rating (HS)	25		
		Serviceability Controls	no	Serviceability Controls	yes

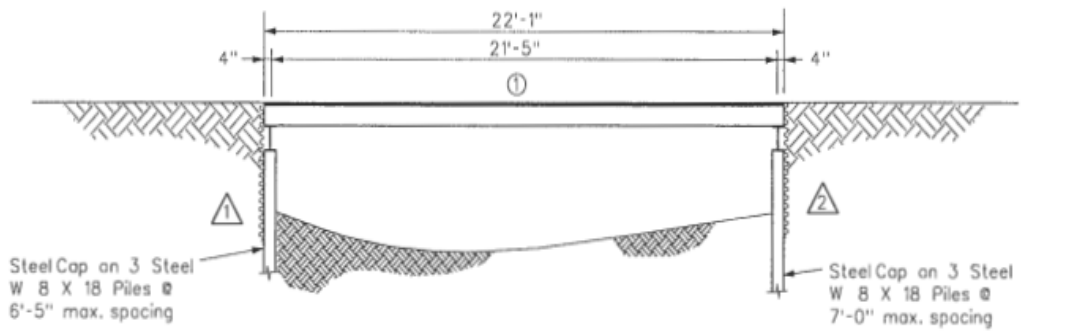
Note: Steel yield strength taken as 27 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.19 Bridge SM-19

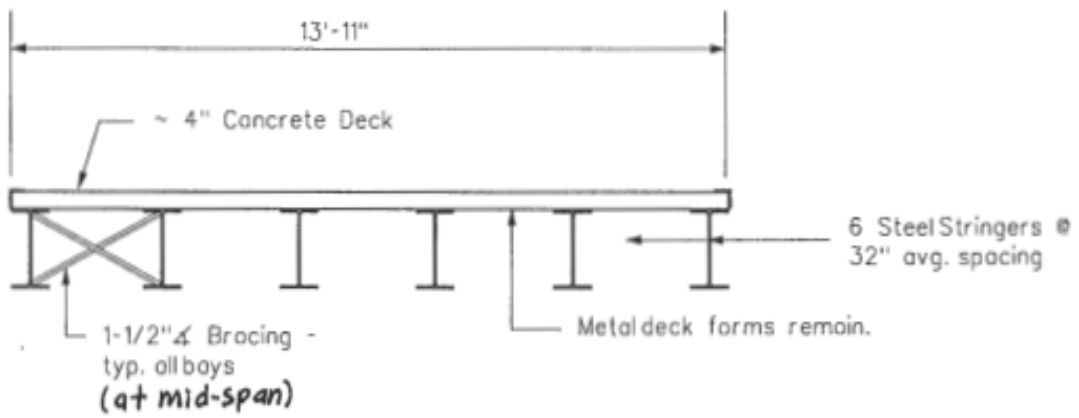
Bridge SM-19 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 22 ft, which is one simply supported span. The controlling span for load rating is 21 ft 5 in. and is 13 ft 11 in. wide, with a roadway width of 13 ft 11 in. The bridge carries one lane and has an ADT of 20 vehicles. The steel caps control the rating of the bridge, which has an inventory gross loading of 18 US tons and an operating gross loading of 26 US tons. The bridge is posted for a 17,500-lbs axle or tandem axle.



Figure B.37. Bridge SM-19 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.38. Bridge Drawings and Photos (TxDOT 2018c)

Table B.19. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-19

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1982				
On/Off-System	Off				
ADT	20				
Controlling Span Length	21'-5"				
Deck Width	13'-11"				
Roadway Width	13'-11"				
Interior Girder Section	S10x25.4				
Exterior Girder Section	S10x25.4				
Interior Girder Spacing	2'-8"				
Slab Thickness (in.)	4.0				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.71				
TxDOT Operating RF	1.04				
TxDOT Inventory Rating (HS)	14.2				
TxDOT Operating Rating (HS)	20.8				
Posted Axle Limit (lbs)	17,500				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	56.1	C_{serv} (kip-ft)	56.1
C_{OP} (ksi)	27.0	C_n (kip-ft)	70.1	C_n (kip-ft)	80.7
f_{DL} (ksi)	5.1	M_{DL} (kip-ft)	10.0	M_{DL} (kip-ft)	10.0
$f_{LL-HS20}$ (ksi)	21.4	$M_{LL-HS20}$ (kip-ft)	41.6	$M_{LL-HL93}$ (kip-ft)	107.6
Inventory HS20 RF	0.69	Inventory HS20 RF	0.63	Inventory HL93 RF	0.31
Operating HS20 RF	1.02	Operating HS20 RF	1.06	Operating HL93 RF	0.41
Inventory Rating (HS)	13.8	Inventory Rating (HS)	12.6		
Operating Rating (HS)	20.4	Operating Rating (HS)	21.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

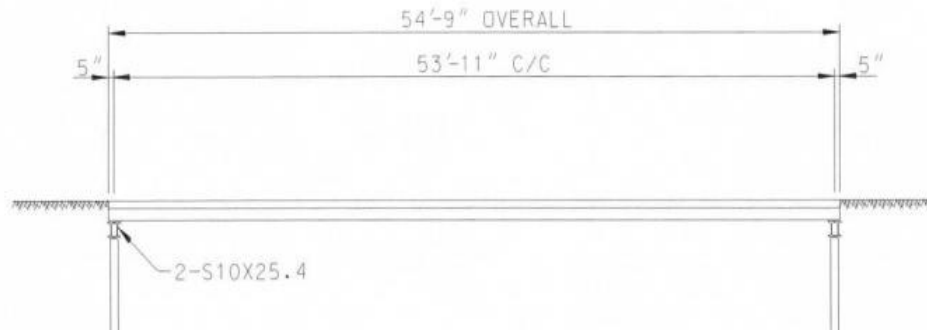
1. Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. The steel cap of the substructure controls the ratings. These calculations were not performed as a part of this project. The ratings shown in this table are for the interior girders.

B.1.20 Bridge SM-20

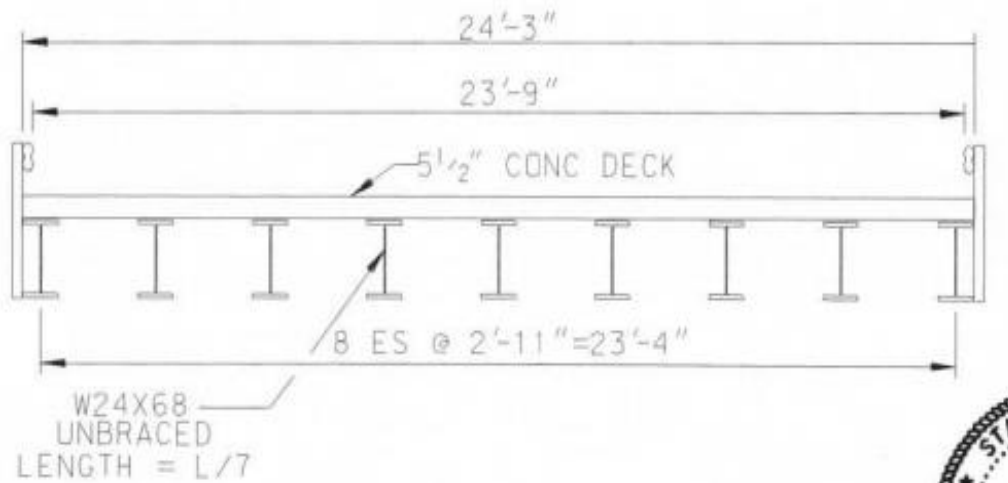
Bridge SM-20 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 54 ft 9 in., which is one simply supported span. The controlling span for load rating is 53 ft 11 in. and is 24 ft 3 in. wide, with a roadway width of 23 ft 9 in. The bridge carries two lanes, one in each direction, and has an ADT of 330 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 20 US tons and an operating gross loading of 33 US tons. The bridge is posted for a 24,000-lbs tandem axle and a 40,000-lbs gross vehicle.



Figure B.39. Bridge SM-20 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.40. Bridge Drawings and Photos (TxDOT 2018c)

Table B.20. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-20

Bridge Characteristics					
Year Built	1986				
On/Off-System	Off				
ADT	330				
Controlling Span Length	53'-11"				
Deck Width	24'-3"				
Roadway Width	23'-9"				
Interior Girder Section	W24x68				
Exterior Girder Section	W24x68				
Interior Girder Spacing	2'-11"				
Slab Thickness (in.)	5.5				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.56				
TxDOT Operating RF	0.91				
TxDOT Inventory Rating (HS)	11.2				
TxDOT Operating Rating (HS)	18.2				
Posted Axle Limit (lbs)	see note 2				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	338.8	C_{serv} (kip-ft)	338.8
C_{OP} (ksi)	24.75	C_n (kip-ft)	486.8	C_n (kip-ft)	486.8
f_{DL} (ksi)	8.5	M_{DL} (kip-ft)	109.1	M_{DL} (kip-ft)	109.1
$f_{LL-HS20}$ (ksi)	18.5	$M_{LL-HS20}$ (kip-ft)	237.3	$M_{LL-HL93}$ (kip-ft)	413.6
Inventory HS20 RF	0.52	Inventory HS20 RF	0.58	Inventory HL93 RF	0.38
Operating HS20 RF	0.88	Operating HS20 RF	0.97	Operating HL93 RF	0.49
Inventory Rating (HS)	10.4	Inventory Rating (HS)	11.6		
Operating Rating (HS)	17.6	Operating Rating (HS)	19.4		
		Serviceability Controls	yes	Serviceability Controls	yes

Notes:

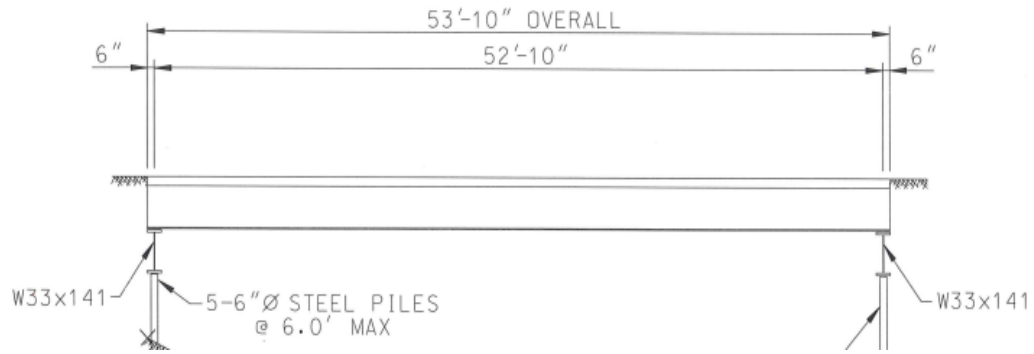
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 24,000-lbs tandem axle and 40,000-lbs gross.

B.1.21 Bridge SM-21

Bridge SM-21 has a deck condition rating of 8 (Very Good), a superstructure condition rating of 7 (Good) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 54 ft, which is one simply supported span. The controlling span for load rating is 52 ft 10 in. and is 25 ft wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 550 vehicles. The assumed deck rating controls the rating of the bridge, which has an inventory gross loading of 25 US tons and an operating gross loading of 36 US tons. The bridge was once posted for a 28,000-lbs tandem axle and a 52,000-lbs gross vehicle; however, it is no longer posted.

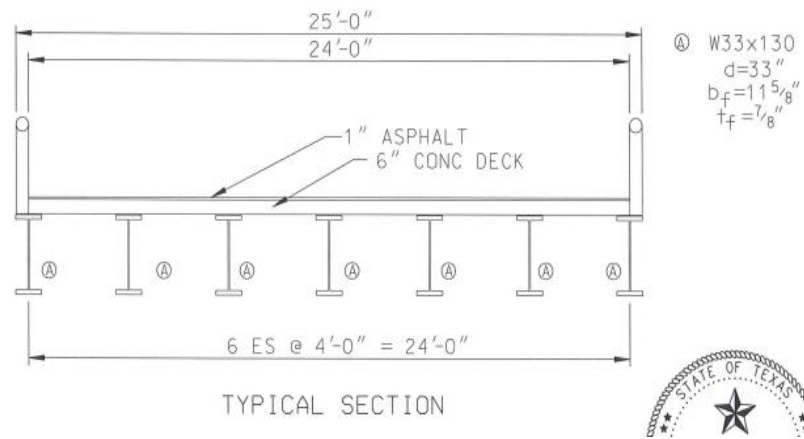


Figure B.41. Bridge SM-21 (TxDOT 2018c)



(a) Elevation View

NOTE: UNBRACED LENGTH = $L/3$



(b) Transverse Section



(c) Underside Superstructure View

Figure B.42. Bridge Drawings and Photos (TxDOT 2018c)

Table B.21. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-21

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1990				
On/Off-System	Off				
ADT	550				
Controlling Span Length	52'-10"				
Deck Width	25'-0"				
Roadway Width	24'-0"				
Interior Girder Section	W33x130				
Exterior Girder Section	W33x130				
Interior Girder Spacing	4'-0"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	1.0				
Composite	no				
TxDOT Inventory RF	1.35				
TxDOT Operating RF	2.02				
TxDOT Inventory Rating (HS)	27.0				
TxDOT Operating Rating (HS)	40.4				
Posted Axle Limit (lbs)	none				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	875.3	C_{serv} (kip-ft)	875.3
C_{OP} (ksi)	24.75	C_n (kip-ft)	1094.0	C_n (kip-ft)	1259.0
f_{DL} (ksi)	5.4	M_{DL} (kip-ft)	180.0	M_{DL} (kip-ft)	180.0
$f_{LL-HS20}$ (ksi)	9.6	$M_{LL-HS20}$ (kip-ft)	317.5	$M_{LL-HL93}$ (kip-ft)	495.0
Inventory HS20 RF	1.33	Inventory HS20 RF	1.25	Inventory HL93 RF	1.01
Operating HS20 RF	2.02	Operating HS20 RF	2.08	Operating HL93 RF	1.31
Inventory Rating (HS)	26.6	Inventory Rating (HS)	25		
Operating Rating (HS)	40.4	Operating Rating (HS)	41.6		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

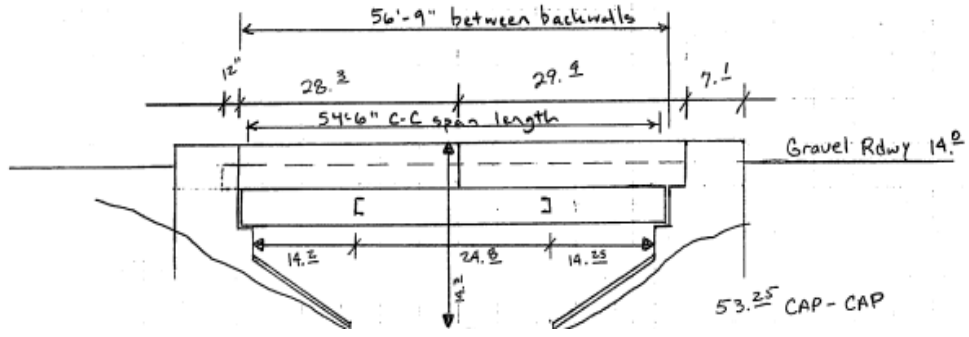
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting has been removed per inspection dated 11/13/2017.

B.1.22 Bridge SM-22

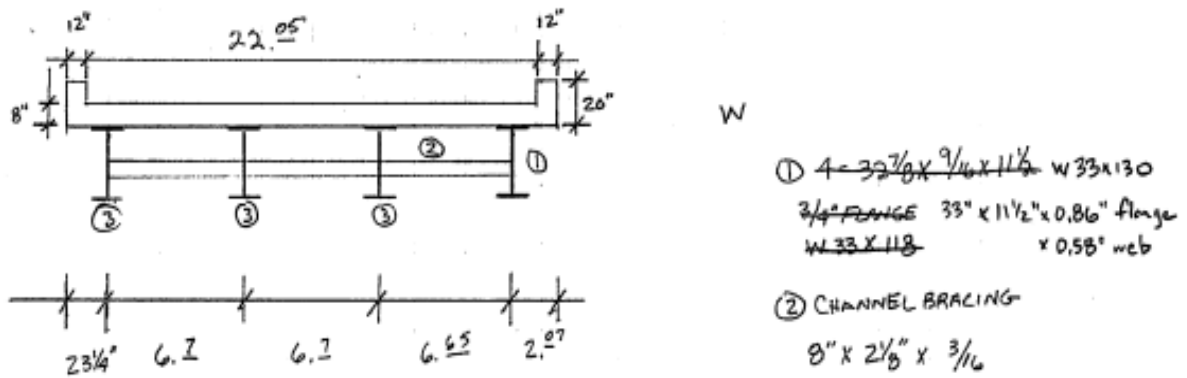
Bridge SM-22 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 58 ft, which is one simply supported span. The controlling span for load rating is 54 ft 6 in. and is 24 ft wide, with a roadway width of 22 ft. The bridge carries two lanes, one in each direction, and has an ADT of 250 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 13 US tons and an operating gross loading of 31 US tons. The bridge is posted for a 15,000-lbs axle or tandem axle and a 28,000-lbs gross vehicle.



Figure B.43. Bridge SM-22 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.44. Bridge Drawings and Photos (TxDOT 2018c)

Table B.22. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-22

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1990				
On/Off-System	Off				
ADT	250				
Controlling Span Length	54'-6"				
Deck Width	24'-0"				
Roadway Width	22'-0"				
Interior Girder Section	W33x118				
Exterior Girder Section	W33x130				
Interior Girder Spacing	6'-8"				
Slab Thickness (in.)	8.0				
Wearing Surface Thickness (in.)	5.0				
Composite	no				
TxDOT Inventory RF	0.38				
TxDOT Operating RF	0.87				
TxDOT Inventory Rating (HS)	7.5				
TxDOT Operating Rating (HS)	17.3				
Posted Axle Limit (lbs)	see note 2				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	844.4	C_{serv} (kip-ft)	844.4
C_{OP} (ksi)	27.0	C_n (kip-ft)	1055.0	C_n (kip-ft)	1220.0
f_{DL} (ksi)	16.8	M_{DL} (kip-ft)	493.0	M_{DL} (kip-ft)	493.0
$f_{LL-HS20}$ (ksi)	18.6	$M_{LL-HS20}$ (kip-ft)	544.7	$M_{LL-HL93}$ (kip-ft)	719.6
Inventory HS20 RF	0.16	Inventory HS20 RF	0.35	Inventory HL93 RF	0.24
Operating HS20 RF	0.55	Operating HS20 RF	0.59	Operating HL93 RF	0.32
Inventory Rating (HS)	3.2	Inventory Rating (HS)	7		
Operating Rating (HS)	11.0	Operating Rating (HS)	11.8		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

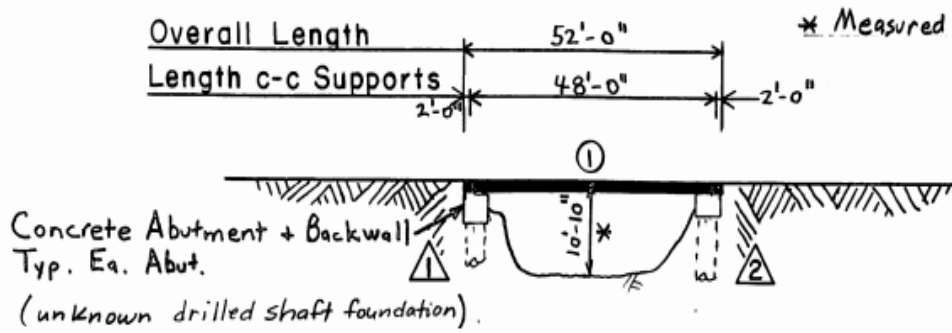
1. Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 15,000-lbs axle or tandem axle and 28,000-lbs gross.

B.1.23 Bridge SM-23

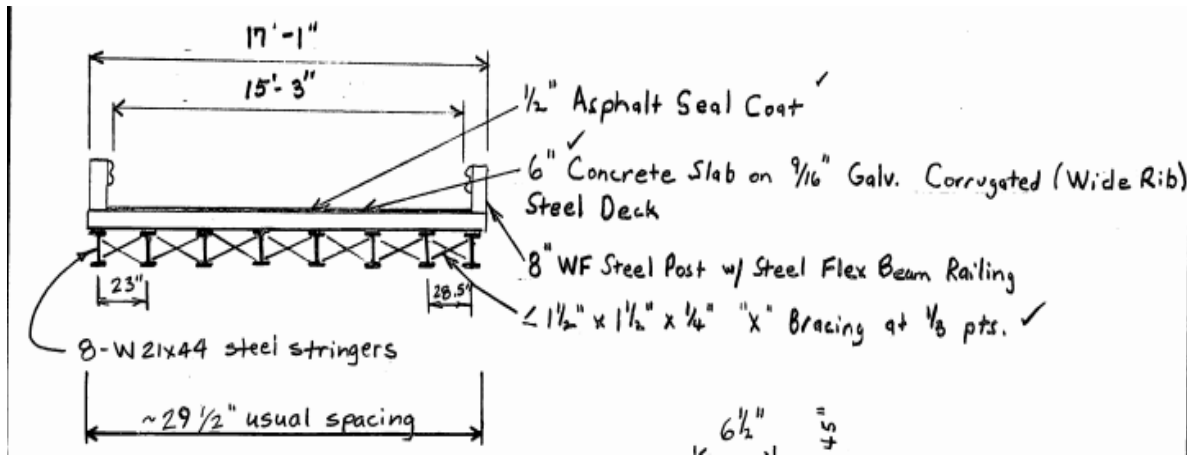
Bridge SM-23 has a deck condition rating of 8 (Very Good), a superstructure condition rating of 6 (Satisfactory) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 52 ft, which is one simply supported span. The controlling span for load rating is 48 ft and is 17 ft 1 in. wide, with a roadway width of 15 ft 3 in. The bridge carries one lane and has an ADT of 60 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 18 US tons and an operating gross loading of 32 US tons. The bridge is posted for a 21,000-lbs tandem axle and a 36,000-lbs gross vehicle.



Figure B.45. Bridge SM-23 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.46. Bridge Drawings and Photos (TxDOT 2018c)

Table B.23. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-23

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1992				
On/Off-System	Off				
ADT	60				
Controlling Span Length	48'-0"				
Deck Width	17'-1"				
Roadway Width	15'-3"				
Interior Girder Section	W21x44				
Exterior Girder Section	W21x44				
Interior Girder Spacing	2'-6" typ.				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	0.5				
Composite	no				
TxDOT Inventory RF	0.51				
TxDOT Operating RF	0.88				
TxDOT Inventory Rating (HS)	10.2				
TxDOT Operating Rating (HS)	17.6				
Posted Axle Limit (lbs)	see note 2				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	191.9	C_{serv} (kip-ft)	191.9
C_{OP} (ksi)	27.0	C_n (kip-ft)	239.9	C_n (kip-ft)	280.5
f_{DL} (ksi)	11.4	M_{DL} (kip-ft)	75.8	M_{DL} (kip-ft)	75.8
$f_{LL-HS20}$ (ksi)	20.1	$M_{LL-HS20}$ (kip-ft)	134.0	$M_{LL-HL93}$ (kip-ft)	260.5
Inventory HS20 RF	0.42	Inventory HS20 RF	0.49	Inventory HL93 RF	0.29
Operating HS20 RF	0.78	Operating HS20 RF	0.81	Operating HL93 RF	0.37
Inventory Rating (HS)	8.4	Inventory Rating (HS)	9.8		
Operating Rating (HS)	15.6	Operating Rating (HS)	16.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

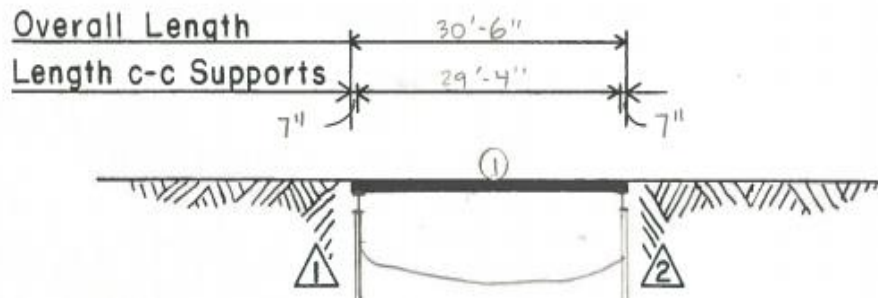
1. Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 21,000-lbs tandem axle and 36,000-lbs gross.

B.1.24 Bridge SM-24

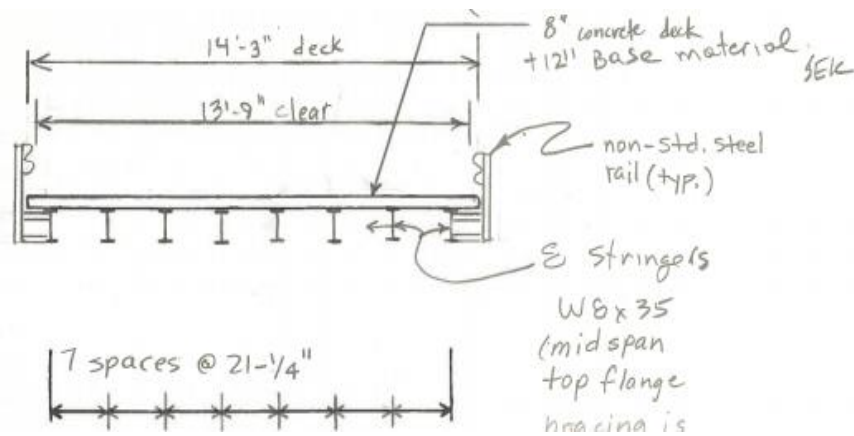
Bridge SM-24 has a deck condition rating of 8 (Very Good), a superstructure condition rating of 5 (Fair) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 30 ft 6 in., which is one simply supported span. The controlling span for load rating is 29 ft 4 in. and is 14 ft 3 in. wide, with a roadway width of 13 ft 9 in. The bridge carries one lane and has an ADT of 50 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 21 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure B.47. Bridge SM-24 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.48. Bridge Drawings and Photos (TxDOT 2018c)

Table B.24. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-24

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1995				
On/Off-System	Off				
ADT	50				
Controlling Span Length	29'-4"				
Deck Width	14'-3"				
Roadway Width	13'-9"				
Interior Girder Section	W8x35				
Exterior Girder Section	W8x35				
Interior Girder Spacing	1'-9" typ.				
Slab Thickness (in.)	8.0				
Wearing Surface Thickness (in.)	12.0				
Composite	no				
TxDOT Inventory RF	0.60				
TxDOT Operating RF	0.99				
TxDOT Inventory Rating (HS)	11.9				
TxDOT Operating Rating (HS)	19.8				
Posted Axle Limit (lbs)	21,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	71.1	C_{serv} (kip-ft)	71.1
C_{OP} (ksi)	27.0	C_n (kip-ft)	88.9	C_n (kip-ft)	98.9
f_{DL} (ksi)	12.6	M_{DL} (kip-ft)	31.2	M_{DL} (kip-ft)	31.2
$f_{LL-HS20}$ (ksi)	17.8	$M_{LL-HS20}$ (kip-ft)	43.9	$M_{LL-HL93}$ (kip-ft)	120.7
Inventory HS20 RF	0.40	Inventory HS20 RF	0.51	Inventory HL93 RF	0.21
Operating HS20 RF	0.81	Operating HS20 RF	0.85	Operating HL93 RF	0.27
Inventory Rating (HS)	8.0	Inventory Rating (HS)	10.2		
Operating Rating (HS)	16.2	Operating Rating (HS)	17		
		Serviceability Controls	no	Serviceability Controls	yes

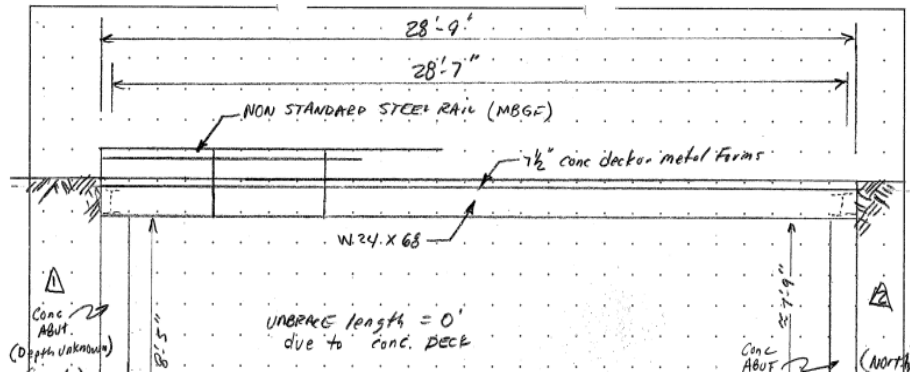
Note: Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

B.1.25 Bridge SM-25

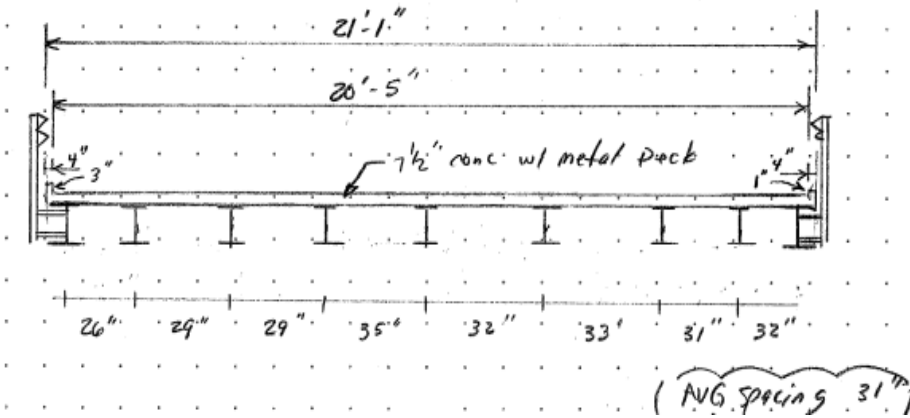
Bridge SM-25 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 28 ft 9 in., which is one simply supported span. The controlling span for load rating is 28 ft 7 in. and is 21 ft 1 in. wide, with a roadway width of 20 ft 5 in. The bridge carries two lanes, one in each direction, and has an ADT of 10 vehicles. The assumed substructure rating controls the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure B.49. Bridge SM-25 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.50. Bridge Drawings and Photos (TxDOT 2018c)

Table B.25. Characteristics and Interior Girder Flexure Load Ratings of Bridge SM-25

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	2000				
On/Off-System	Off				
ADT	10				
Controlling Span Length	28'-7"				
Deck Width	21'-1"				
Roadway Width	20'-5"				
Interior Girder Section	W24x68				
Exterior Girder Section	W24x68				
Interior Girder Spacing	2'-7" avg.				
Slab Thickness (in.)	7.5				
Wearing Surface Thickness (in.)	1.0				
Composite	no				
TxDOT Inventory RF	3.37				
TxDOT Operating RF	4.77				
TxDOT Inventory Rating (HS)	67.4				
TxDOT Operating Rating (HS)	95.4				
Posted Axle Limit (lbs)	21,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	351.1	C_{serv} (kip-ft)	351.1
C_{OP} (ksi)	27.0	C_n (kip-ft)	438.9	C_n (kip-ft)	504.5
f_{DL} (ksi)	2.9	M_{DL} (kip-ft)	35.3	M_{DL} (kip-ft)	35.3
$f_{LL-HS20}$ (ksi)	6.7	$M_{LL-HS20}$ (kip-ft)	81.5	$M_{LL-HL93}$ (kip-ft)	174.5
Inventory HS20 RF	2.53	Inventory HS20 RF	2.22	Inventory HL93 RF	1.35
Operating HS20 RF	3.61	Operating HS20 RF	3.71	Operating HL93 RF	1.76
Inventory Rating (HS)	50.6	Inventory Rating (HS)	44.4		
Operating Rating (HS)	72.2	Operating Rating (HS)	74.2		
		Serviceability Controls	no	Serviceability Controls	yes

Note: Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.

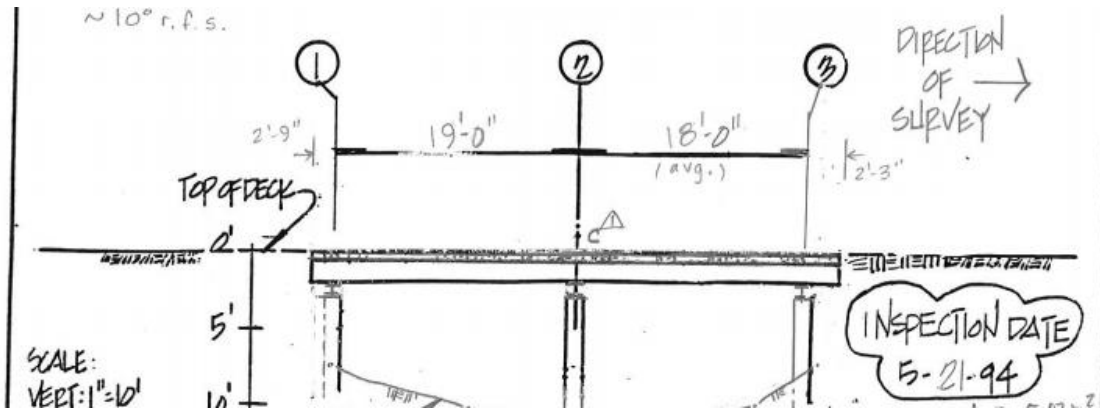
B.2 STEEL CONTINUOUS MULTI-GIRDER BRIDGES

B.2.1 Bridge SC-1

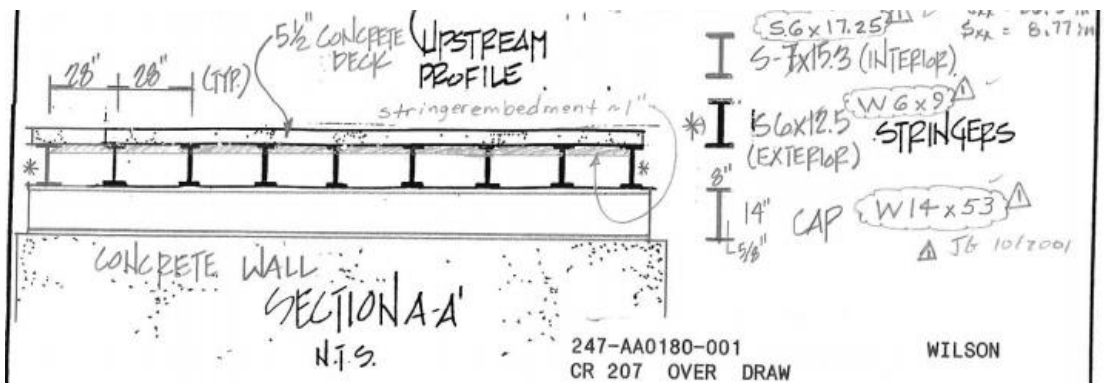
Bridge SC-1 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 5 (Fair) with 10 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 42 ft, consisting of two continuous spans. The controlling span for load rating is 19 ft and is 19 ft wide, with a roadway width of 19 ft. The bridge carries two lanes, one in each direction, and has an ADT of 40 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 13 US tons and an operating gross loading of 20 US tons. The bridge is posted for a 10,000-lbs axle or tandem axle.



Figure B.51. Bridge SC-1 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.52. Bridge Drawings and Photos (TxDOT 2018c)

Table B.26. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-1

Bridge Characteristics					
Year Built	1910				
On/Off-System	Off				
ADT	40				
Span Lengths	19' – 18'				
Deck Width	19'-0"				
Roadway Width	19'-0"				
Interior Girder Section	S6x17.25				
Exterior Girder Section	W6x9				
Interior Girder Spacing	2'-4"				
Slab Thickness (in.)	5.5				
Wearing Surface Thickness (in.)	0.0				
Composite	yes				
TxDOT Inventory RF	0.37				
TxDOT Operating RF	0.56				
TxDOT Inventory Rating (HS)	7.3				
TxDOT Operating Rating (HS)	11.2				
Posted Axle Limit (lbs)	10,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	16.5	C_{serv} (kip-ft)	30.3	C_{serv} (kip-ft)	18.7
C_{OP} (ksi)	22.5	C_n (kip-ft)	42.5	C_n (kip-ft)	23.6
f_{DL} (ksi)	7.4	M_{DL} (kip-ft)	7.8	M_{DL} (kip-ft)	7.8
$f_{LL-HS20}$ (ksi)	28.6	$M_{LL-HS20}$ (kip-ft)	30.4	$M_{LL-HL93}$ (kip-ft)	83.3
Inventory HS20 RF	0.32	Inventory HS20 RF	0.44	Inventory HL93 RF	0.08
Operating HS20 RF	0.53	Operating HS20 RF	0.74	Operating HL93 RF	0.11
Inventory Rating (HS)	6.4	Inventory Rating (HS)	8.8		
Operating Rating (HS)	10.6	Operating Rating (HS)	14.8		
$S_{x,negative}$ (in ³)	12.8	Serviceability Controls	yes	Serviceability Controls	yes

Notes:

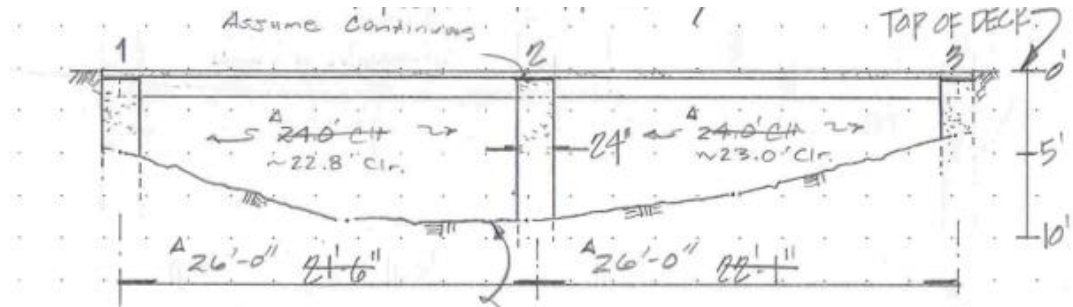
1. Steel yield strength taken as 30 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Negative moment region controls calculated ratings.

B.2.2 Bridge SC-2

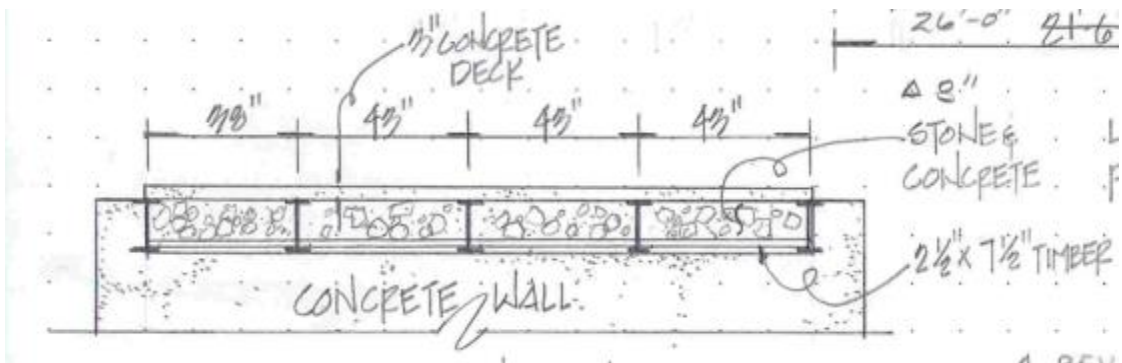
Bridge SC-2 has a deck condition rating of 5 (Fair), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 52 ft, consisting of two continuous spans. The controlling span for load rating is 26 ft and is 13 ft 9 in. wide, with a roadway width of 13 ft 9 in. The bridge carries one lane and has an ADT of 10 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 15 US tons and an operating gross loading of 26 US tons. The bridge is posted for a 12,500-lbs axle or tandem axle.



Figure B.53. Bridge SC-2 (TxDOT 2018c)



(a) Elevation View



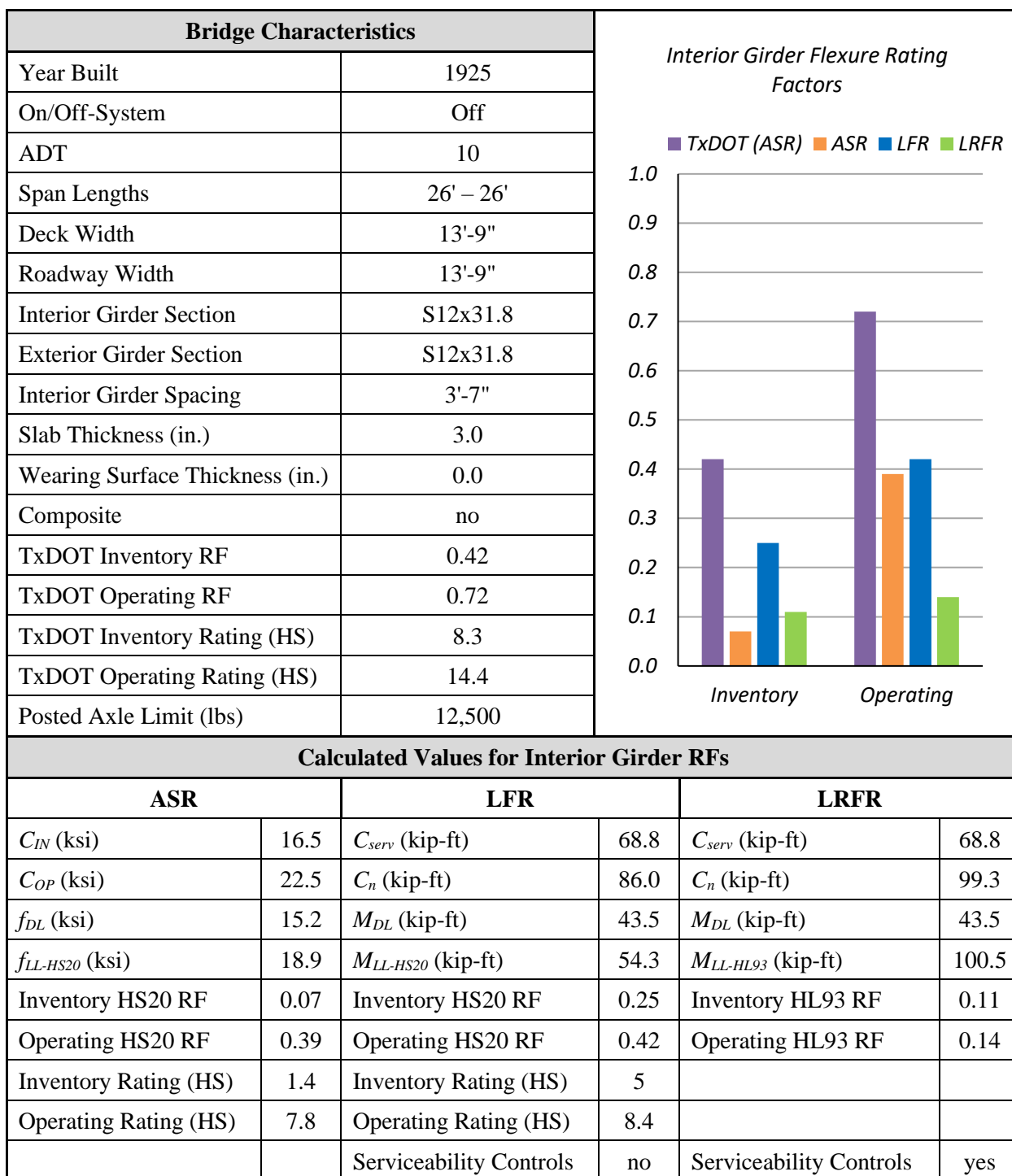
(b) Transverse Section



(c) Underside Superstructure View

Figure B.54. Bridge Drawings and Photos (TxDOT 2018c)

Table B.27. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-2



Notes:

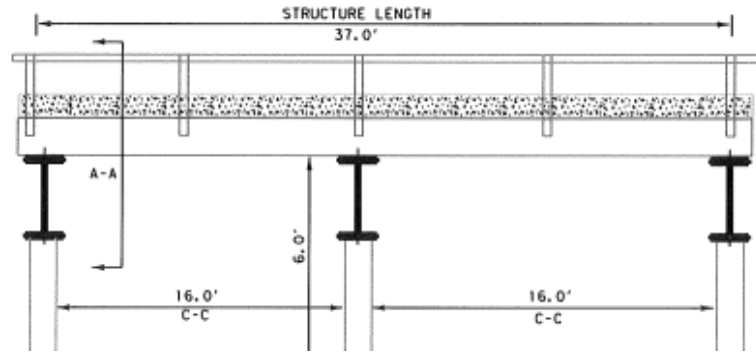
1. Steel yield strength taken as 30 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Negative moment region controls calculated ratings.

B.2.3 Bridge SC-3

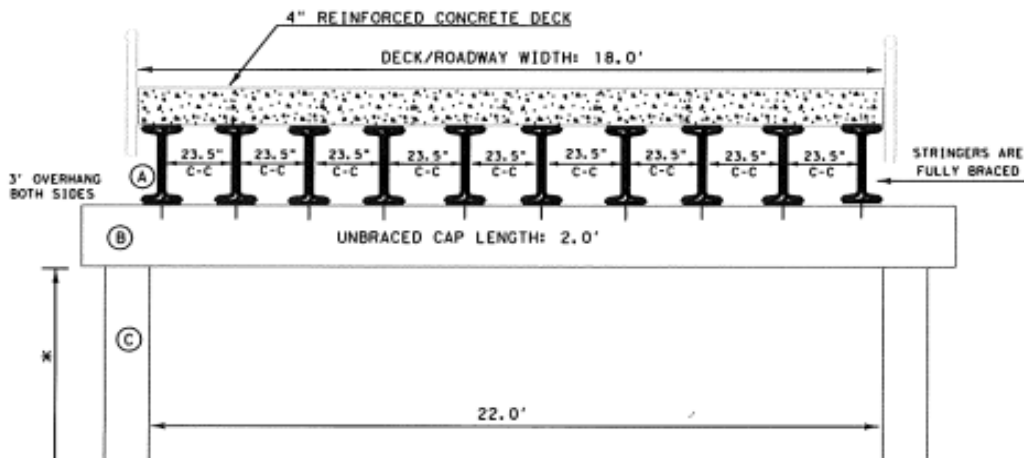
Bridge SC-3 has a deck condition rating of 5 (Fair), a superstructure condition rating of 6 (Satisfactory) with 10 percent beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 37 ft, consisting of two continuous spans. The controlling span for load rating is 18 ft and is 18 ft wide with a roadway width of 18 ft. The bridge carries two lanes, one in each direction, and has an ADT of 50 vehicles. The deck controls the rating of the bridge, which has an inventory gross loading of 23 US tons and an operating gross loading of 38 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure B.55. Bridge SC-3 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.56. Bridge Drawings and Photos (TxDOT 2018c)

Table B.28. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-3

Bridge Characteristics	
Year Built	1935
On/Off-System	Off
ADT	50
Span Lengths	18' – 18'
Deck Width	18'-0"
Roadway Width	18'-0"
Interior Girder Section	S10x25.4
Exterior Girder Section	S10x25.4
Interior Girder Spacing	1'-11.5"
Slab Thickness (in.)	4.0
Wearing Surface Thickness (in.)	5.0
Composite	no
TxDOT Inventory RF	1.00
TxDOT Operating RF	1.46
TxDOT Inventory Rating (HS)	19.9
TxDOT Operating Rating (HS)	29.1
Posted Axle Limit (lbs)	24,000

Rating Factor	Inventory	Operating
TxDOT (ASR)	1.00	1.46
ASR	1.20	1.80
LFR	1.10	1.90
LRFR	0.30	0.35

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	16.5	C_{serv} (kip-ft)	44.3	C_{serv} (kip-ft)	44.3
C_{OP} (ksi)	22.5	C_n (kip-ft)	55.4	C_n (kip-ft)	63.7
f_{DL} (ksi)	4.9	M_{DL} (kip-ft)	9.0	M_{DL} (kip-ft)	5.1
$f_{LL-HS20}$ (ksi)	9.6	$M_{LL-HS20}$ (kip-ft)	17.7	$M_{LL-HL93}$ (kip-ft)	106.0
Inventory HS20 RF	1.21	Inventory HS20 RF	1.13	Inventory HL93 RF	0.28
Operating HS20 RF	1.83	Operating HS20 RF	1.89	Operating HL93 RF	0.36
Inventory Rating (HS)	24.2	Inventory Rating (HS)	22.6		
Operating Rating (HS)	36.6	Operating Rating (HS)	37.8		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

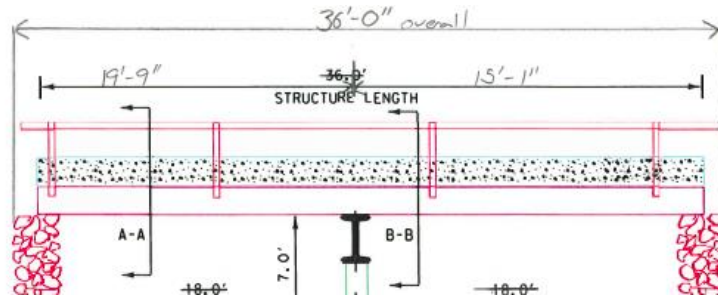
1. Steel yield strength taken as 30 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Negative moment region controls calculated ratings for ASR/LFR, positive controls for LRFR.

B.2.4 Bridge SC-4

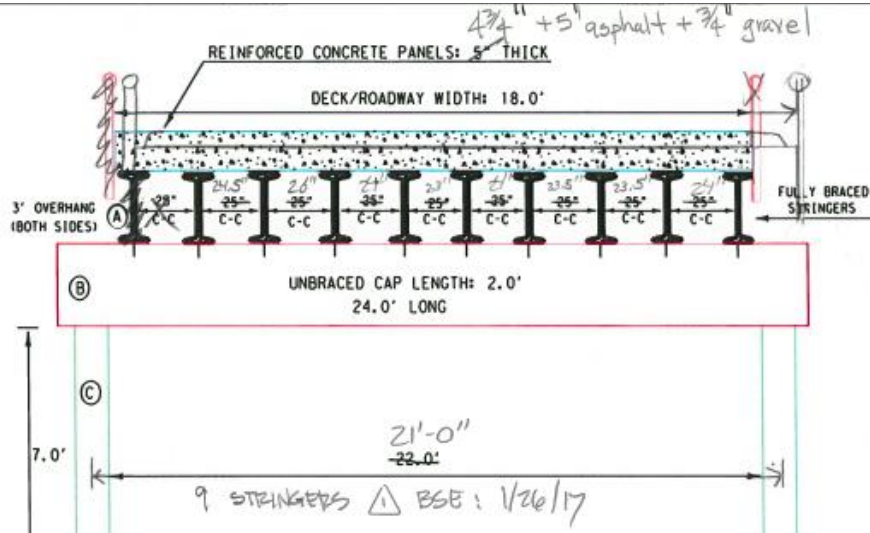
Bridge SC-4 has a deck condition rating of 5 (Fair), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 36 ft, consisting of two continuous spans. The controlling span for load rating is 19 ft 9 in. and is 18 ft wide, with a roadway width of 18 ft. The bridge carries two lanes, one in each direction, and has an ADT of 50 vehicles. The deck controls the rating of the bridge, which has an inventory gross loading of 21 US tons and an operating gross loading of 37 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure B.57. Bridge SC-4 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.58. Bridge Drawings and Photos (TxDOT 2018c)

Table B.29. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-4

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1937				
On/Off-System	Off				
ADT	50				
Span Lengths	18' – 15'				
Deck Width	18'-0"				
Roadway Width	18'-0"				
Interior Girder Section	S10x25.4				
Exterior Girder Section	S10x25.4				
Interior Girder Spacing	2'-2"				
Slab Thickness (in.)	4.75				
Wearing Surface Thickness (in.)	5.75				
Composite	no				
TxDOT Inventory RF	1.09				
TxDOT Operating RF	1.61				
TxDOT Inventory Rating (HS)	21.7				
TxDOT Operating Rating (HS)	32.2				
Posted Axle Limit (lbs)	24,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	51.4	C_{serv} (kip-ft)	51.4
C_{OP} (ksi)	24.75	C_n (kip-ft)	64.3	C_n (kip-ft)	73.9
f_{DL} (ksi)	3.4	M_{DL} (kip-ft)	6.6	M_{DL} (kip-ft)	6.6
$f_{LL-HS20}$ (ksi)	12.9	$M_{LL-HS20}$ (kip-ft)	25.1	$M_{LL-HL93}$ (kip-ft)	137.1
Inventory HS20 RF	1.12	Inventory HS20 RF	1.02	Inventory HL93 RF	0.24
Operating HS20 RF	1.66	Operating HS20 RF	1.71	Operating HL93 RF	0.32
Inventory Rating (HS)	22.4	Inventory Rating (HS)	20.4		
Operating Rating (HS)	33.2	Operating Rating (HS)	34.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

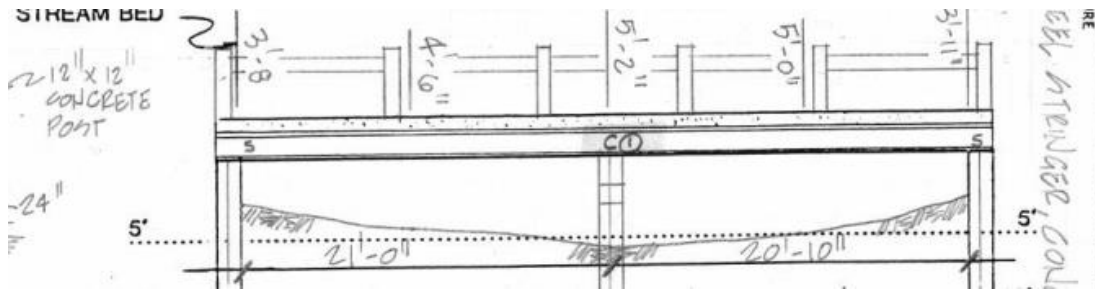
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.

B.2.5 Bridge SC-5

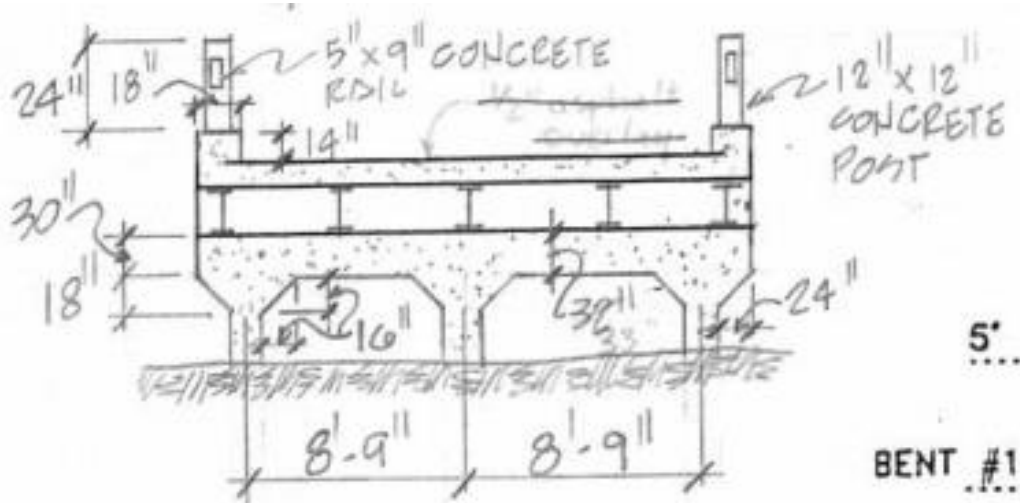
Bridge SC-5 has a deck condition rating of 5 (Fair), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 43 ft 10 in., consisting of two continuous spans. The controlling span for load rating is 21 ft and is 23 ft 4 in., wide with a roadway width of 20 ft 5 in. The bridge carries two lanes, one in each direction, and has an ADT of 50 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 21 US tons and an operating gross loading of 32 US tons. The bridge is posted for a 17,500-lbs axle or tandem axle.



Figure B.59. Bridge SC-5 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.60. Bridge Drawings and Photos (TxDOT 2018c)

Table B.30. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-5

Bridge Characteristics	
Year Built	1940
On/Off-System	Off
ADT	50
Span Lengths	21' – 21'
Deck Width	23'-4"
Roadway Width	20'-5"
Interior Girder Section	S10x25.4
Exterior Girder Section	S10x25.4
Interior Girder Spacing	5'-6.5"
Slab Thickness (in.)	6.25
Wearing Surface Thickness (in.)	0.0
Composite	yes
TxDOT Inventory RF	0.58
TxDOT Operating RF	0.89
TxDOT Inventory Rating (HS)	11.6
TxDOT Operating Rating (HS)	17.8
Posted Axle Limit (lbs)	17,500

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.58	0.17	0.34	0.13
Operating	0.89	0.37	0.57	0.16

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	61.1	C_{serv} (kip-ft)	61.1
C_{OP} (ksi)	24.75	C_n (kip-ft)	105.4	C_n (kip-ft)	73.9
f_{DL} (ksi)	12.8	M_{DL} (kip-ft)	32.7	M_{DL} (kip-ft)	32.7
$f_{LL-HS20}$ (ksi)	32.3	$M_{LL-HS20}$ (kip-ft)	82.5	$M_{LL-HL93}$ (kip-ft)	124.5
Inventory HS20 RF	0.17	Inventory HS20 RF	0.34	Inventory HL93 RF	0.13
Operating HS20 RF	0.37	Operating HS20 RF	0.57	Operating HL93 RF	0.16
Inventory Rating (HS)	3.4	Inventory Rating (HS)	6.8		
Operating Rating (HS)	7.4	Operating Rating (HS)	11.4		
$S_{x,negative}$ (in ³)	30.6	Serviceability Controls	yes	Serviceability Controls	yes

Notes:

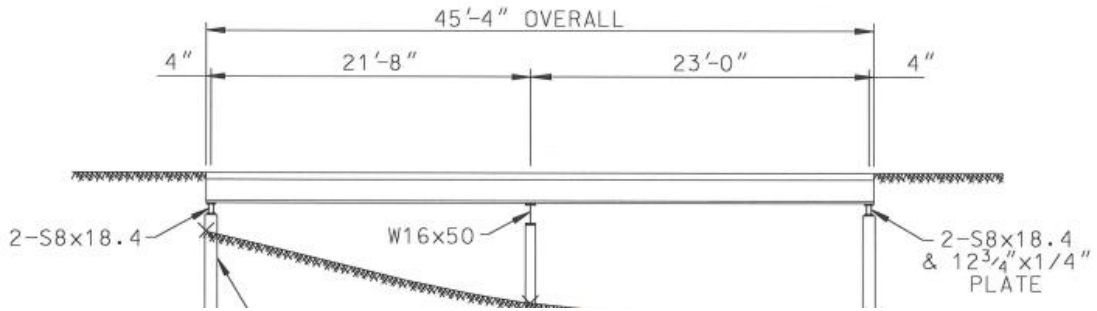
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Negative moment region controls calculated ratings.

B.2.6 Bridge SC-6

Bridge SC-6 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 45 ft 4 in., consisting of two continuous spans. The controlling span for load rating is 23 ft and is 23 ft 6 in. wide, with a roadway width of 23 ft 6 in. The bridge carries two lanes, one in each direction, and has an ADT of 350 vehicles. The cap controls the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.

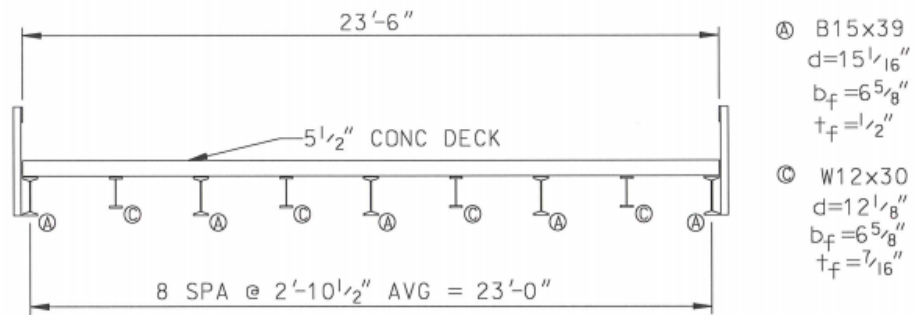


Figure B.61. Bridge SC-6 (TxDOT 2018c)



(a) Elevation View

NOTE: UNBRACED LENGTH = 15.0'



(b) Transverse Section



(c) Underside Superstructure View

Figure B.62. Bridge Drawings and Photos (TxDOT 2018c)

Table B.31. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-6

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1942				
On/Off-System	Off				
ADT	350				
Span Lengths	23' – 22'				
Deck Width	23'-6"				
Roadway Width	23'-6"				
Interior Girder Section	S12x31.8				
Exterior Girder Section	B15x39				
Interior Girder Spacing	2'-10.5"				
Slab Thickness (in.)	5.5				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	1.08				
TxDOT Operating RF	1.53				
TxDOT Inventory Rating (HS)	21.5				
TxDOT Operating Rating (HS)	30.6				
Posted Axle Limit (lbs)	24,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	100.1	C_{serv} (kip-ft)	100.1
C_{OP} (ksi)	24.75	C_n (kip-ft)	125.1	C_n (kip-ft)	141.3
f_{DL} (ksi)	2.3	M_{DL} (kip-ft)	8.9	M_{DL} (kip-ft)	8.9
$f_{LL-HS20}$ (ksi)	13.4	$M_{LL-HS20}$ (kip-ft)	50.9	$M_{LL-HL93}$ (kip-ft)	115.0
Inventory HS20 RF	1.18	Inventory HS20 RF	1.03	Inventory HL93 RF	0.60
Operating HS20 RF	1.67	Operating HS20 RF	1.72	Operating HL93 RF	0.77
Inventory Rating (HS)	23.6	Inventory Rating (HS)	20.6		
Operating Rating (HS)	33.4	Operating Rating (HS)	34.4		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

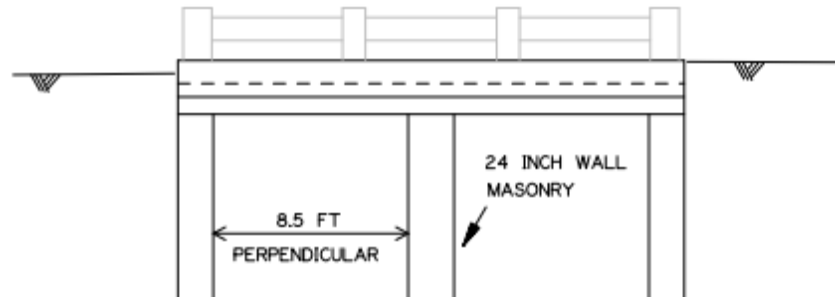
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.
3. Posting has been removed per inspection dated 12/27/2017.

B.2.7 Bridge SC-7

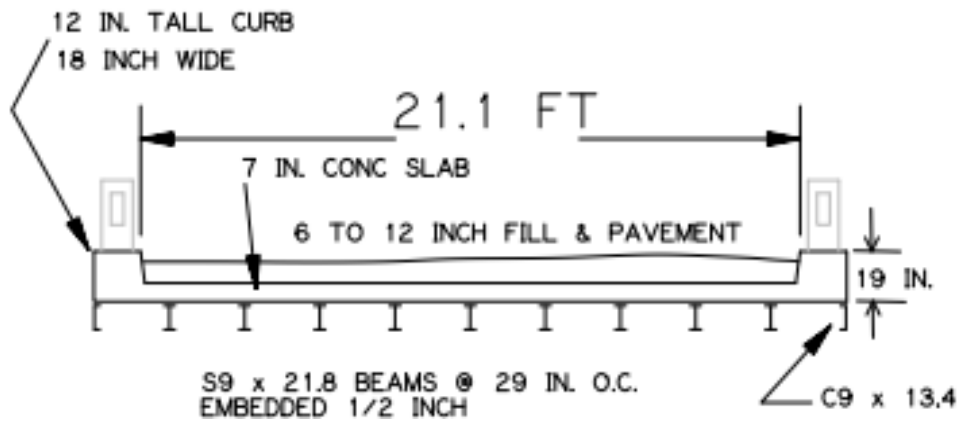
Bridge SC-7 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 26 ft, consisting of two continuous spans. The controlling span for load rating is 11 ft 9 in. and is 24 ft 2 in. wide, with a roadway width of 21 ft 2 in. The bridge carries two lanes, one in each direction, and has an ADT of 180 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure B.63. Bridge SC-7 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.64. Bridge Drawings and Photos (TxDOT 2018c)

Table B.32. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-7

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1943				
On/Off-System	Off				
ADT	180				
Span Lengths	12' – 11'				
Deck Width	24'-2"				
Roadway Width	21'-2"				
Interior Girder Section	S9x21.8				
Exterior Girder Section	C9x13.4				
Interior Girder Spacing	2'-5"				
Slab Thickness (in.)	7.0				
Wearing Surface Thickness (in.)	7.0				
Composite	no				
TxDOT Inventory RF	0.84				
TxDOT Operating RF	1.23				
TxDOT Inventory Rating (HS)	16.7				
TxDOT Operating Rating (HS)	24.6				
Posted Axle Limit (lbs)	24,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	16.5	C_{serv} (kip-ft)	35.9	C_{serv} (kip-ft)	35.9
C_{OP} (ksi)	22.5	C_n (kip-ft)	44.9	C_n (kip-ft)	44.9
f_{DL} (ksi)	3.0	M_{DL} (kip-ft)	4.6	M_{DL} (kip-ft)	4.6
$f_{LL-HS20}$ (ksi)	14.2	$M_{LL-HS20}$ (kip-ft)	21.3	$M_{LL-HL93}$ (kip-ft)	57.5
Inventory HS20 RF	0.95	Inventory HS20 RF	0.84	Inventory HL93 RF	0.40
Operating HS20 RF	1.37	Operating HS20 RF	1.41	Operating HL93 RF	0.53
Inventory Rating (HS)	19.0	Inventory Rating (HS)	16.8		
Operating Rating (HS)	27.4	Operating Rating (HS)	28.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

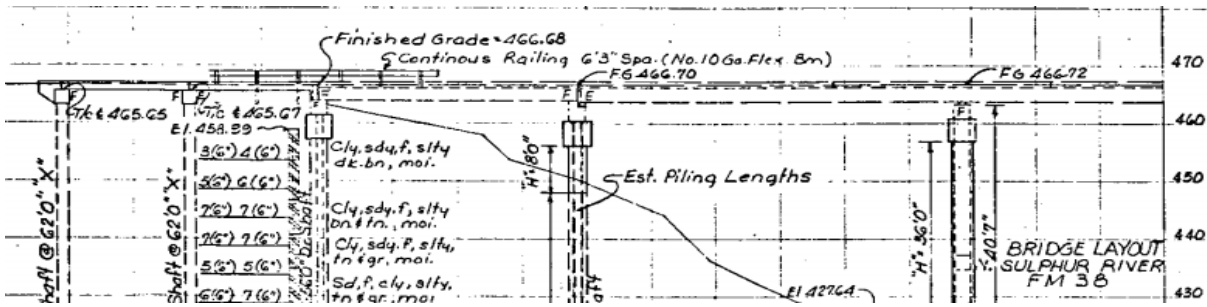
1. Steel yield strength taken as 30 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.

B.2.8 Bridge SC-8

Bridge SC-8 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 3 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 349 ft, which consists of multiple simply supported spans and three continuous spans. The controlling span for load rating is 59 ft 8 in. and is 25 ft 8 in. wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 230 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 20 US tons and an operating gross loading of 33 US tons. The bridge is posted for 20,000-lbs single axle, 29,000-lbs tandem axle, 58,000-lbs single vehicle, or 69,000-lbs combination vehicle.



Figure B.65. Bridge SC-8 (TxDOT 2018c)



(a) Elevation View



(b) Roadway View—No Cross-Section plans available for the steel continuous section of this bridge



(c) Underside Superstructure View

Figure B.66. Bridge Drawings and Photos (TxDOT 2018c)

Table B.33. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-8

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1949				
On/Off-System	On				
ADT	230				
Span Lengths	60' – 70' – 60'				
Deck Width	25'-8"				
Roadway Width	24'-0"				
Interior Girder Section	30WFx108				
Exterior Girder Section	30WFx108				
Interior Girder Spacing	7'-0"				
Slab Thickness (in.)	12.0				
Wearing Surface Thickness (in.)	1.0				
Composite	no				
TxDOT Inventory RF	0.56				
TxDOT Operating RF	0.92				
TxDOT Inventory Rating (HS)	11.1				
TxDOT Operating Rating (HS)	18.3				
Posted Axle Limit (lbs)	see note 3				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	638.5	C_{serv} (kip-ft)	638.5
C_{OP} (ksi)	24.75	C_n (kip-ft)	798.1	C_n (kip-ft)	910.3
f_{DL} (ksi)	10.8	M_{DL} (kip-ft)	261.4	M_{DL} (kip-ft)	326.8
$f_{LL-HS20}$ (ksi)	17.4	$M_{LL-HS20}$ (kip-ft)	419.7	$M_{LL-HL93}$ (kip-ft)	563.9
Inventory HS20 RF	0.42	Inventory HS20 RF	0.50	Inventory HL93 RF	0.31
Operating HS20 RF	0.80	Operating HS20 RF	0.84	Operating HL93 RF	0.41
Inventory Rating (HS)	8.4	Inventory Rating (HS)	10		
Operating Rating (HS)	16.0	Operating Rating (HS)	16.8		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

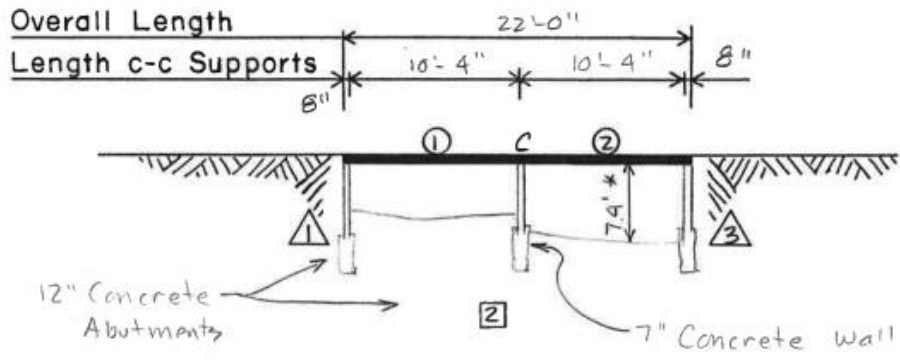
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings for ASR/LFR, negative controls for LRFR.
3. Load restrictions include a 20,000-lbs single axle, a 29,000-lbs tandem axle, a 58,000-lbs single vehicle, and a 69,000-lbs combination vehicle.

B.2.9 Bridge SC-9

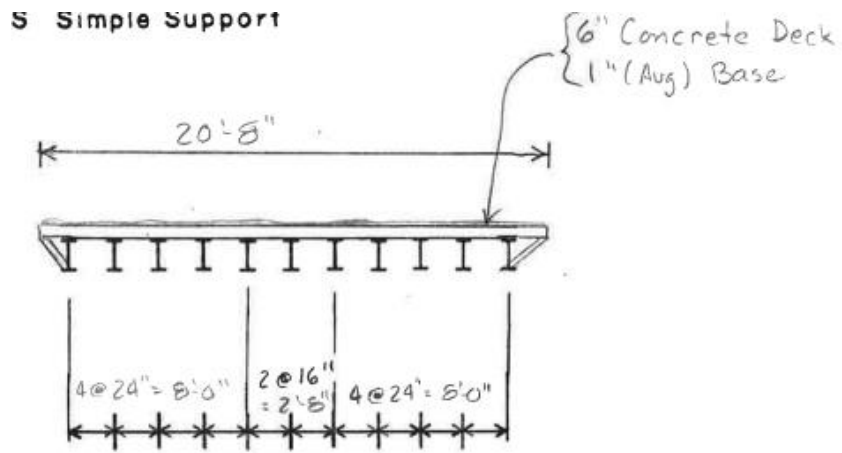
Bridge SC-9 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 22 ft, consisting of two continuous spans. The controlling span for load rating is 10 ft 4 in. and is 20 ft 8 in. wide, with a roadway width of 20 ft 8 in. The bridge carries two lanes, one in each direction, and has an ADT of 60 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 20 US tons and an operating gross loading of 33 US tons. The bridge is posted for a 17,500-lbs axle or tandem axle.



Figure B.67. Bridge SC-9 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.68. Bridge Drawings and Photos (TxDOT 2018c)

Table B.34. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-9

Bridge Characteristics					
Year Built	1950				
On/Off-System	Off				
ADT	60				
Span Lengths	11' – 11'				
Deck Width	20'-8"				
Roadway Width	20'-8"				
Interior Girder Section	S6x12.5				
Exterior Girder Section	S6x12.5				
Interior Girder Spacing	2'-0"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.56				
TxDOT Operating RF	0.93				
TxDOT Inventory Rating (HS)	11.1				
TxDOT Operating Rating (HS)	18.5				
Posted Axle Limit (lbs)	17,500				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	15.3	C_{serv} (kip-ft)	15.3
C_{OP} (ksi)	24.75	C_n (kip-ft)	19.2	C_n (kip-ft)	22.1
f_{DL} (ksi)	2.2	M_{DL} (kip-ft)	1.3	M_{DL} (kip-ft)	1.3
$f_{LL-HS20}$ (ksi)	27.5	$M_{LL-HS20}$ (kip-ft)	16.0	$M_{LL-HL93}$ (kip-ft)	47.9
Inventory HS20 RF	0.58	Inventory HS20 RF	0.50	Inventory HL93 RF	0.22
Operating HS20 RF	0.82	Operating HS20 RF	0.84	Operating HL93 RF	0.29
Inventory Rating (HS)	11.6	Inventory Rating (HS)	10		
Operating Rating (HS)	16.4	Operating Rating (HS)	16.8		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

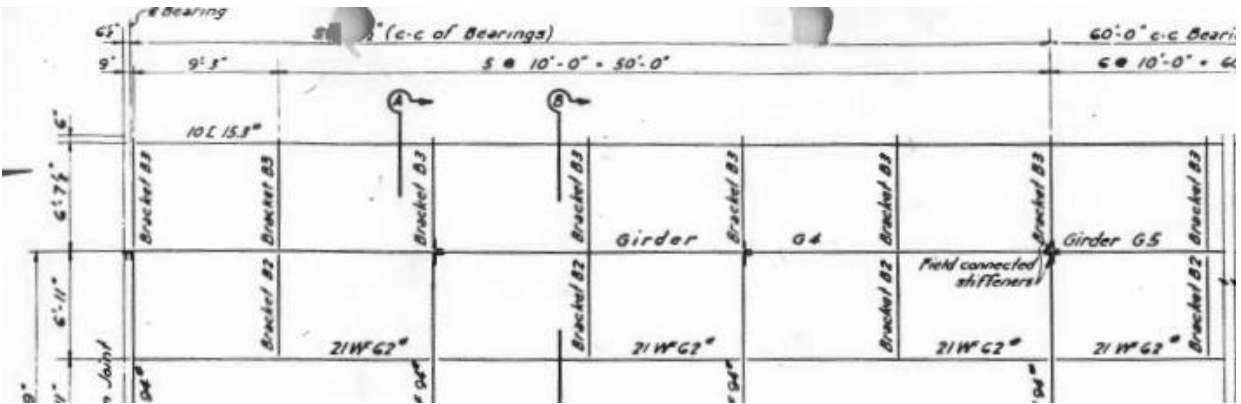
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.

B.2.10 Bridge SC-10

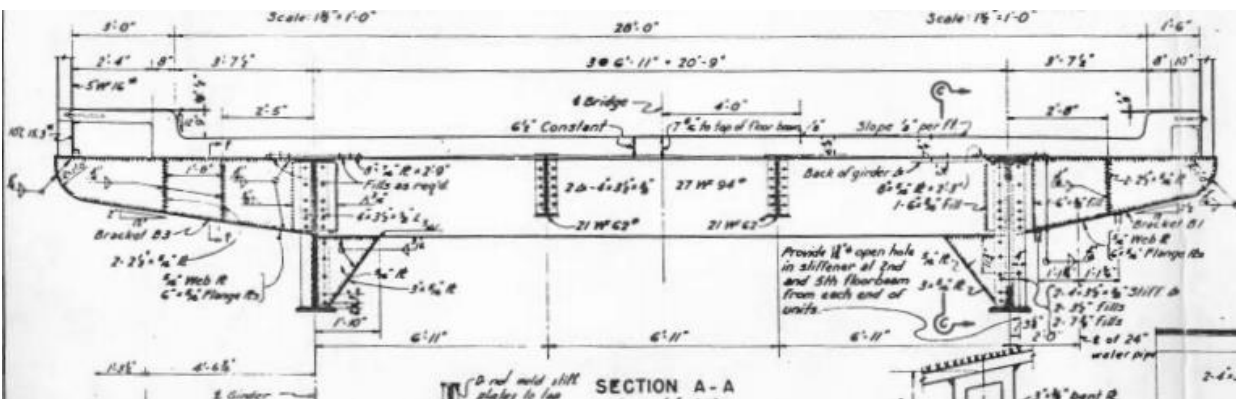
Bridge SC-10 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 5 (Fair) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 2,723 ft, consisting of 42 continuous spans. The controlling span for load rating is 20 ft 9 in. and is 34 ft 6 in. wide, with a roadway width of 32 ft 6 in. The bridge carries two lanes, both in the same direction, and has an ADT of 7300 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 21 US tons and an operating gross loading of 34 US tons. The bridge is posted for 18,000-lbs axle or 23,000-lbs tandem axle.



Figure B.69. Bridge SC-10 (TxDOT 2018c)



(a) Plan View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.70. Bridge Drawings and Photos (TxDOT 2018c)

Table B.35. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-10

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1950				
On/Off-System	On				
ADT	7300				
Span Lengths	20'-9"				
Deck Width	10'-0"				
Roadway Width	-				
Interior Girder Section	27WFx94				
Exterior Girder Section	-				
Interior Girder Spacing	10'-0"				
Slab Thickness (in.)	7.5				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.58				
TxDOT Operating RF	0.96				
TxDOT Inventory Rating (HS)	11.6				
TxDOT Operating Rating (HS)	19.2				
Posted Axle Limit (lbs)	18,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	534.6	C_{serv} (kip-ft)	534.6
C_{OP} (ksi)	24.75	C_n (kip-ft)	764.5	C_n (kip-ft)	756.2
f_{DL} (ksi)	2.6	M_{DL} (kip-ft)	53.1	M_{DL} (kip-ft)	53.1
$f_{LL-HS20}$ (ksi)	11.2	$M_{LL-HS20}$ (kip-ft)	227.5	$M_{LL-HL93}$ (kip-ft)	261.0
Inventory HS20 RF	1.38	Inventory HS20 RF	1.27	Inventory HL93 RF	1.42
Operating HS20 RF	1.97	Operating HS20 RF	2.12	Operating HL93 RF	1.84
Inventory Rating (HS)	27.6	Inventory Rating (HS)	25.4		
Operating Rating (HS)	39.4	Operating Rating (HS)	42.4		
		Serviceability Controls	yes	Serviceability Controls	yes

Notes:

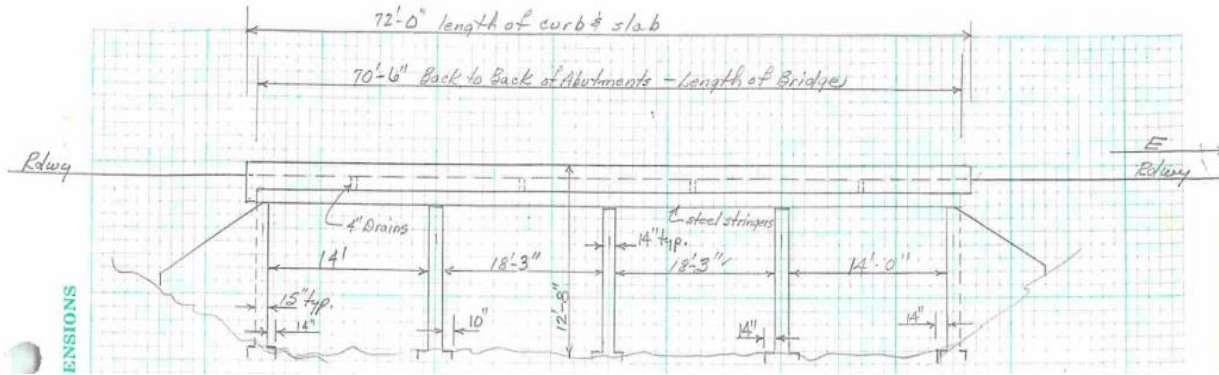
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.
3. Ratings are for the floor beam.

B.2.11 Bridge SC-11

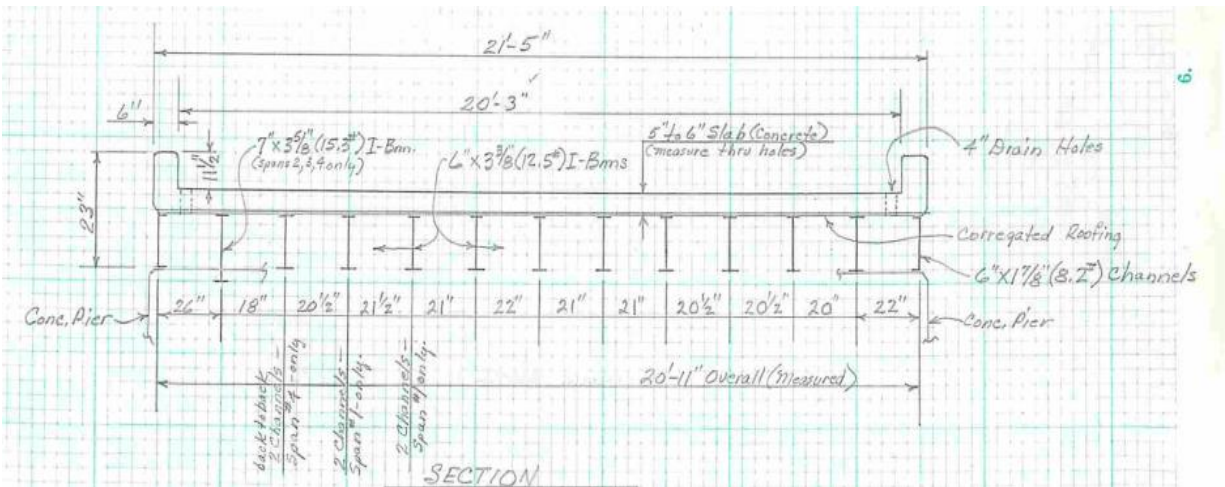
Bridge SC-11 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 5 (Fair) with 5 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 70 ft 6 in., consisting of four continuous spans. The controlling span for load rating is 19 ft 4 in. and is 21 ft 5 in. wide, with a roadway width of 20 ft 3 in. The bridge carries one lane and has an ADT of 60 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 14 US tons and an operating gross loading of 22 US tons. The bridge is posted for a 10,000-lbs axle or tandem axle.



Figure B.71. Bridge SC-11 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.72. Bridge Drawings and Photos (TxDOT 2018c)

Table B.36. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-11

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1955				
On/Off-System	Off				
ADT	60				
Span Lengths	15' – 19' – 19'				
Deck Width	21'-5"				
Roadway Width	20'-3"				
Interior Girder Section	S6x12.5				
Exterior Girder Section	C6x8.2				
Interior Girder Spacing	1'-8"				
Slab Thickness (in.)	5.5				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.40				
TxDOT Operating RF	0.61				
TxDOT Inventory Rating (HS)	7.9				
TxDOT Operating Rating (HS)	12.1				
Posted Axle Limit (lbs)	10,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	15.3	C_{serv} (kip-ft)	15.3
C_{OP} (ksi)	24.75	C_n (kip-ft)	19.2	C_n (kip-ft)	22.1
f_{DL} (ksi)	11.4	M_{DL} (kip-ft)	6.6	M_{DL} (kip-ft)	6.6
$f_{LL-HS20}$ (ksi)	27.3	$M_{LL-HS20}$ (kip-ft)	15.8	$M_{LL-HL93}$ (kip-ft)	88.7
Inventory HS20 RF	0.25	Inventory HS20 RF	0.31	Inventory HL93 RF	0.06
Operating HS20 RF	0.49	Operating HS20 RF	0.51	Operating HL93 RF	0.08
Inventory Rating (HS)	5.0	Inventory Rating (HS)	6.2		
Operating Rating (HS)	9.8	Operating Rating (HS)	10.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

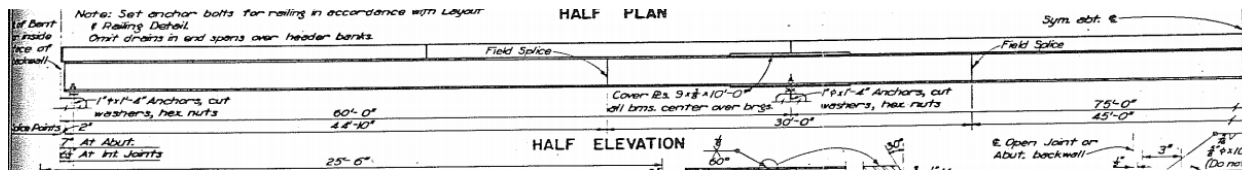
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Negative moment region controls calculated ratings.

B.2.12 Bridge SC-12

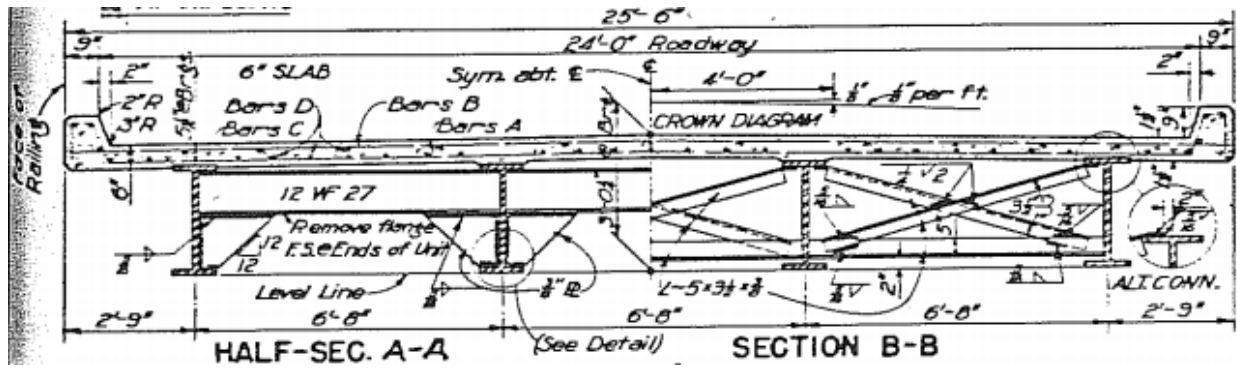
Bridge SC-12 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 7 (Good) without beam section loss due to corrosion, and a substructure condition rating of 7 (Good). The bridge has a total length of 195 ft, consisting of three continuous spans. The controlling span for load rating is 75 ft and is 25 ft 6 in. wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 260 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 19 US tons and an operating gross loading of 32 US tons. The bridge is posted for 20,000-lbs single axle, 34,000-lbs tandem axle, 58,000-lbs single vehicle or 75,000-lbs combination vehicle.



Figure B.73. Bridge SC-12 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.74. Bridge Drawings and Photos (TxDOT 2018c)

Table B.37. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-12

Bridge Characteristics	
Year Built	1959
On/Off-System	On
ADT	260
Span Lengths	60' – 75' – 60'
Deck Width	25'-6"
Roadway Width	24'-0"
Interior Girder Section	30WFx108
Exterior Girder Section	30WFx108
Interior Girder Spacing	6'-8"
Slab Thickness (in.)	6.0
Wearing Surface Thickness (in.)	2.0
Composite	no
TxDOT Inventory RF	0.55
TxDOT Operating RF	0.93
TxDOT Inventory Rating (HS)	11.0
TxDOT Operating Rating (HS)	18.5
Posted Axle Limit (lbs)	see note 3

Condition	TxDOT (LFR)	ASR	LFR	LRFr
Inventory	0.55	0.71	0.64	0.66
Operating	0.93	1.04	1.07	0.86

Calculated Values for Interior Girder RFs					
ASR		LFR		LRFr	
C_{IN} (ksi)	18.15	C_{serv} (kip-ft)	658.2	C_{serv} (kip-ft)	658.2
C_{OP} (ksi)	24.75	C_n (kip-ft)	822.8	C_n (kip-ft)	938.5
f_{DL} (ksi)	3.8	M_{DL} (kip-ft)	95.4	M_{DL} (kip-ft)	95.4
$f_{LL-HS20}$ (ksi)	20.2	$M_{LL-HS20}$ (kip-ft)	503.1	$M_{LL-HL93}$ (kip-ft)	625.8
Inventory HS20 RF	0.71	Inventory HS20 RF	0.64	Inventory HL93 RF	0.66
Operating HS20 RF	1.04	Operating HS20 RF	1.07	Operating HL93 RF	0.86
Inventory Rating (HS)	14.2	Inventory Rating (HS)	12.8		
Operating Rating (HS)	20.8	Operating Rating (HS)	21.4		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

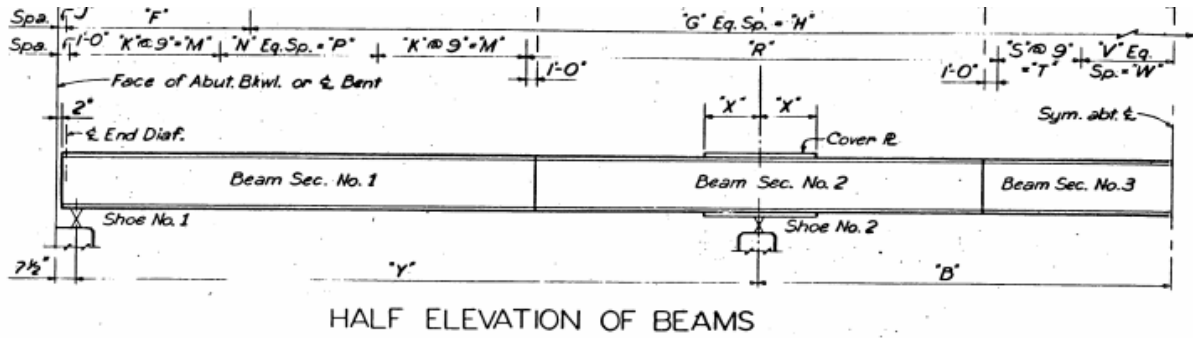
1. Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.
3. Load restrictions include a 20,000-lbs single axle, a 34,000-lbs tandem axle, a 58,000-lbs single vehicle, and a 75,000-lbs combination vehicle.

B.2.13 Bridge SC-13

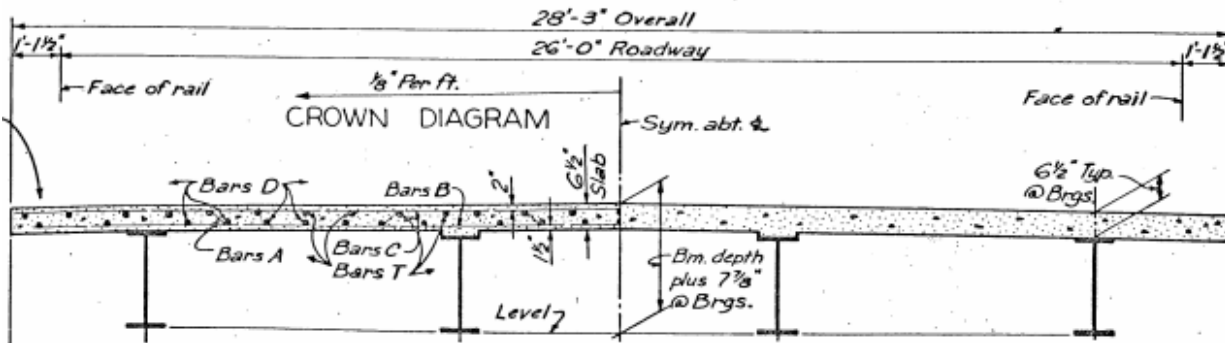
Bridge SC-13 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory) without beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 355 ft, which is multiple simply supported spans and three continuous spans. The controlling span for load rating is 75 ft and is 28 ft 3 in. wide, with a roadway width of 26 ft. The bridge carries two lanes, one in each direction, and has an ADT of 1160 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 26 US tons and an operating gross loading of 44 US tons. The bridge is posted for a 20,000-lbs tandem axle.



Figure B.75. Bridge SC-13 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.76. Bridge Drawings and Photos (TxDOT 2018c)

Table B.38. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-13

Bridge Characteristics					
Year Built	1965	<p style="text-align: center;"><i>Interior Girder Flexure Rating Factors</i></p> <p style="text-align: center;"> ■ TxDOT (LFR) ■ ASR ■ LFR ■ LRF </p>			
On/Off-System	On				
ADT	1160				
Span Lengths	60' – 75' – 60'				
Deck Width	28'-3"				
Roadway Width	26'-0"				
Interior Girder Section	W30x108				
Exterior Girder Section	W30x108				
Interior Girder Spacing	7'-0"				
Slab Thickness (in.)	6.5				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.73				
TxDOT Operating RF	1.22				
TxDOT Inventory Rating (HS)	14.6				
TxDOT Operating Rating (HS)	24.3				
Posted Axle Limit (lbs)	20,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRF	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	717.6	C_{serv} (kip-ft)	717.6
C_{OP} (ksi)	27	C_n (kip-ft)	897.0	C_n (kip-ft)	1038.0
f_{DL} (ksi)	6.3	M_{DL} (kip-ft)	157.9	M_{DL} (kip-ft)	157.9
$f_{LL-HS20}$ (ksi)	21.2	$M_{LL-HS20}$ (kip-ft)	528.2	$M_{LL-HL93}$ (kip-ft)	647.5
Inventory HS20 RF	0.64	Inventory HS20 RF	0.60	Inventory HL93 RF	0.74
Operating HS20 RF	0.98	Operating HS20 RF	1.01	Operating HL93 RF	0.96
Inventory Rating (HS)	12.8	Inventory Rating (HS)	12		
Operating Rating (HS)	19.6	Operating Rating (HS)	20.2		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

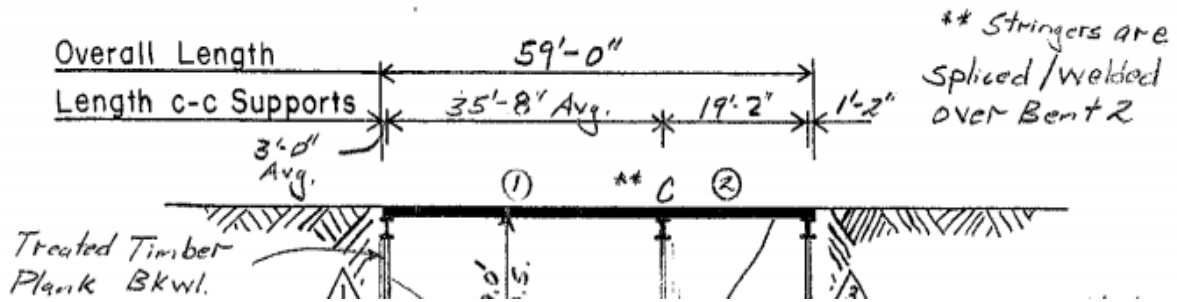
1. Steel yield strength taken as 30 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.

B.2.14 Bridge SC-14

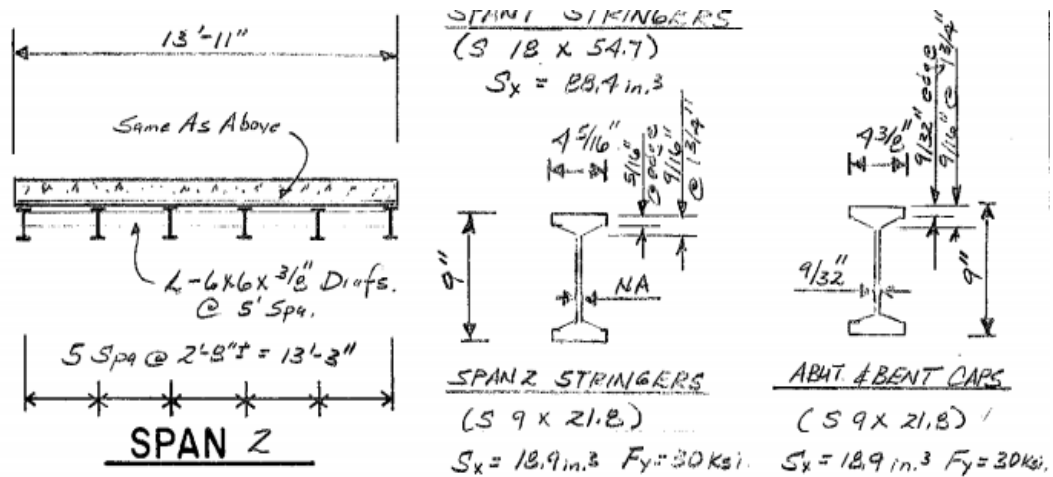
Bridge SC-14 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 5 (Fair). The bridge has a total length of 59 ft, consisting of two continuous spans. The controlling span for load rating is 35 ft 8 in. and is 13 ft 11 in. wide, with a roadway width of 13 ft 11 in. The bridge carries one lane and has an ADT of 80 vehicles. The girders and caps control the rating of the bridge, which has an inventory gross loading of 14 US tons and an operating gross loading of 22 US tons. The bridge is posted for a 10,000-lbs axle or tandem axle.



Figure B.77. Bridge SC-14 (TxDOT 2018c)



(a) Elevation View



5

(b) Transverse Section



(c) Underside Superstructure View

Figure B.78. Bridge Drawings and Photos (TxDOT 2018c)

Table B.39. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-14

Bridge Characteristics		Interior Girder Flexure Rating Factors			
Year Built	1970				
On/Off-System	Off				
ADT	80				
Span Lengths	36' – 19'				
Deck Width	13'-11"				
Roadway Width	13'-11"				
Interior Girder Section	S9x21.8				
Exterior Girder Section	S9x21.8				
Interior Girder Spacing	2'-8"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.48				
TxDOT Operating RF	0.73				
TxDOT Inventory Rating (HS)	9.5				
TxDOT Operating Rating (HS)	14.6				
Posted Axle Limit (lbs)	10,000				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	16.5	C_{serv} (kip-ft)	37.0	C_{serv} (kip-ft)	37.0
C_{OP} (ksi)	22.5	C_n (kip-ft)	53.2	C_n (kip-ft)	53.2
f_{DL} (ksi)	6.3	M_{DL} (kip-ft)	9.7	M_{DL} (kip-ft)	9.7
$f_{LL-HS20}$ (ksi)	21.8	$M_{LL-HS20}$ (kip-ft)	33.7	$M_{LL-HL93}$ (kip-ft)	121.5
Inventory HS20 RF	0.47	Inventory HS20 RF	0.49	Inventory HL93 RF	0.16
Operating HS20 RF	0.74	Operating HS20 RF	0.81	Operating HL93 RF	0.21
Inventory Rating (HS)	9.4	Inventory Rating (HS)	9.8		
Operating Rating (HS)	14.8	Operating Rating (HS)	16.2		
		Serviceability Controls	yes	Serviceability Controls	yes

Notes:

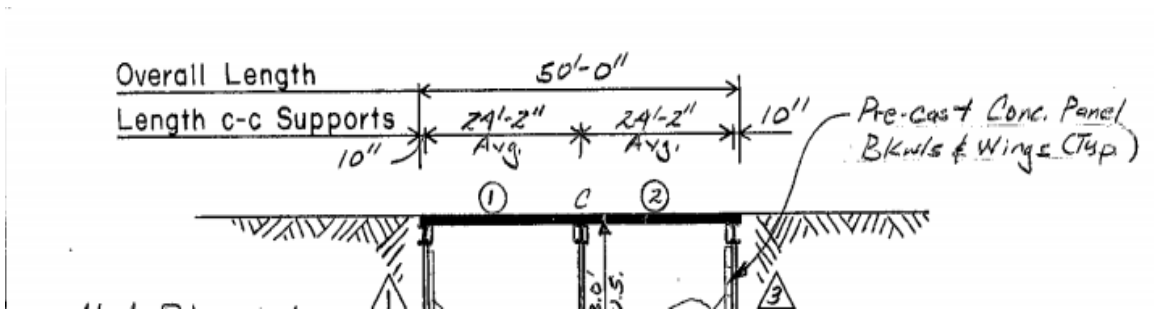
1. Steel yield strength taken as 33 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Positive moment region controls calculated ratings.

B.2.15 Bridge SC-15

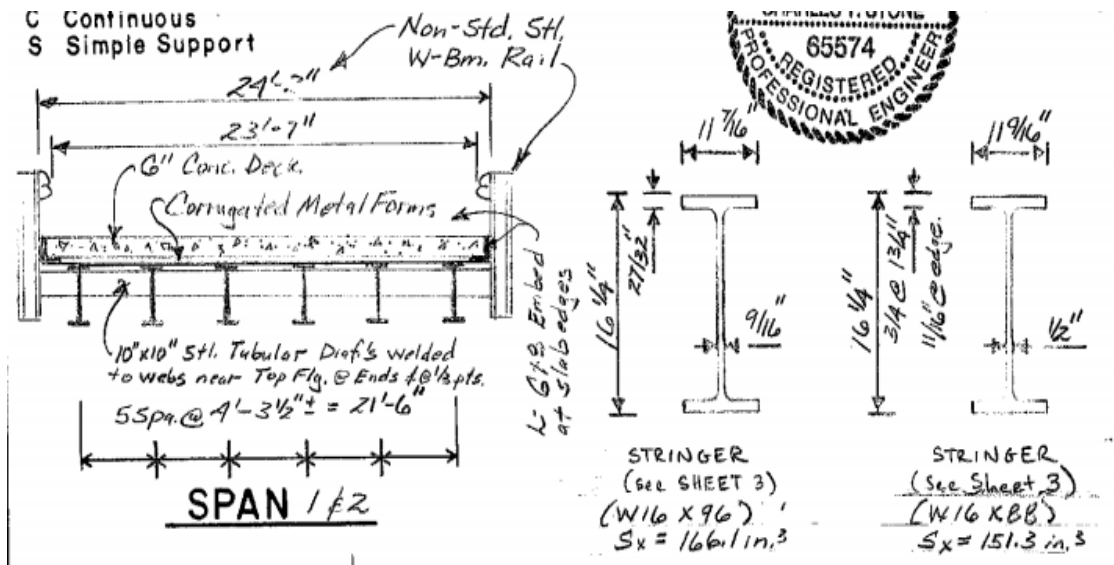
Bridge SC-15 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good) without beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 50 ft, consisting of two continuous spans. The controlling span for load rating is 24 ft 2 in. and is 24 ft 2 in. wide, with a roadway width of 23 ft 7 in. The bridge carries two lanes, one in each direction, and has an ADT of 60 vehicles. The steel cap controls the rating of the bridge, which has an inventory gross loading of 20 US tons and an operating gross loading of 33 US tons. The bridge is posted for a 21,000-lbs tandem axle and 40,000-lbs gross vehicle.



Figure B.79. Bridge SC-15 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.80. Bridge Drawings and Photos (TxDOT 2018c)

Table B.40. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-15

Bridge Characteristics					
Year Built	Muenster	<p style="text-align: center;"><i>Interior Stringer Flexure Rating Factors</i></p> <p style="text-align: center;"> ■ TxDOT (ASR) ■ ASR ■ LFR ■ LRFR </p>			
On/Off-System	Off				
ADT	60				
Span Lengths	24' – 24'				
Deck Width	24'-2"				
Roadway Width	23'-7"				
Interior Girder Section	W16x88				
Exterior Girder Section	W16x96				
Interior Girder Spacing	4'-3.5"				
Slab Thickness (in.)	6.0				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.56				
TxDOT Operating RF	0.94				
TxDOT Inventory Rating (HS)	11.1				
TxDOT Operating Rating (HS)	18.8				
Posted Axle Limit (lbs)	see note 2				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	363.1	C_{serv} (kip-ft)	363.1
C_{OP} (ksi)	27.0	C_n (kip-ft)	501.7	C_n (kip-ft)	501.7
f_{DL} (ksi)	10.8	M_{DL} (kip-ft)	136.3	M_{DL} (kip-ft)	136.3
$f_{LL-HS20}$ (ksi)	23.6	$M_{LL-HS20}$ (kip-ft)	297.7	$M_{LL-HL93}$ (kip-ft)	450.9
Inventory HS20 RF	0.38	Inventory HS20 RF	0.46	Inventory HL93 RF	0.33
Operating HS20 RF	0.69	Operating HS20 RF	0.76	Operating HL93 RF	0.43
Inventory Rating (HS)	7.6	Inventory Rating (HS)	9.2		
Operating Rating (HS)	13.8	Operating Rating (HS)	15.2		
		Serviceability Controls	yes	Serviceability Controls	yes

Notes:

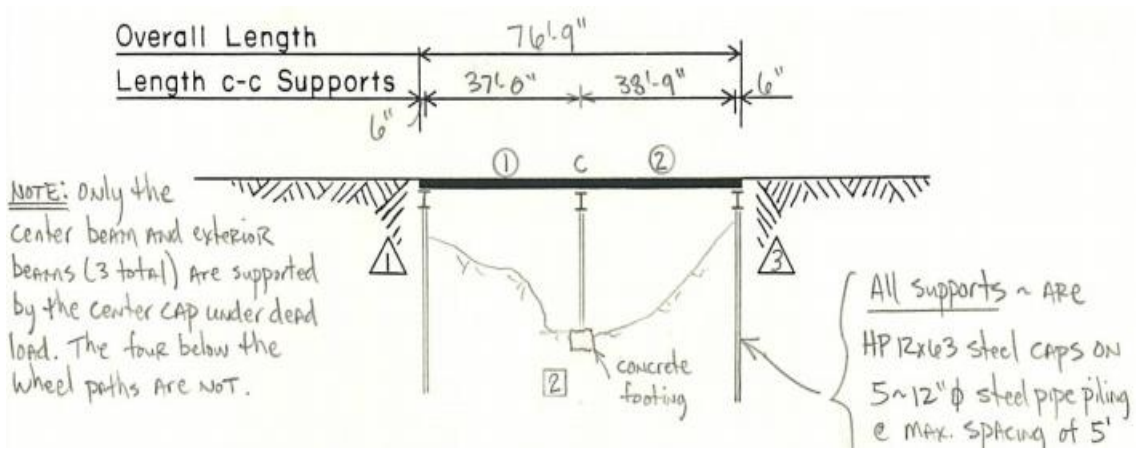
1. Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 21,000-lbs tandem axle and 40,000-lbs gross.
3. Bridge was analyzed as simple span over the full length per TxDOT Load Rating Calculations.
4. Bridge is controlled by steel cap rating—interior stringer ratings are shown.

B.2.16 Bridge SC-16

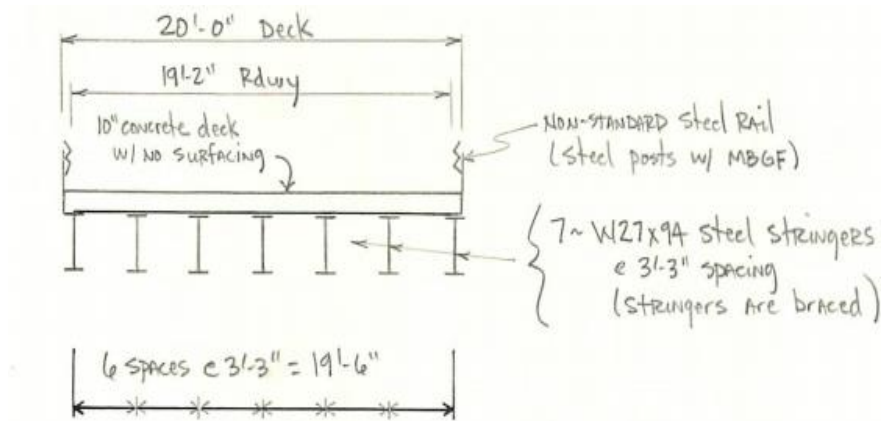
Bridge SC-16 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory) with 2 percent beam section loss due to corrosion, and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 76 ft 9 in., consisting of two continuous spans. The controlling span for load rating is 38 ft 9 in. and is 20 ft wide, with a roadway width of 19 ft 2 in. The bridge carries one lane and has an ADT of 50 vehicles. The girders control the rating of the bridge, which has an inventory gross loading of 12 US tons and an operating gross loading of 20 US tons. The bridge is posted for a 12,500-lbs axle or tandem axle and a 20,000-lbs gross vehicle.



Figure B.81. Bridge SC-16 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure B.82. Bridge Drawings and Photos (TxDOT 2018c)

Table B.41. Characteristics and Interior Girder Flexure Load Ratings of Bridge SC-16

Bridge Characteristics		Interior Stringer Flexure Rating Factors			
Year Built	1999				
On/Off-System	Off				
ADT	50				
Span Lengths	39' – 37'				
Deck Width	20'-0"				
Roadway Width	19'-2"				
Interior Girder Section	W27x94				
Exterior Girder Section	W27x94				
Interior Girder Spacing	3'-3"				
Slab Thickness (in.)	10.0				
Wearing Surface Thickness (in.)	0.0				
Composite	no				
TxDOT Inventory RF	0.34				
TxDOT Operating RF	0.56				
TxDOT Inventory Rating (HS)	6.7				
TxDOT Operating Rating (HS)	11.2				
Posted Axle Limit (lbs)	see note 2				
Calculated Values for Interior Girder RFs					
ASR		LFR		LRFR	
C_{IN} (ksi)	19.8	C_{serv} (kip-ft)	571.5	C_{serv} (kip-ft)	571.5
C_{OP} (ksi)	27.0	C_n (kip-ft)	714.4	C_n (kip-ft)	817.3
f_{DL} (ksi)	19.1	M_{DL} (kip-ft)	379.6	M_{DL} (kip-ft)	379.6
$f_{LL-HS20}$ (ksi)	16.0	$M_{LL-HS20}$ (kip-ft)	317.0	$M_{LL-HL93}$ (kip-ft)	542.4
Inventory HS20 RF	0.04	Inventory HS20 RF	0.32	Inventory HL93 RF	0.14
Operating HS20 RF	0.49	Operating HS20 RF	0.54	Operating HL93 RF	0.18
Inventory Rating (HS)	0.8	Inventory Rating (HS)	6.4		
Operating Rating (HS)	9.8	Operating Rating (HS)	10.8		
		Serviceability Controls	no	Serviceability Controls	yes

Notes:

1. Steel yield strength taken as 36 ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. Posting restrictions include a 12,500-lbs axle or tandem axle and 20,000-lbs gross.
3. Bridge was analyzed as simple span over the full length per TxDOT Load Rating Calculations.

APPENDIX C
SUMMARY OF CONCRETE BRIDGE DETAILS AND LOAD RATING

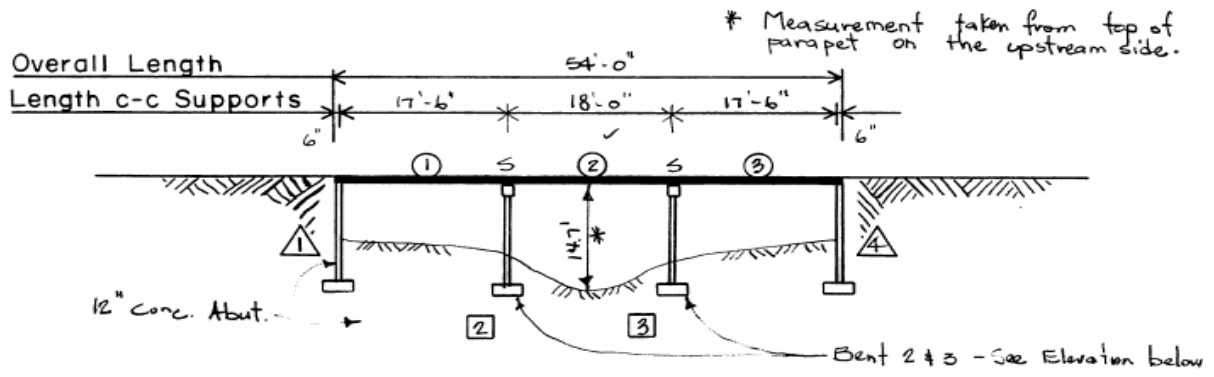
C.1 CONCRETE SLAB BRIDGES

C.1.1 Bridge CS-1

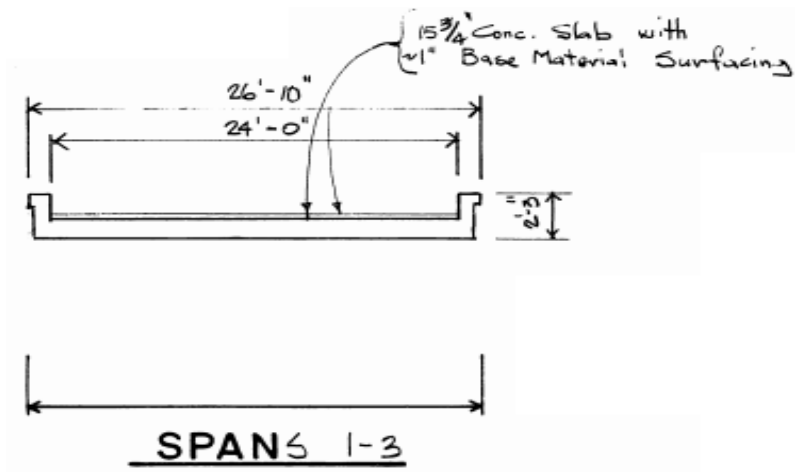
Bridge CS-1 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 5 (Fair). The bridge has a total length of 54 ft and includes three simply supported spans. The controlling span for load rating is 18 ft and is 26 ft 10 in. wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 45 vehicles. The substructure controls the rating of the bridge, which has an inventory gross loading of 31.9 US tons and an operating gross loading of 53.1 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure C.1. Bridge CS-1 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.2. Bridge CS-1 Details (TxDOT 2018c)

Table C.1. Characteristics and Load Ratings of Bridge CS-1

Bridge Characteristics	
Year Built	1920
On/Off-System	Off
ADT	45
Controlling Span Length	18'-0"
Deck Width	26'-1"
Roadway Width	24'-0"
Slab Thickness (in.)	15.8
Top Reinforcement	N/A
Bottom Reinforcement	7/8 in. sq. @ 7 in.
Curb	15 to 18 in. x 27.75 in. high
Curb Reinforcement	Unknown
Wearing Surface Thickness (in.)	1.0
TxDOT Inventory RF	0.89
TxDOT Operating RF	1.48
TxDOT Inventory Rating (HS)	17.7
TxDOT Operating Rating (HS)	29.5
Posted Axle Limit (lbs)	21,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.89	0.90	0.85	0.75
Operating	1.48	1.45	1.40	0.98

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	24.8	C_n (kip-ft/ft)	43.5	C_n (kip-ft/ft)	43.5
C_{OP} (kip-ft/ft)	34.5				5
M_{DL} (kip-ft/ft)	8.9	M_{DL} (kip-ft/ft)	8.9	M_{DL} (kip-ft/ft)	8.9
$M_{LL-HS20}$ (kip-ft/ft)	17.6	$M_{LL-HS20}$ (kip-ft/ft)	17.6	$M_{LL-HL93}$ (kip-ft/ft)	25.4
Inventory HS20 RF	0.90	Inventory HS20 RF	0.84	Inventory HL93 RF	0.75
Operating HS20 RF	1.45	Operating HS20 RF	1.39	Operating HL93 RF	0.98
Inventory Rating (HS)	18.0	Inventory Rating (HS)	16.7		
Operating Rating (HS)	29.0	Operating Rating (HS)	27.86		

Notes:

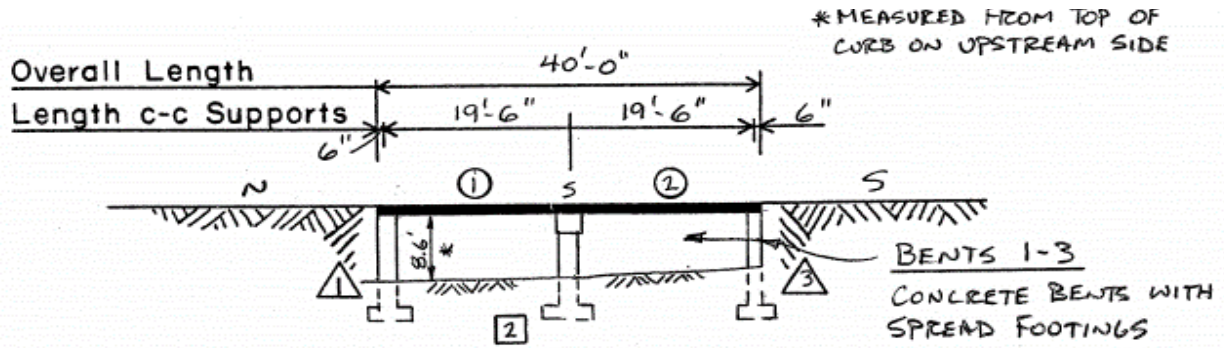
1. $f'_c = 2.5$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is nonstructural.

C.1.2 Bridge CS-2

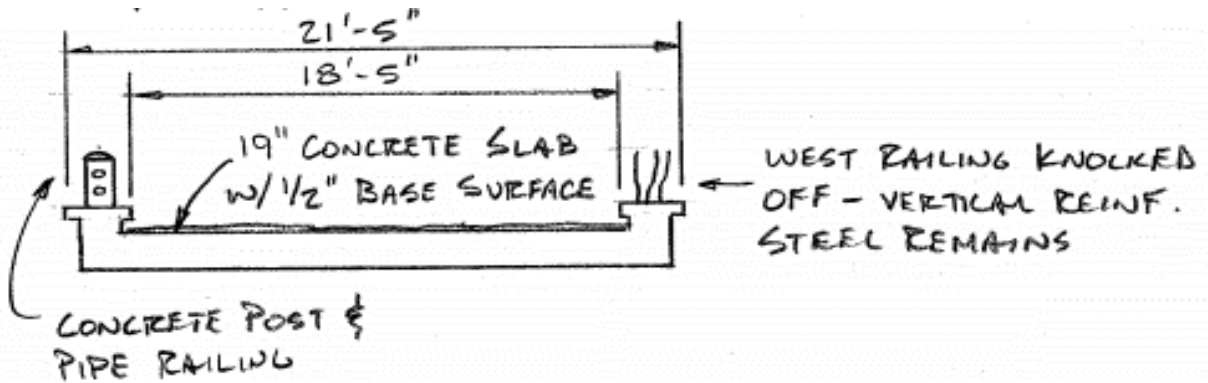
Bridge CS-2 has a deck condition rating of 4 (Poor), a superstructure condition rating of 4 (Poor), and a substructure condition rating of 4 (Poor). The bridge has a total length of 40 ft and includes two simply supported spans. The controlling span for load rating is 19 ft 6 in. and is 21 ft 5 in. wide, with a roadway width of 18 ft 5 in. The bridge carries one lane and has an ADT of 10 vehicles. The CS and substructure control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure C.3. Bridge CS-2 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.4. Bridge CS-2 Details (TxDOT 2018c)

Table C.2. Characteristics and Load Ratings of Bridge CS-2

Bridge Characteristics		CS-2 Flexure Rating Factors									
Year Built	1930	<p>CS-2 Flexure Rating Factors</p> <p>Legend: TxDOT (ASR)</p> <table border="1"> <caption>Data for CS-2 Flexure Rating Factors</caption> <thead> <tr> <th>Condition</th> <th>TxDOT (ASR)</th> </tr> </thead> <tbody> <tr> <td>Inventory</td> <td>0.75</td> </tr> <tr> <td>Operating</td> <td>1.00</td> </tr> </tbody> </table>				Condition	TxDOT (ASR)	Inventory	0.75	Operating	1.00
Condition	TxDOT (ASR)										
Inventory	0.75										
Operating	1.00										
On/Off-System	Off										
ADT	10										
Controlling Span Length	19'-6"										
Deck Width	21'-5"										
Roadway Width	18'-5"										
Slab Thickness (in.)	19.0										
Top Reinforcement	N/A										
Bottom Reinforcement	Unknown										
Curb	Unknown										
Curb Reinforcement	Unknown										
Wearing Surface Thickness (in.)	0.5										
TxDOT Inventory RF	0.75										
TxDOT Operating RF	1.00										
TxDOT Inventory Rating (HS)	15.0										
TxDOT Operating Rating (HS)	20.0										
Posted Axle Limit (lbs)	24,000										
Calculated Values for RFs											
ASR		LFR		LRFR							
C_{IN} (kip-ft/ft)	-	C_n (kip-ft/ft)	-	C_n (kip-ft/ft)	-						
C_{OP} (kip-ft/ft)	-										
M_{DL} (kip-ft/ft)	10.4	M_{DL} (kip-ft/ft)	10.4	M_{DL} (kip-ft/ft)	10.4						
$M_{LL-HS20}$ (kip-ft/ft)	19.3	$M_{LL-HS20}$ (kip-ft/ft)	19.3	$M_{LL-HL93}$ (kip-ft/ft)	30.9						
Inventory HS20 RF	-	Inventory HS20 RF	-	Inventory HL93 RF	-						
Operating HS20 RF	-	Operating HS20 RF	-	Operating HL93 RF	-						
Inventory Rating (HS)	-	Inventory Rating (HS)	-								
Operating Rating (HS)	-	Operating Rating (HS)	-								

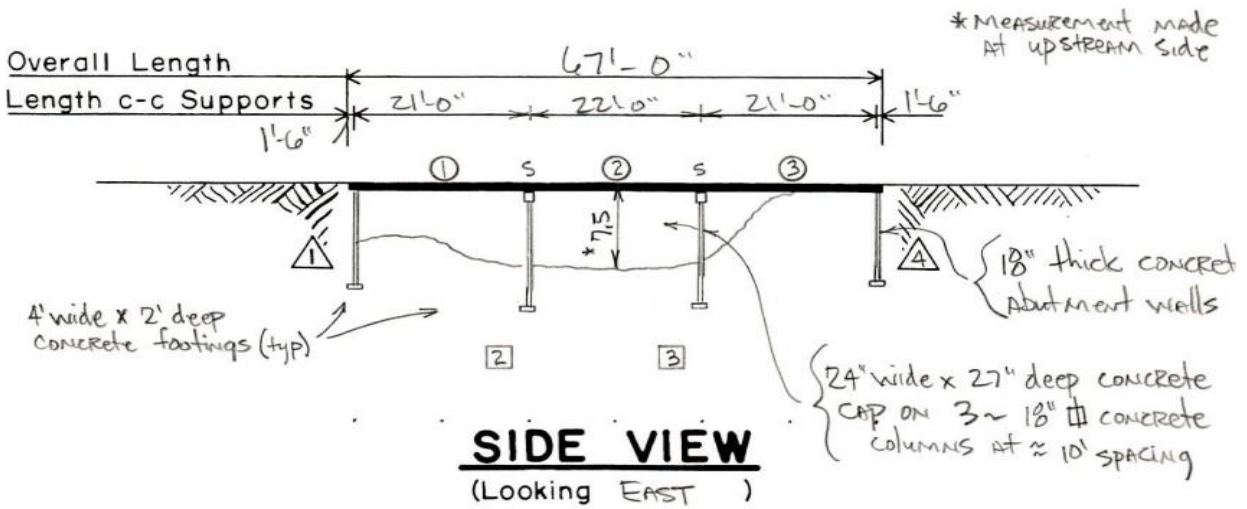
Note: No drawings available.

C.1.3 Bridge CS-3

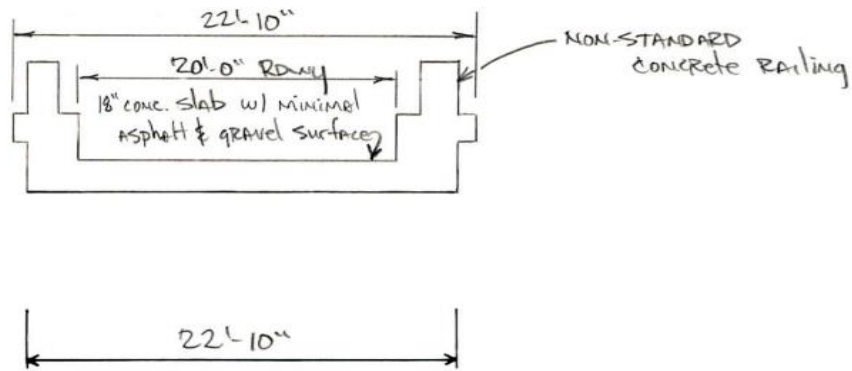
Bridge CS-3 has a deck condition rating of 5 (Fair), a superstructure condition rating of 5 (Fair), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 67 ft and includes three simply supported spans. The controlling span for load rating is 22 ft and is 22 ft 10 in. wide, with a roadway width of 20 ft. The bridge carries two lanes, one in each direction, and has an ADT of 45 vehicles. The substructure controls the rating of the bridge, which has an inventory gross loading of 72.4 US tons and an operating gross loading of 43.4 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure C.5. Bridge CS-3 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.6. Bridge CS-3 Details (TxDOT 2018c)

Table C.3. Characteristics and Load Ratings of Bridge CS-3

Bridge Characteristics	
Year Built	1935
On/Off-System	Off
ADT	45
Controlling Span Length	22'-0"
Deck Width	22'-10"
Roadway Width	20'-0"
Slab Thickness (in.)	19.3
Top Reinforcement	N/A
Bottom Reinforcement	7/8 in. sq. @ 5.5 in.
Curb	15 to 18 in. wide x 12 in. high
Curb Reinforcement	Unknown
Wearing Surface Thickness (in.)	5.0
TxDOT Inventory RF	1.21
TxDOT Operating RF	2.01
TxDOT Inventory Rating (HS)	40.2
TxDOT Operating Rating (HS)	24.1
Posted Axle Limit (lbs)	24,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	1.21	1.21	1.16	1.02
Operating	2.01	2.01	1.93	1.33

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	39.3	C_n (kip-ft/ft)	68.9	C_n (kip-ft/ft)	68.9
C_{OP} (kip-ft/ft)	54.6				
M_{DL} (kip-ft/ft)	15.9	M_{DL} (kip-ft/ft)	15.9	M_{DL} (kip-ft/ft)	15.9
$M_{LL-HS20}$ (kip-ft/ft)	19.2	$M_{LL-HS20}$ (kip-ft/ft)	19.2	$M_{LL-HL93}$ (kip-ft/ft)	29.6
Inventory HS20 RF	1.22	Inventory HS20 RF	1.16	Inventory HL93 RF	1.02
Operating HS20 RF	2.01	Operating HS20 RF	1.93	Operating HL93 RF	1.33
Inventory Rating (HS)	24.4	Inventory Rating (HS)	23.14		
Operating Rating (HS)	40.2	Operating Rating (HS)	38.64		

Notes:

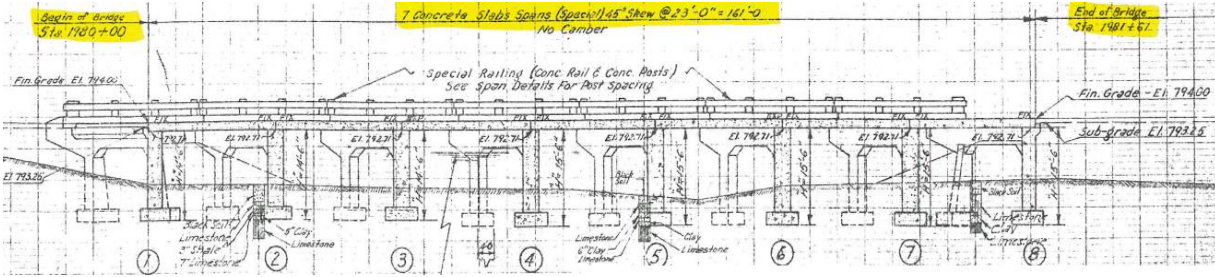
1. $f'_c = 2.5$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is nonstructural.

C.1.4 Bridge CS-4

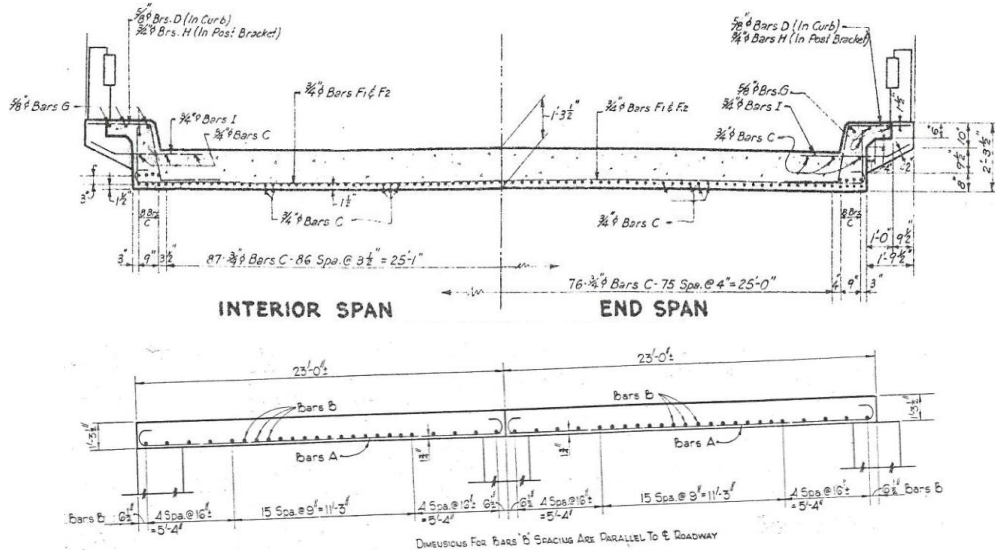
Bridge CS-4 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 161 ft and includes seven simply supported spans. The controlling span for load rating is 23 ft and is 46 ft wide, with a roadway width of 43 ft 10 in. The bridge carries two lanes, one in each direction, and has an ADT of 3890 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 18.7 US tons and an operating gross loading of 31.3 US tons. The bridge is posted for a 33,000-lbs tandem axle and a 44,000-lbs single vehicle.



Figure C.7. Bridge CS-4 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.8. Bridge CS-4 Details (TxDOT 2018c)

Table C.4. Characteristics and Load Ratings of Bridge CS-4

Bridge Characteristics		CS-4 Flexure Rating Factors			
Year Built	1943				
On/Off-System	On				
ADT	3890				
Controlling Span Length	23'-0"				
Deck Width	46'-0"				
Roadway Width	43'-10"				
Slab Thickness (in.)	15.5				
Top Reinforcement	N/A				
Bottom Reinforcement	3/4 in. diam. @ 3.5 in.				
Curb	12 in. wide x 12 in. high				
Curb Reinforcement	Unknown				
Wearing Surface Thickness (in.)	2.0				
TxDOT Inventory RF	0.52				
TxDOT Operating RF	0.87				
TxDOT Inventory Rating (HS)	10.4				
TxDOT Operating Rating (HS)	17.4				
Posted Axle Limit (lbs)	see note 1				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	54	C_n (kip-ft/ft)	26	C_n (kip-ft/ft)	52
C_{OP} (kip-ft/ft)	17.2				
M_{DL} (kip-ft/ft)	6.4	M_{DL} (kip-ft/ft)	6.4	M_{DL} (kip-ft/ft)	12.9
$M_{LL-HS20}$ (kip-ft/ft)	15.8	$M_{LL-HS20}$ (kip-ft/ft)	15.8	$M_{LL-HL93}$ (kip-ft/ft)	41.8
Inventory HS20 RF	0.37	Inventory HS20 RF	0.51	Inventory HL93 RF	0.72
Operating HS20 RF	0.68	Operating HS20 RF	0.86	Operating HL93 RF	0.94
Inventory Rating (HS)	7.4	Inventory Rating (HS)	10.2		
Operating Rating (HS)	13.6	Operating Rating (HS)	17.2		

Notes:

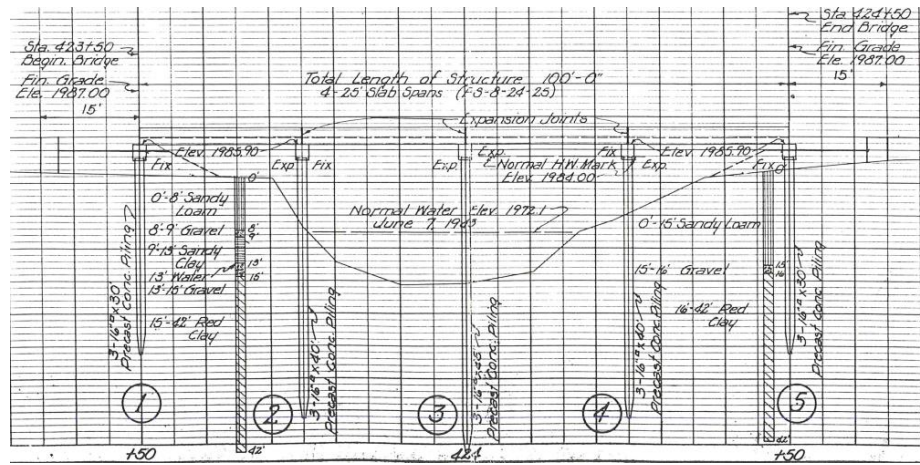
1. Posting restrictions include a 33,000-lbs tandem axle and a 44,000-lbs single vehicle.
2. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
3. $f_y = 40$ ksi per TxDOT Load Rating Calculations.
4. Curb is nonstructural. Both curbs removed during widening of bridge.

C.1.5 Bridge CS-5

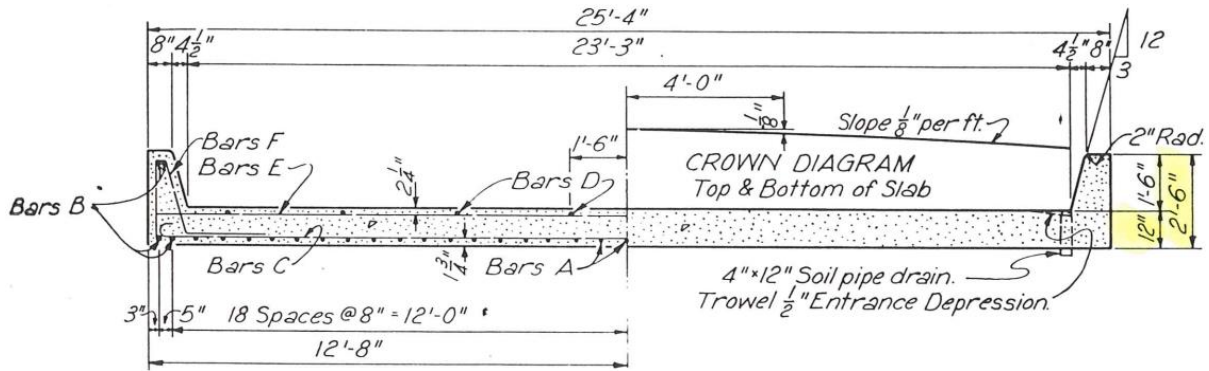
Bridge CS-5 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 7 (Good). The bridge has a total length of 100 ft and includes four simply supported spans. The controlling span for load rating is 25 ft and is 46 ft 4 in. wide, with a roadway width of 43 ft 5 in. The bridge carries two lanes, one in each direction, and has an ADT of 1280 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 15.1 US tons and an operating gross loading of 25.2 US tons. The bridge is posted for a 20,000-lbs single axle and a 26,000-lbs tandem axle.



Figure C.9. Bridge CS-5 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.10. Bridge CS-5 Details (TxDOT 2018c)

Table C.5. Characteristics and Load Ratings of Bridge CS-5

Bridge Characteristics	
Year Built	1946
On/Off-System	On
ADT	1280
Controlling Span Length	25'-0"
Deck Width	46'-4"
Roadway Width	43'-5"
Slab Thickness (in.)	12.0
Top Reinforcement	N/A
Bottom Reinforcement	1 in. diam. @ 8 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 – 1.25 in. sq. bars T&B
Wearing Surface Thickness (in.)	1.3
TxDOT Inventory RF	0.42
TxDOT Operating RF	0.70
TxDOT Inventory Rating (HS)	8.4
TxDOT Operating Rating (HS)	14.0
Posted Axle Limit (lbs)	see note 1

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	16.1	C_n (kip-ft/ft)	28.2	C_n (kip-ft/ft)	28.2
C_{OP} (kip-ft/ft)	22.3				
M_{DL} (kip-ft/ft)	12.9	M_{DL} (kip-ft/ft)	12.9	M_{DL} (kip-ft/ft)	12.9
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	45
Inventory HS20 RF	0.30	Inventory HS20 RF	1.01	Inventory HL93 RF	0.19
Operating HS20 RF	1.65	Operating HS20 RF	1.74	Operating HL93 RF	0.25
Inventory Rating (HS)	6.0	Inventory Rating (HS)	20.2		
Operating Rating (HS)	33.0	Operating Rating (HS)	34.8		

Notes:

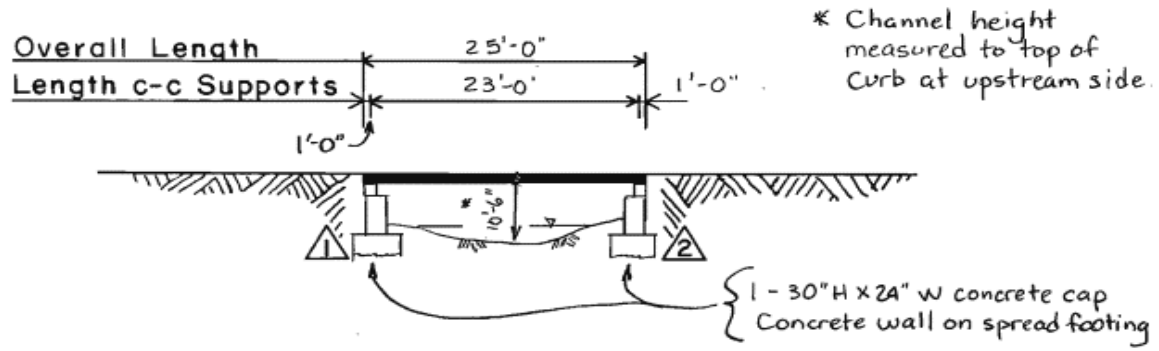
1. Posting restrictions include a 20,000-lbs single axle and a 26,000-lbs tandem axle.
2. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
3. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
4. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.6 Bridge CS-6

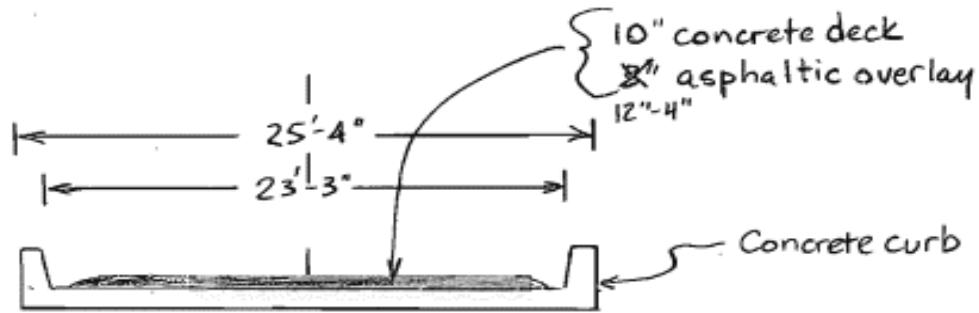
Bridge CS-6 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 5 (Fair). The bridge has a total length of 25 ft and includes one simply supported span. The controlling span for load rating is 23 ft and is 25 ft 4 in. wide, with a roadway width of 23 ft 2 in. The bridge carries two lanes, one in each direction, and has an ADT of 170 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure C.11. Bridge CS-6 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.12. Bridge CS-6 Details (TxDOT 2018c)

Table C.6. Characteristics and Load Ratings of Bridge CS-6

Bridge Characteristics		CS-6 Flexure Rating Factors			
Year Built	1947				
On/Off-System	Off				
ADT	170				
Controlling Span Length	23'-0"				
Deck Width	25'-4"				
Roadway Width	23'-2"				
Slab Thickness (in.)	10.0				
Top Reinforcement	N/A				
Bottom Reinforcement	Unknown				
Curb	Unknown				
Curb Reinforcement	Unknown				
Wearing Surface Thickness (in.)	12.0				
TxDOT Inventory RF	0.75				
TxDOT Operating RF	1.00				
TxDOT Inventory Rating (HS)	15.0				
TxDOT Operating Rating (HS)	20.0				
Posted Axle Limit (lbs)	24,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	-	C_n (kip-ft/ft)	-	C_n (kip-ft/ft)	-
C_{OP} (kip-ft/ft)	-				
M_{DL} (kip-ft/ft)	16.3	M_{DL} (kip-ft/ft)	16.3	M_{DL} (kip-ft/ft)	16.3
$M_{LL-HS20}$ (kip-ft/ft)	21.5	$M_{LL-HS20}$ (kip-ft/ft)	21.5	$M_{LL-HL93}$ (kip-ft/ft)	4
Inventory HS20 RF	-	Inventory HS20 RF	-	Inventory HL93 RF	-
Operating HS20 RF	-	Operating HS20 RF	-	Operating HL93 RF	-
Inventory Rating (HS)	-	Inventory Rating (HS)	-		
Operating Rating (HS)	-	Operating Rating (HS)	-		

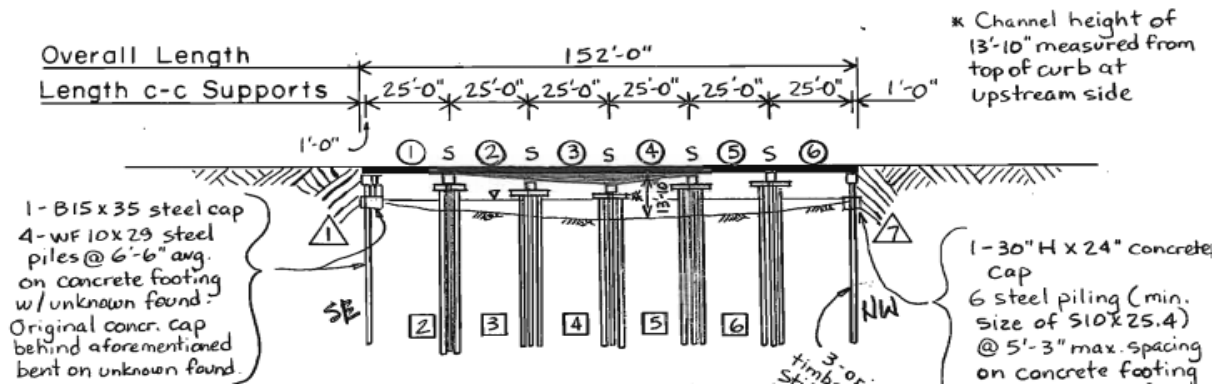
Note: No drawings available.

C.1.7 Bridge CS-7

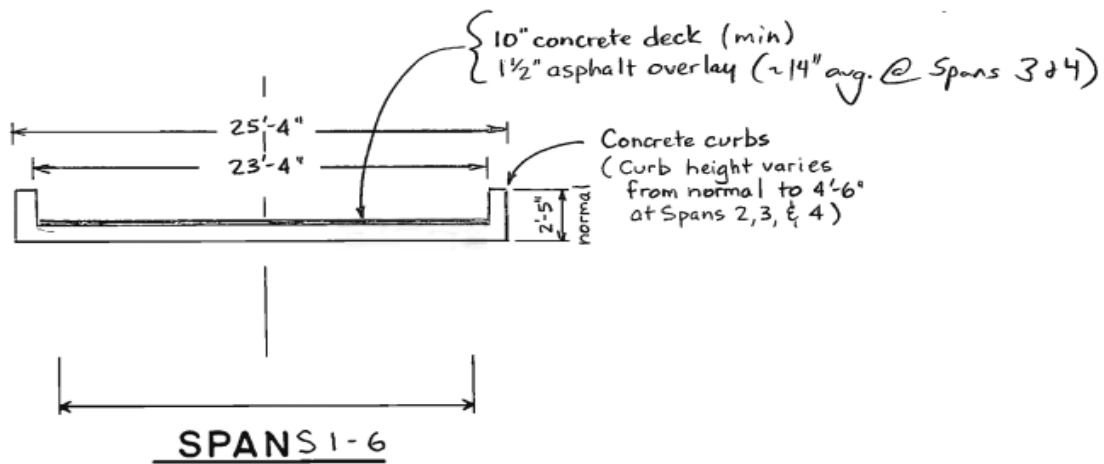
Bridge CS-7 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 5 (Fair), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 152 ft and includes six simply supported spans. The controlling span for load rating is 25 ft and is 25 ft 4 in. wide, with a roadway width of 23 ft 4 in. The bridge carries two lanes, one in each direction, and has an ADT of 60 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 12.2 US tons and an operating gross loading of 18.2 US tons. The bridge is posted for a 10,000-lbs axle.



Figure C.13. Bridge CS-7 (TxDOT 2018c)



(a) Elevation View



(b) Cross-Section



(c) Underside Superstructure View

Figure C.14. Bridge CS-7 Details (TxDOT 2018c)

Table C.7. Characteristics and Load Ratings of Bridge CS-7

Bridge Characteristics	
Year Built	1947
On/Off-System	Off
ADT	60
Controlling Span Length	25'-0"
Deck Width	25'-4"
Roadway Width	23'-4"
Slab Thickness (in.)	10.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	1 in. diam. @ 8 in.
Curb	8 to 12.5 in. wide x 12 in. high
Curb Reinforcement	2 - #8 bars T & 2 - #11 bars B
Wearing Surface Thickness (in.)	14.0
TxDOT Inventory RF	0.34
TxDOT Operating RF	0.51
TxDOT Inventory Rating (HS)	6.8
TxDOT Operating Rating (HS)	10.1
Posted Axle Limit (lbs)	10,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.34	0.34	0.26	0.32
Operating	0.51	0.44	0.51	0.41

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	12.8	C_n (kip-ft/ft)	22.3	C_n (kip-ft/ft)	22.3
C_{OP} (kip-ft/ft)	17.8				
M_{DL} (kip-ft/ft)	22.4	M_{DL} (kip-ft/ft)	22.4	M_{DL} (kip-ft/ft)	22.4
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	47.8
Inventory HS20 RF	0.00	Inventory HS20 RF	0.26	Inventory HL93 RF	0.32
Operating HS20 RF	0.44	Operating HS20 RF	0.51	Operating HL93 RF	0.41
Inventory Rating (HS)	0.0*	Inventory Rating (HS)	5.14		
Operating Rating (HS)	8.8	Operating Rating (HS)	10.1		

Notes:

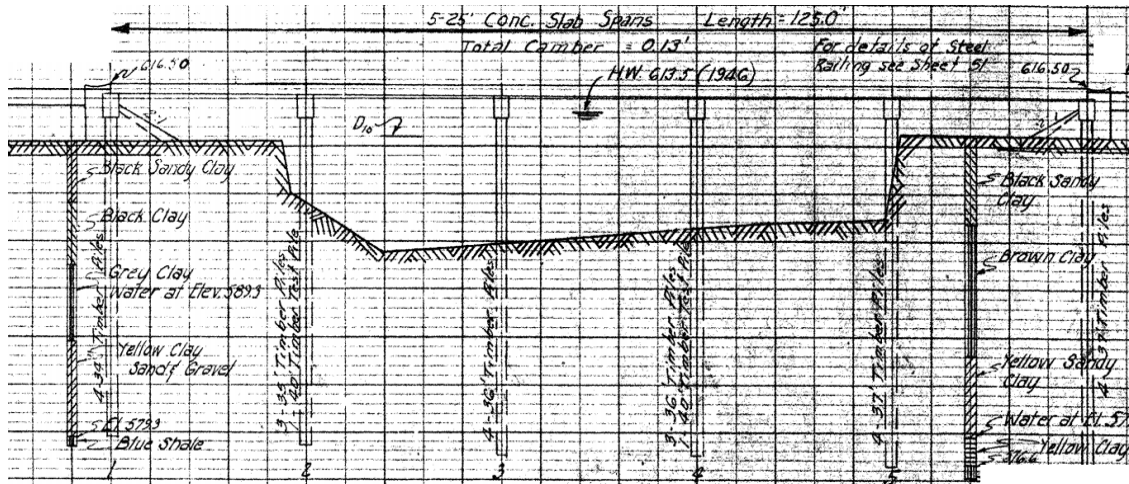
1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
 2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
 3. Curb is structural. Exterior strip controls for ASR and LFR ratings.
- *RF calculated to be negative due to high dead load demands.

C.1.8 Bridge CS-8

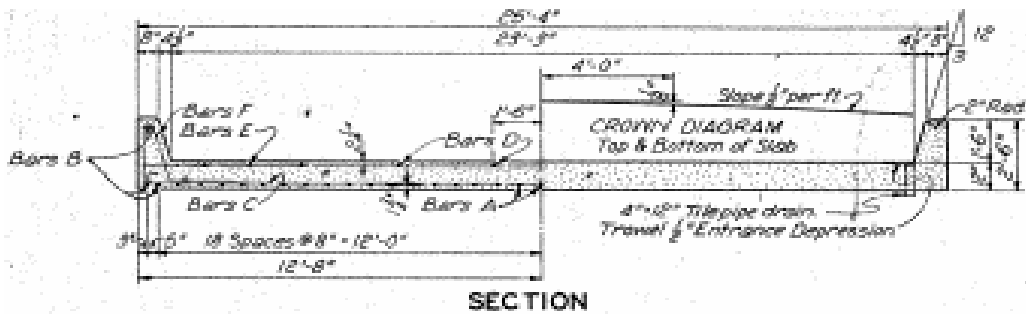
Bridge CS-8 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 125 ft and includes five simply supported spans. The controlling span for load rating is 25 ft and is 25 ft 4 in. wide, with a roadway width of 23 ft. The bridge carries two lanes, one in each direction, and has an ADT of 670 vehicles. The substructure controls the rating of the bridge, which has an inventory gross loading of 19.6 US tons and an operating gross loading of 38.5 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.15. Bridge CS-8 (TxDOT 2018c)



(a) Elevation View



(b) Cross-Section



(c) Underside Superstructure View

Figure C.16. Bridge CS-8 Details (TxDOT 2018c)

Table C.8. Characteristics and Load Ratings of Bridge CS-8

Bridge Characteristics	
Year Built	1947
On/Off-System	On
ADT	670
Controlling Span Length	25'-0"
Deck Width	25'-4"
Roadway Width	23'-0"
Slab Thickness (in.)	12.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	1 in. diam. @ 8 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 – 1.25 in. sq. bars T&B
Wearing Surface Thickness (in.)	6.0
TxDOT Inventory RF	0.55
TxDOT Operating RF	1.07
TxDOT Inventory Rating (HS)	10.9
TxDOT Operating Rating (HS)	21.4
Posted Axle Limit (lbs)	28,000

CS-8 Flexure Rating Factors

State	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.55	0.16	0.55	0.25
Operating	1.07	0.86	1.07	0.31

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	16.1	C_n (kip-ft/ft)	28.2	C_n (kip-ft/ft)	28.2
C_{OP} (kip-ft/ft)	22.3				
M_{DL} (kip-ft/ft)	17.8	M_{DL} (kip-ft/ft)	17.8	M_{DL} (kip-ft/ft)	17.8
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	47.8
Inventory HS20 RF	0.16	Inventory HS20 RF	0.55	Inventory HL93 RF	0.24
Operating HS20 RF	0.86	Operating HS20 RF	1.09	Operating HL93 RF	0.31
Inventory Rating (HS)	3.2	Inventory Rating (HS)	11		
Operating Rating (HS)	17.2	Operating Rating (HS)	21.8		

Notes:

- $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
- $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
- Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.9 Bridge CS-9

Bridge CS-9 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 75 ft and includes three simply supported spans. The controlling span for load rating is 25 ft and is 21 ft 4 in. wide, with a roadway width of 20 ft. The bridge carries two lanes, one in each direction, and has an ADT of 30 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 16 US tons and an operating gross loading of 33.7 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.17. Bridge CS-9 (TxDOT 2018c)

Table C.9. Characteristics and Load Ratings of Bridge CS-9

Bridge Characteristics	
Year Built	1948
On/Off-System	On
ADT	30
Controlling Span Length	25'-0"
Deck Width	21'-4"
Roadway Width	20'-0"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	1 in. diam. @ 8.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 – 1.25 in. sq. bars T&B
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	0.45
TxDOT Operating RF	0.94
TxDOT Inventory Rating (HS)	8.9
TxDOT Operating Rating (HS)	18.7
Posted Axle Limit (lbs)	28,000

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.45	0.15	0.45	0.15
Operating	0.94	0.67	0.94	0.18

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.5	C_n (kip-ft/ft)	23.5	C_n (kip-ft/ft)	23.5
C_{OP} (kip-ft/ft)	18.7				
M_{DL} (kip-ft/ft)	14.6	M_{DL} (kip-ft/ft)	14.6	M_{DL} (kip-ft/ft)	14.6
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	50.6
Inventory HS20 RF	0.15	Inventory HS20 RF	0.45	Inventory HL93 RF	0.14
Operating HS20 RF	0.67	Operating HS20 RF	0.95	Operating HL93 RF	0.18
Inventory Rating (HS)	3.0	Inventory Rating (HS)	9		
Operating Rating (HS)	13.4	Operating Rating (HS)	19		

Notes:

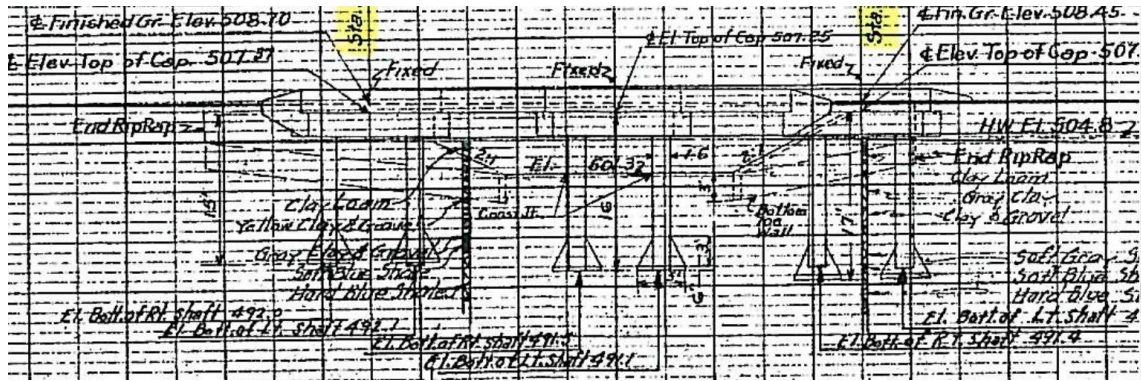
1. $f'_c = 2.5$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.10 Bridge CS-10

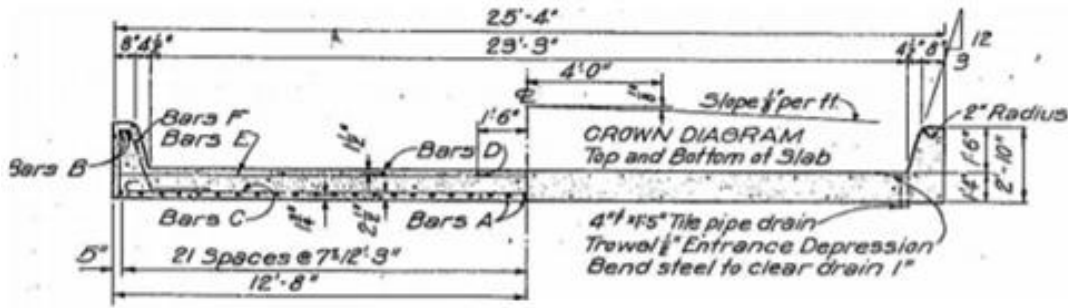
Bridge CS-10 has a deck condition rating of 8 (Very Good), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 50 ft and includes two simply supported spans. The controlling span for load rating is 25 ft and is 44 ft wide, with a roadway width of 43 ft. The bridge carries two lanes, one in each direction, and has an ADT of 9590 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 19.1 US tons and an operating gross loading of 31.9 US tons. The bridge is posted for a 20,000-lbs single axle and a 34,000-lbs tandem axle.



Figure C.19. Bridge CS-10 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.20. Bridge CS-10 Details (TxDOT 2018c)

Table C.10. Characteristics and Load Ratings of Bridge CS-10

Bridge Characteristics	
Year Built	1949
On/Off-System	On
ADT	9590
Controlling Span Length	25'-0"
Deck Width	44'-0"
Roadway Width	43'-0"
Slab Thickness (in.)	16.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	1 in. diam. @ 7 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 - #5 bars T & 2 - #8 bars B
Wearing Surface Thickness (in.)	6.0
TxDOT Inventory RF	0.54
TxDOT Operating RF	0.89
TxDOT Inventory Rating (HS)	10.6
TxDOT Operating Rating (HS)	17.7
Posted Axle Limit (lbs)	see note 1

CS-10 Flexure Rating Factors

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.54	0.45	0.91	0.50
Operating	0.89	1.87	1.87	0.63

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	19.6	C_n (kip-ft/ft)	34.4	C_n (kip-ft/ft)	45.3
C_{OP} (kip-ft/ft)	27.2				
M_{DL} (kip-ft/ft)	15.6	M_{DL} (kip-ft/ft)	15.6	M_{DL} (kip-ft/ft)	20.8
$M_{LL-HS20}$ (kip-ft/ft)	20.8	$M_{LL-HS20}$ (kip-ft/ft)	20.8	$M_{LL-HL93}$ (kip-ft/ft)	45
Inventory HS20 RF	0.45	Inventory HS20 RF	0.91	Inventory HL93 RF	0.49
Operating HS20 RF	1.87	Operating HS20 RF	1.87	Operating HL93 RF	0.63
Inventory Rating (HS)	9.0	Inventory Rating (HS)	18.2		
Operating Rating (HS)	37.4	Operating Rating (HS)	37.4		

Notes:

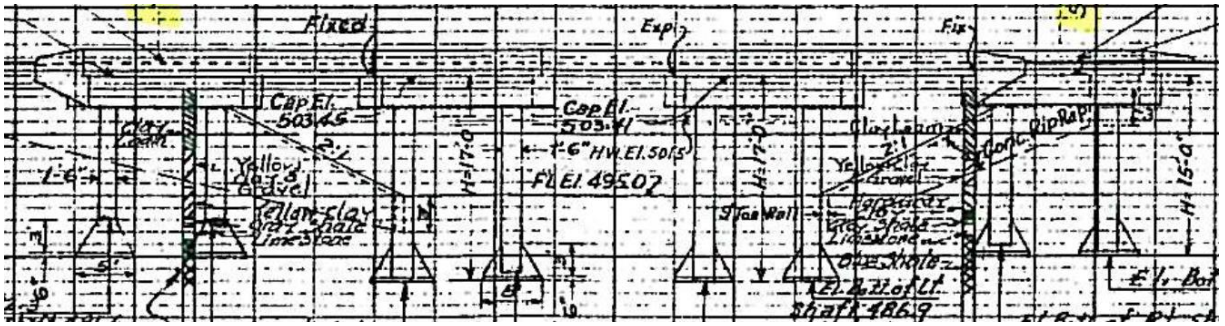
1. Posting restrictions include a 20,000-lbs single axle and a 34,000-lbs tandem axle.
2. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
3. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
4. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.11 Bridge CS-11

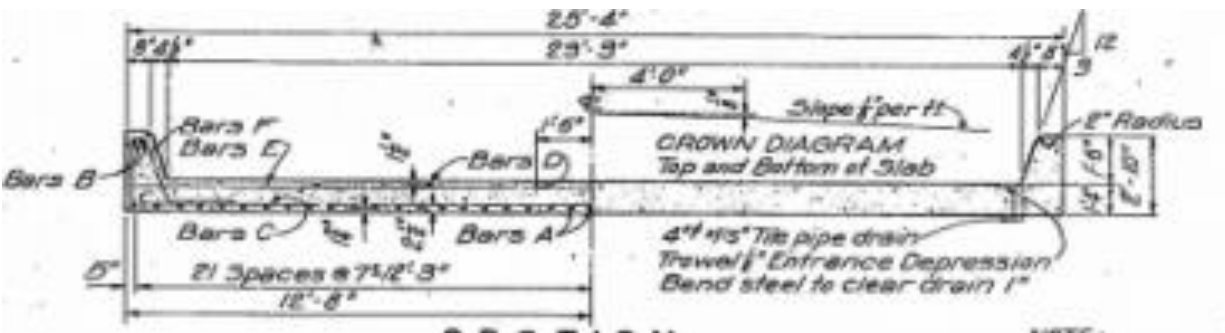
Bridge CS-11 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 75 ft and includes three simply supported spans. The controlling span for load rating is 25 ft and is 44 ft wide, with a roadway width of 44 ft. The bridge carries two lanes, one in each direction, and has an ADT of 9590 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 19.1 US tons and an operating gross loading of 31.9 US tons. The bridge is posted for a 20,000-lbs single axle and a 34,000-lbs tandem axle.



Figure C.21. Bridge CS-11 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.22. Bridge CS-11 (TxDOT 2018c)

Table C.11. Characteristics and Load Ratings of Bridge CS-11

Bridge Characteristics	
Year Built	1949
On/Off-System	On
ADT	9590
Controlling Span Length	25'-0"
Deck Width	44'-0"
Roadway Width	44'-0"
Slab Thickness (in.)	16.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	1 in. diam. @ 7 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 - #5 bars T & 2 - #8 bars B
Wearing Surface Thickness (in.)	6.0
TxDOT Inventory RF	0.53
TxDOT Operating RF	0.89
TxDOT Inventory Rating (HS)	10.6
TxDOT Operating Rating (HS)	17.7
Posted Axle Limit (lbs)	see note 1

CS-11 Flexure Rating Factors

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.53	0.45	0.91	0.50
Operating	0.89	1.87	1.87	0.63

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	19.6	C_n (kip-ft/ft)	34.4	C_n (kip-ft/ft)	45.3
C_{OP} (kip-ft/ft)	27.2				
M_{DL} (kip-ft/ft)	15.6	M_{DL} (kip-ft/ft)	15.6	M_{DL} (kip-ft/ft)	20.8
$M_{LL-HS20}$ (kip-ft/ft)	20.8	$M_{LL-HS20}$ (kip-ft/ft)	20.8	$M_{LL-HL93}$ (kip-ft/ft)	45
Inventory HS20 RF	0.45	Inventory HS20 RF	0.91	Inventory HL93 RF	0.49
Operating HS20 RF	1.87	Operating HS20 RF	1.87	Operating HL93 RF	0.63
Inventory Rating (HS)	9.0	Inventory Rating (HS)	18.2		
Operating Rating (HS)	37.4	Operating Rating (HS)	37.4		

Notes:

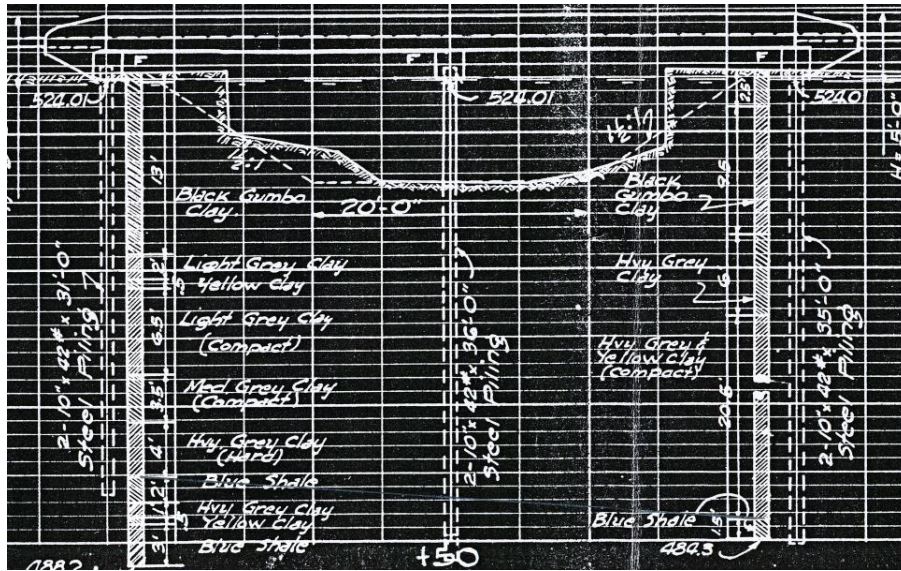
1. Posting restrictions include a 20,000-lbs single axle and a 34,000-lbs tandem axle.
2. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
3. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
4. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.12 Bridge CS-12

Bridge CS-12 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 50 ft and includes two simply supported spans. The controlling span for load rating is 25 ft and is 21 ft 4 in. wide, with a roadway width of 19 ft 6 in. The bridge carries two lanes, one in each direction, and has an ADT of 40 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 16.4 US tons and an operating gross loading of 34.7 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.23. Bridge CS-12 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.24. Bridge CS-12 Details (TxDOT 2018c)

Table C.12. Characteristics and Load Ratings of Bridge CS-12

Bridge Characteristics	
Year Built	1950
On/Off-System	On
ADT	40
Controlling Span Length	25'-0"
Deck Width	21'-4"
Roadway Width	19'-6"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	7/8 in. diam. @ 6.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 – 1.25 in. sq. bars T&B
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	0.46
TxDOT Operating RF	0.97
TxDOT Inventory Rating (HS)	9.1
TxDOT Operating Rating (HS)	19.3
Posted Axle Limit (lbs)	28,000

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.5	C_n (kip-ft/ft)	23.7	C_n (kip-ft/ft)	23.7
C_{OP} (kip-ft/ft)	18.8				
M_{DL} (kip-ft/ft)	14.6	M_{DL} (kip-ft/ft)	14.6	M_{DL} (kip-ft/ft)	14.6
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	50.6
Inventory HS20 RF	0.25	Inventory HS20 RF	0.45	Inventory HL93 RF	0.14
Operating HS20 RF	0.76	Operating HS20 RF	0.97	Operating HL93 RF	0.19
Inventory Rating (HS)	5.0	Inventory Rating (HS)	9		
Operating Rating (HS)	15.2	Operating Rating (HS)	19.4		

Notes:

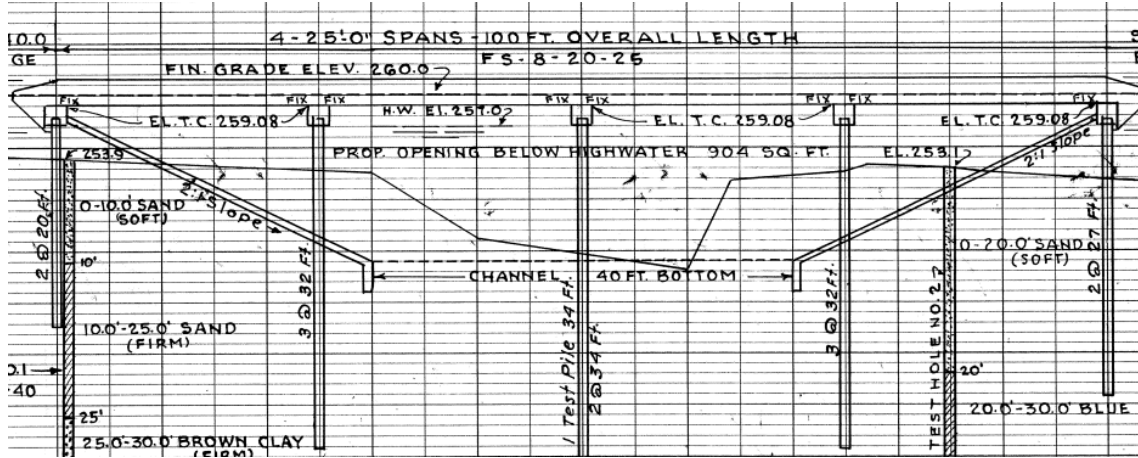
1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.13 Bridge CS-13

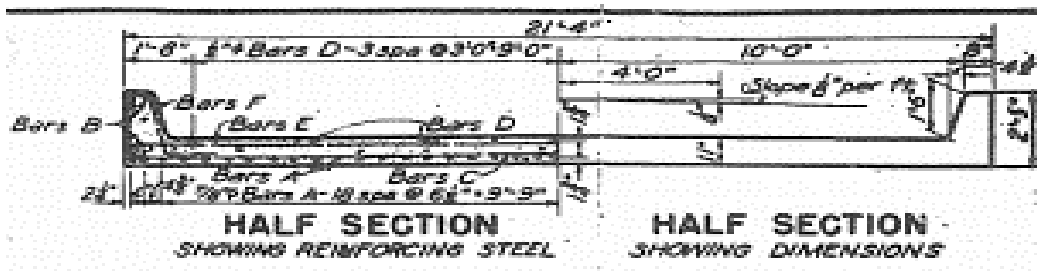
Bridge CS-13 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 50 ft and includes two simply supported spans. The controlling span for load rating is 25 ft and is 21 ft 4 in. wide, with a roadway width of 19 ft 4 in. The bridge carries two lanes, one in each direction, and has an ADT of 50 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 19.6 US tons and an operating gross loading of 41.2 US tons. The bridge is posted for a 28,000-lbs tandem axle and a 58240-lbs gross.



Figure C.25. Bridge CS-13 (TxDOT 2018c)



(a) Elevation View



(b) Cross-Section



(c) Underside Superstructure View

Figure C.26. Bridge CS-13 Details (TxDOT 2018c)

Table C.13. Characteristics and Load Ratings of Bridge CS-13

Bridge Characteristics	
Year Built	1951
On/Off-System	On
ADT	50
Controlling Span Length	25'-0"
Deck Width	21'-4"
Roadway Width	19'-4"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	7/8 in. diam. @ 6.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 #11 in. bars T&B
Wearing Surface Thickness (in.)	2.5
TxDOT Inventory RF	0.55
TxDOT Operating RF	1.15
TxDOT Inventory Rating (HS)	10.9
TxDOT Operating Rating (HS)	22.9
Posted Axle Limit (lbs)	see note 1

CS-13 Flexure Rating Factors

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.55	0.18	0.55	0.12
Operating	1.15	0.67	1.15	0.17

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.4	C_n (kip-ft/ft)	26.6	C_n (kip-ft/ft)	23.4
C_{OP} (kip-ft/ft)	18.6				
M_{DL} (kip-ft/ft)	14.2	M_{DL} (kip-ft/ft)	14.2	M_{DL} (kip-ft/ft)	14.2
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	50.6
Inventory HS20 RF	0.16	Inventory HS20 RF	0.54	Inventory HL93 RF	0.13
Operating HS20 RF	0.67	Operating HS20 RF	1.16	Operating HL93 RF	0.17
Inventory Rating (HS)	3.2	Inventory Rating (HS)	10.86		
Operating Rating (HS)	13.4	Operating Rating (HS)	23.2		

Notes:

1. Posting restrictions include a 28,000-lbs tandem axle and a 58240-lbs gross.
2. $f'_c = 2.5$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. $f_y = 38$ ksi per TxDOT Load Rating Calculations and Mill Test Certificate.
4. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.14 Bridge CS-14

Bridge CS-14 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 100 ft and includes four simply supported spans. The controlling span for load rating is 25 ft and is 25 ft 5 in. wide, with a roadway width of 23 ft 2 in. The bridge carries two lanes, one in each direction, and has an ADT of 310 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 26.1 US tons and an operating gross loading of 50 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.27. Bridge CS-14 (TxDOT 2018c)

Table C.14. Characteristics and Load Ratings of Bridge CS-14

Bridge Characteristics	
Year Built	1951
On/Off-System	On
ADT	310
Controlling Span Length	25'-0"
Deck Width	25'-5"
Roadway Width	23'-2"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	7/8 in. diam. @ 6.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 #11 in. bars T&B
Wearing Surface Thickness (in.)	0.5
TxDOT Inventory RF	0.57
TxDOT Operating RF	0.96
TxDOT Inventory Rating (HS)	14.5
TxDOT Operating Rating (HS)	27.8
Posted Axle Limit (lbs)	28,000

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.5	C_n (kip-ft/ft)	23.7	C_n (kip-ft/ft)	23.7
C_{OP} (kip-ft/ft)	18.8				
M_{DL} (kip-ft/ft)	12.1	M_{DL} (kip-ft/ft)	12.1	M_{DL} (kip-ft/ft)	12.1
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	47.7
Inventory HS20 RF	0.31	Inventory HS20 RF	0.57	Inventory HL93 RF	0.12
Operating HS20 RF	0.92	Operating HS20 RF	1.10	Operating HL93 RF	0.15
Inventory Rating (HS)	6.2	Inventory Rating (HS)	11.3		
Operating Rating (HS)	18.4	Operating Rating (HS)	22		

Notes:

1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.15 Bridge CS-15

Bridge CS-15 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 125 ft and includes five simply supported spans. The controlling span for load rating is 25 ft and is 25 ft 4 in. wide, with a roadway width of 23 ft 1 in. The bridge carries two lanes, one in each direction, and has an ADT of 310 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 26.1 US tons and an operating gross loading of 50 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.29. Bridge CS-15 (TxDOT 2018c)

Table C.15. Characteristics and Load Ratings of Bridge CS-15

Bridge Characteristics	
Year Built	1951
On/Off-System	On
ADT	310
Controlling Span Length	25'-0"
Deck Width	25'-4"
Roadway Width	23'-1"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	7/8 in. diam. @ 6.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 #11 in. bars T&B
Wearing Surface Thickness (in.)	0.5
TxDOT Inventory RF	0.57
TxDOT Operating RF	0.96
TxDOT Inventory Rating (HS)	14.5
TxDOT Operating Rating (HS)	27.8
Posted Axle Limit (lbs)	28,000

CS-15 Flexure Rating Factors

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.57	0.31	0.57	0.11
Operating	0.96	0.92	1.10	0.15

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.5	C_n (kip-ft/ft)	23.7	C_n (kip-ft/ft)	23.7
C_{OP} (kip-ft/ft)	18.8				
M_{DL} (kip-ft/ft)	12.1	M_{DL} (kip-ft/ft)	12.1	M_{DL} (kip-ft/ft)	12.1
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	47.8
Inventory HS20 RF	0.31	Inventory HS20 RF	0.56	Inventory HL93 RF	0.12
Operating HS20 RF	0.92	Operating HS20 RF	1.10	Operating HL93 RF	0.15
Inventory Rating (HS)	6.2	Inventory Rating (HS)	11.2		
Operating Rating (HS)	18.4	Operating Rating (HS)	22		

Notes:

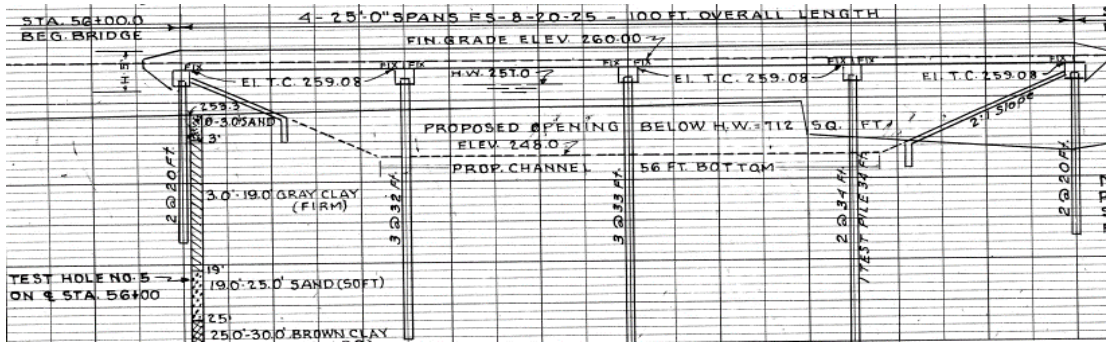
1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.16 Bridge CS-16

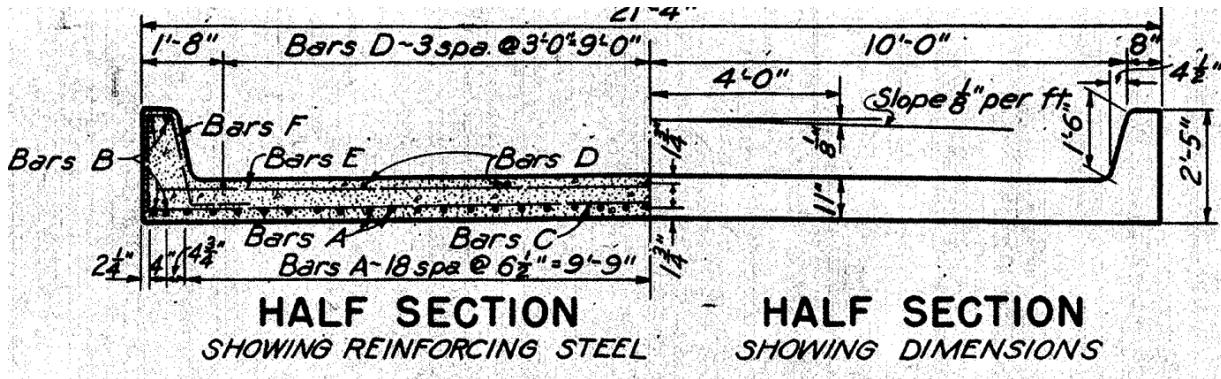
Bridge CS-16 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 100 ft and includes four simply supported spans. The controlling span for load rating is 25 ft and is 25 ft 4 in. wide, with a roadway width of 23 ft 4 in. The bridge carries two lanes, one in each direction, and has an ADT of 310 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 20.5 US tons and an operating gross loading of 34.4 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.31. Bridge CS-16 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.32. Bridge CS-16 Details (TxDOT 2018c)

Table C.16. Characteristics and Load Ratings of Bridge CS-16

Bridge Characteristics	
Year Built	1951
On/Off-System	On
ADT	310
Controlling Span Length	25'-0"
Deck Width	25'-4"
Roadway Width	23'-4"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	7/8 in. diam. @ 6.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 #11 in. bars T&B
Wearing Surface Thickness (in.)	0.5
TxDOT Inventory RF	0.57
TxDOT Operating RF	0.96
TxDOT Inventory Rating (HS)	11.4
TxDOT Operating Rating (HS)	19.1
Posted Axle Limit (lbs)	28,000

Rating Type	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.57	0.31	0.57	0.12
Operating	0.96	1.10	1.10	0.15

CS-16 Flexure Rating Factors

■ TxDOT (ASR) ■ ASR ■ LFR ■ LRFR

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.5	C_n (kip-ft/ft)	23.7	C_n (kip-ft/ft)	23.7
C_{OP} (kip-ft/ft)	18.8				
M_{DL} (kip-ft/ft)	12.1	M_{DL} (kip-ft/ft)	12.1	M_{DL} (kip-ft/ft)	12.1
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	47.8
Inventory HS20 RF	0.31	Inventory HS20 RF	0.56	Inventory HL93 RF	0.12
Operating HS20 RF	0.92	Operating HS20 RF	1.10	Operating HL93 RF	0.15
Inventory Rating (HS)	6.2	Inventory Rating (HS)	11.2		
Operating Rating (HS)	18.4	Operating Rating (HS)	22		

Notes:

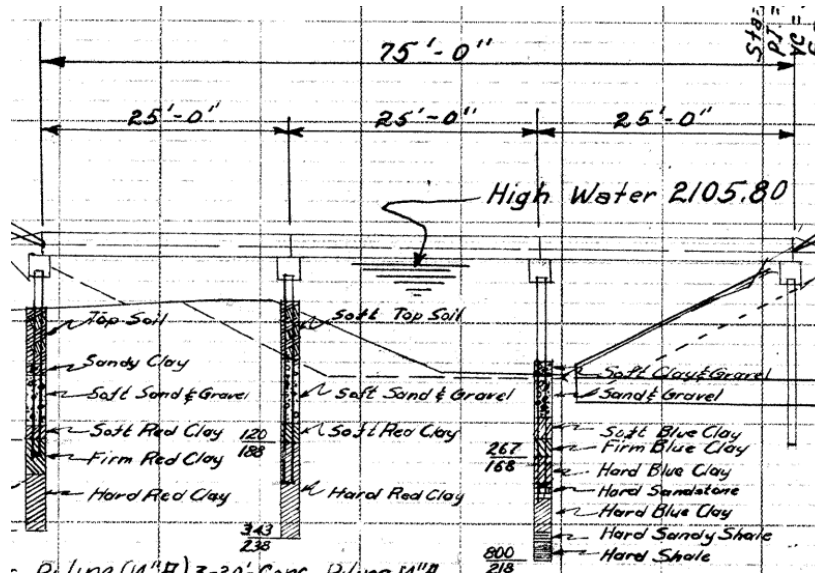
1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.17 Bridge CS-17

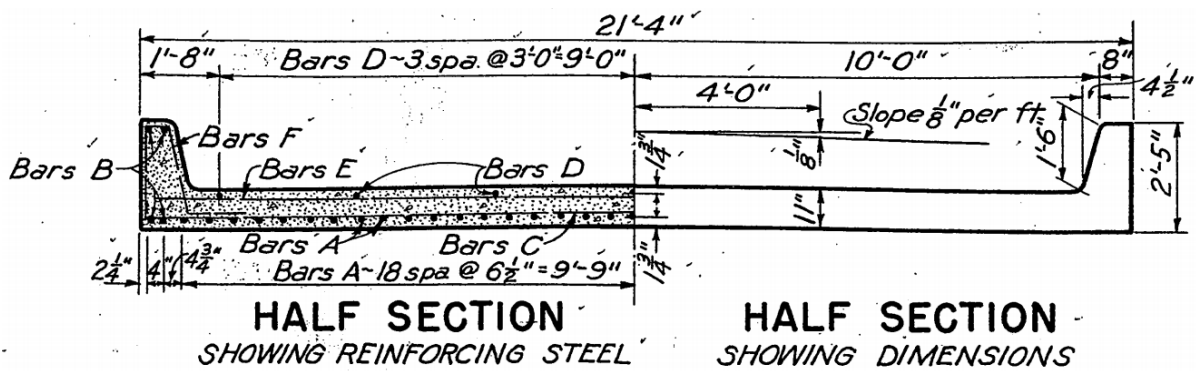
Bridge CS-17 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 7 (Good). The bridge has a total length of 75 ft and includes three simply supported spans. The controlling span for load rating is 25 ft and is 21 ft 4 in. wide, with a roadway width of 19 ft 5 in. The bridge carries two lanes, one in each direction, and has an ADT of 270 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 16.9 US tons and an operating gross loading of 35.5 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.33. Bridge CS-17 (TxDOT 2018c)



(a) Elevation View



(b) Cross-Section



(c) Underside Superstructure View

Figure C.34. Bridge CS-17 Details (TxDOT 2018c)

Table C.17. Characteristics and Load Ratings of Bridge CS-17

Bridge Characteristics	
Year Built	1952
On/Off-System	On
ADT	270
Controlling Span Length	25'-0"
Deck Width	21'-4"
Roadway Width	19'-5"
Slab Thickness (in.)	11.0
Top Reinforcement	1/2 in. diam. @ 36 in.
Bottom Reinforcement	7/8 in. diam. @ 6.5 in.
Curb	8 to 12.5 in. wide x 18 in. high
Curb Reinforcement	2 #11 in. bars T&B
Wearing Surface Thickness (in.)	2.0
TxDOT Inventory RF	0.47
TxDOT Operating RF	0.99
TxDOT Inventory Rating (HS)	9.4
TxDOT Operating Rating (HS)	19.7
Posted Axle Limit (lbs)	28,000

CS-17 Flexure Rating Factors

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.47	0.28	0.47	0.13
Operating	0.99	0.79	0.99	0.17

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	13.5	C_n (kip-ft/ft)	23.7	C_n (kip-ft/ft)	23.7
C_{OP} (kip-ft/ft)	18.8				
M_{DL} (kip-ft/ft)	13.7	M_{DL} (kip-ft/ft)	13.7	M_{DL} (kip-ft/ft)	13.7
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	50.6
Inventory HS20 RF	0.28	Inventory HS20 RF	0.47	Inventory HL93 RF	0.13
Operating HS20 RF	0.79	Operating HS20 RF	0.99	Operating HL93 RF	0.17
Inventory Rating (HS)	5.6	Inventory Rating (HS)	9.4		
Operating Rating (HS)	15.8	Operating Rating (HS)	19.8		

Notes:

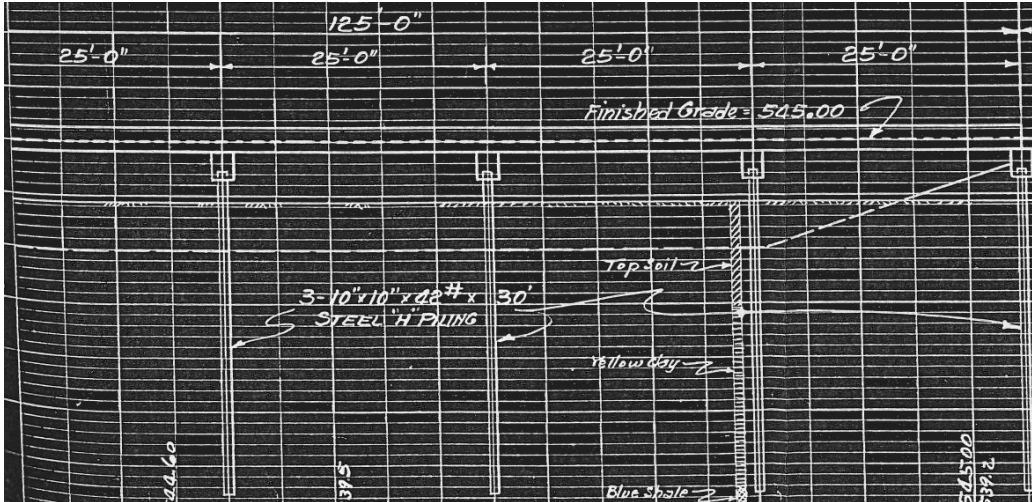
1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.18 Bridge CS-18

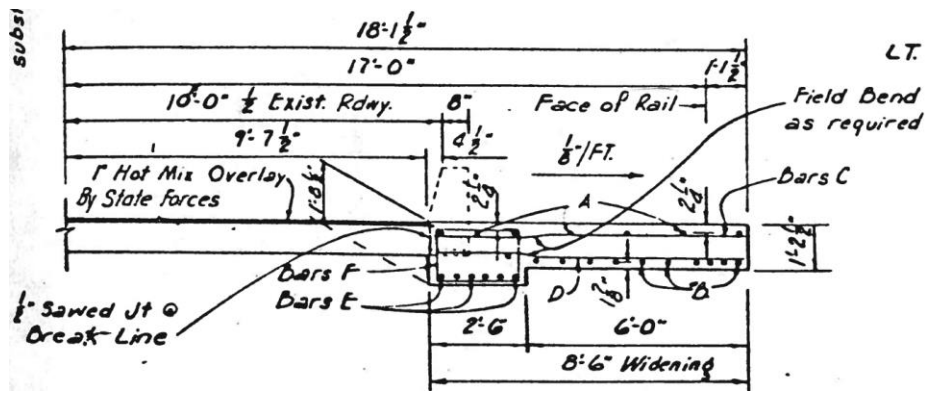
Bridge CS-18 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 125 ft and includes five simply supported spans. The controlling span for load rating is 25 ft and is 36 ft 4 in. wide, with a roadway width of 34 ft 2 in. The bridge carries two lanes, one in each direction, and has an ADT of 880 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 15.9 US tons and an operating gross loading of 27.3 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.35. Bridge CS-18 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.36. Bridge CS-18 Details (TxDOT 2018c)

Table C.18. Characteristics and Load Ratings of Bridge CS-18

Bridge Characteristics		CS-18 Flexure Rating Factors			
Year Built	1952				
On/Off-System	On				
ADT	880				
Controlling Span Length	25'-0"				
Deck Width	36'-4"				
Roadway Width	34'-2"				
Slab Thickness (in.)	11.0				
Top Reinforcement	7/8 in. diam. @ 12 in.				
Bottom Reinforcement	7/8 in. diam. @ 12 in.				
Beam	30 in. wide x 30 in. high				
Beam Reinforcement	6 #11 in. bars B				
Wearing Surface Thickness (in.)	1.5				
TxDOT Inventory RF	0.44				
TxDOT Operating RF	0.76				
TxDOT Inventory Rating (HS)	8.8				
TxDOT Operating Rating (HS)	15.1				
Posted Axle Limit (lbs)	28,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	7.6	C_n (kip-ft/ft)	13.3	C_n (kip-ft/ft)	13.3
C_{OP} (kip-ft/ft)	10.5				
M_{DL} (kip-ft/ft)	5.4	M_{DL} (kip-ft/ft)	5.4	M_{DL} (kip-ft/ft)	10.7
$M_{LL-HS20}$ (kip-ft/ft)	17.7	$M_{LL-HS20}$ (kip-ft/ft)	17.7	$M_{LL-HL93}$ (kip-ft/ft)	45.5
Inventory HS20 RF	0.12	Inventory HS20 RF	0.16	Inventory HL93 RF	0.08
Operating HS20 RF	0.29	Operating HS20 RF	0.27	Operating HL93 RF	0.11
Inventory Rating (HS)	2.4	Inventory Rating (HS)	3.2		
Operating Rating (HS)	5.8	Operating Rating (HS)	5.4		

Notes:

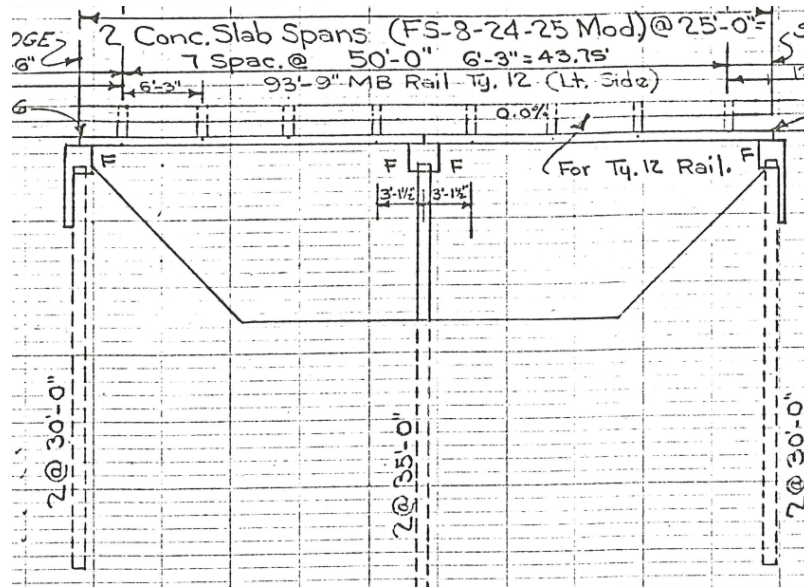
1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.

C.1.19 Bridge CS-19

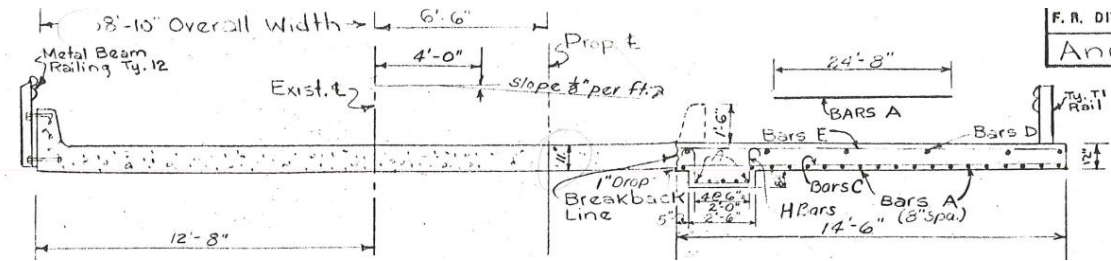
Bridge CS-19 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 50 ft and includes two simply supported spans. The controlling span for load rating is 25 ft and is 35 ft 4 in. wide, with a roadway width of 33 ft. The bridge carries two lanes, one in each direction, and has an ADT of 740 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 18.5 US tons and an operating gross loading of 34.7 US tons. The bridge is posted for a 28,000-lbs tandem axle.



Figure C.37. Bridge CS-19 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.38. Bridge CS-19 Details (TxDOT 2018c)

Table C.19. Characteristics and Load Ratings of Bridge CS-19

Bridge Characteristics		CS-19 Flexure Rating Factors			
Year Built	1953				
On/Off-System	On				
ADT	740				
Controlling Span Length	25'-0"				
Deck Width	35'-4"				
Roadway Width	33'-0"				
Slab Thickness (in.)	11.0				
Top Reinforcement	1/2 in. diam. @ 34 in.				
Bottom Reinforcement	1 1/4 in. diam. @ 8 in.				
Curb/Beam	Unknown				
Curb/Beam Reinforcement	Unknown				
Wearing Surface Thickness (in.)	0.5				
TxDOT Inventory RF	0.51				
TxDOT Operating RF	0.97				
TxDOT Inventory Rating (HS)	10.3				
TxDOT Operating Rating (HS)	19.3				
Posted Axle Limit (lbs)	28,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	16.9	C_n (kip-ft/ft)	37.8	C_n (kip-ft/ft)	37.8
C_{OP} (kip-ft/ft)	31				
M_{DL} (kip-ft/ft)	11.6	M_{DL} (kip-ft/ft)	11.6	M_{DL} (kip-ft/ft)	11.6
$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HS20}$ (kip-ft/ft)	23	$M_{LL-HL93}$ (kip-ft/ft)	45
Inventory HS20 RF	0.28	Inventory HS20 RF	0.86	Inventory HL93 RF	0.31
Operating HS20 RF	1.05	Operating HS20 RF	1.44	Operating HL93 RF	0.40
Inventory Rating (HS)	5.6	Inventory Rating (HS)	17.2		
Operating Rating (HS)	21.0	Operating Rating (HS)	28.8		

Notes:

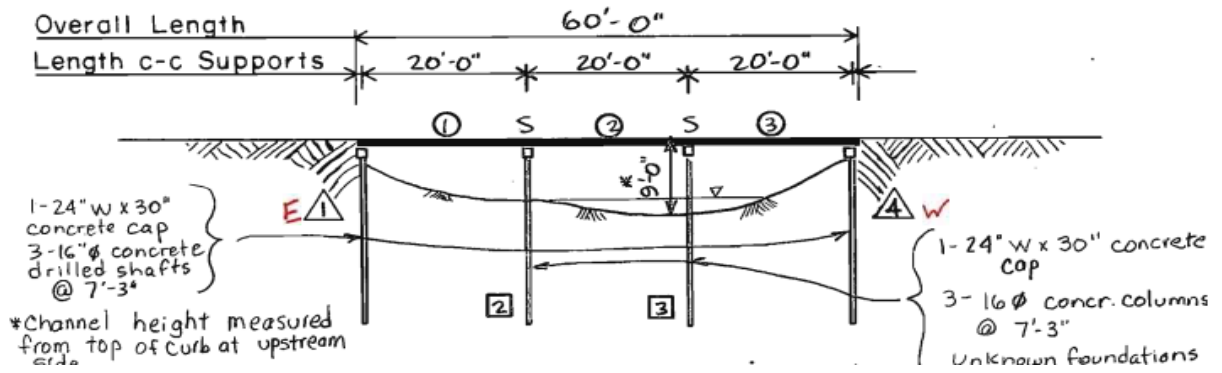
1. $f'_c = 2.5$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. Curb is structural. Exterior strip controls for ASR and LFR ratings.

C.1.20 Bridge CS-20

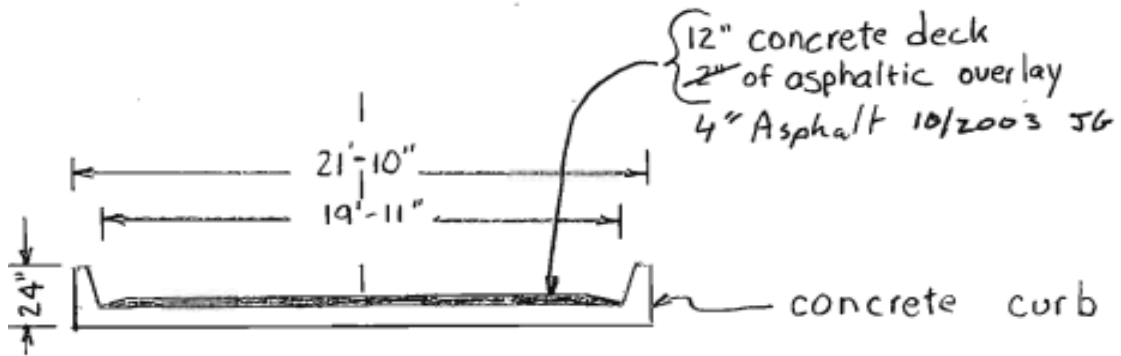
Bridge CS-20 has a deck condition rating of 7 (Good), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 5 (Fair). The bridge has a total length of 60 ft and includes three simply supported spans. The controlling span for load rating is 20 ft and is 21 ft 10 in. wide, with a roadway width of 19 ft 11 in. The bridge carries two lanes, one in each direction, and has an ADT of 60 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 20.7 US tons and an operating gross loading of 43.2 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure C.39. Bridge CS-20 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.40. Bridge CS-20 Details (TxDOT 2018c)

Table C.20. Characteristics and Load Ratings of Bridge CS-20

Bridge Characteristics		CS-20 Flexure Rating Factors			
Year Built	1960				
On/Off-System	Off				
ADT	320				
Controlling Span Length	25'-0"				
Deck Width	25'-4"				
Roadway Width	23'-4"				
Slab Thickness (in.)	14.0				
Top Reinforcement	N/A				
Bottom Reinforcement	Unknown				
Curb	Unknown				
Curb Reinforcement	Unknown				
Wearing Surface Thickness (in.)	Unknown				
TxDOT Inventory RF	0.75				
TxDOT Operating RF	1.00				
TxDOT Inventory Rating (HS)	15.0				
TxDOT Operating Rating (HS)	20.0				
Posted Axle Limit (lbs)	24,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	-	C_n (kip-ft/ft)	-	C_n (kip-ft/ft)	-
C_{OP} (kip-ft/ft)	-				
M_{DL} (kip-ft/ft)	8.9	M_{DL} (kip-ft/ft)	8.9	M_{DL} (kip-ft/ft)	8.9
$M_{LL-HS20}$ (kip-ft/ft)	19.2	$M_{LL-HS20}$ (kip-ft/ft)	19.2	$M_{LL-HL93}$ (kip-ft/ft)	30.2
Inventory HS20 RF	-	Inventory HS20 RF	-	Inventory HL93 RF	-
Operating HS20 RF	-	Operating HS20 RF	-	Operating HL93 RF	-
Inventory Rating (HS)	-	Inventory Rating (HS)	-		
Operating Rating (HS)	-	Operating Rating (HS)	-		

Notes:

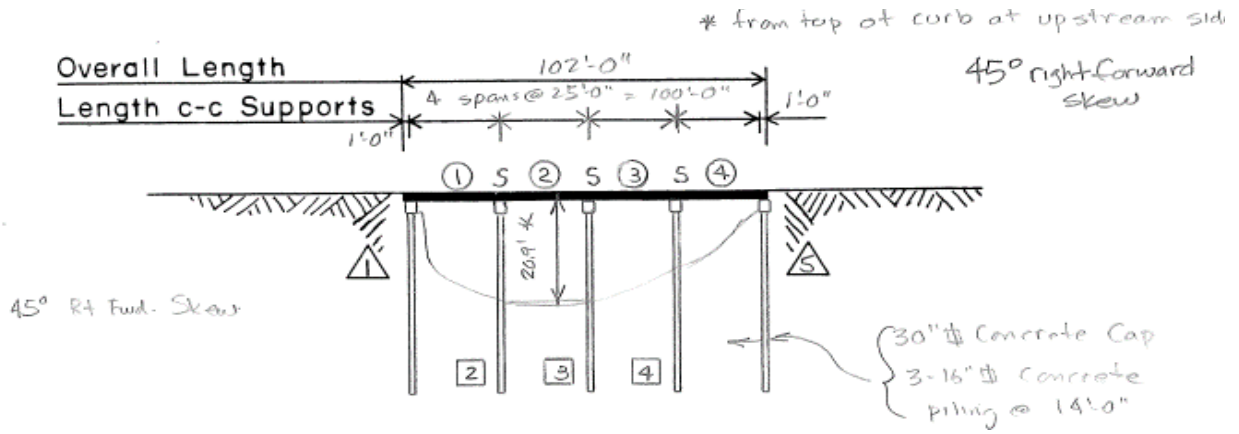
1. $f'_c = 2.5$ ksi per AASHTO MBE.
2. $f_y = 33$ ksi per AASHTO MBE.

C.1.21 Bridge CS-21

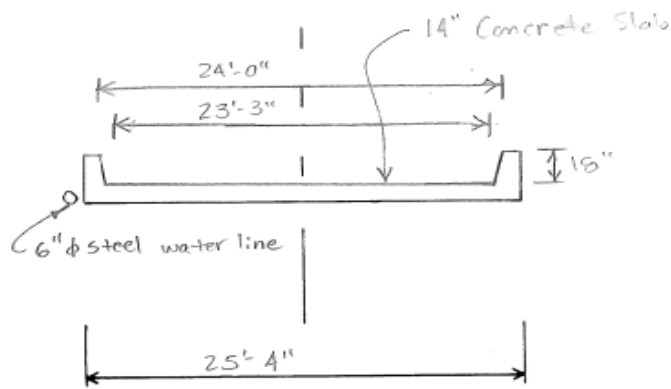
Bridge CS-21 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 5 (Fair). The bridge has a total length of 102 ft and includes four simply supported spans. The controlling span for load rating is 25 ft and is 25 ft 4 in. wide, with a roadway width of 23 ft 4 in. The bridge carries two lanes, one in each direction, and has an ADT of 320 vehicles. The CS and substructure control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure C.41. Bridge CS-21 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.42. Bridge CS-21 Details (TxDOT 2018c)

Table C.21. Characteristics and Load Ratings of Bridge CS-21

Bridge Characteristics		CS-21 Flexure Rating Factors			
Year Built	1960				
On/Off-System	Off				
ADT	320				
Controlling Span Length	25'-0"				
Deck Width	25'-4"				
Roadway Width	23'-4"				
Slab Thickness (in.)	14.0				
Top Reinforcement	N/A				
Bottom Reinforcement	Unknown				
Curb	Unknown				
Curb Reinforcement	Unknown				
Wearing Surface Thickness (in.)	Unknown				
TxDOT Inventory RF	0.75				
TxDOT Operating RF	1.00				
TxDOT Inventory Rating (HS)	15.0				
TxDOT Operating Rating (HS)	20.0				
Posted Axle Limit (lbs)	24,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	-	C_n (kip-ft/ft)	-	C_n (kip-ft/ft)	-
C_{OP} (kip-ft/ft)	-				
M_{DL} (kip-ft/ft)	6.3	M_{DL} (kip-ft/ft)	6.3	M_{DL} (kip-ft/ft)	12.6
$M_{LL-HS20}$ (kip-ft/ft)	17.6	$M_{LL-HS20}$ (kip-ft/ft)	17.6	$M_{LL-HL93}$ (kip-ft/ft)	99.5
Inventory HS20 RF	-	Inventory HS20 RF	-	Inventory HL93 RF	-
Operating HS20 RF	-	Operating HS20 RF	-	Operating HL93 RF	-
Inventory Rating (HS)	-	Inventory Rating (HS)	-		
Operating Rating (HS)	-	Operating Rating (HS)	-		

Notes:

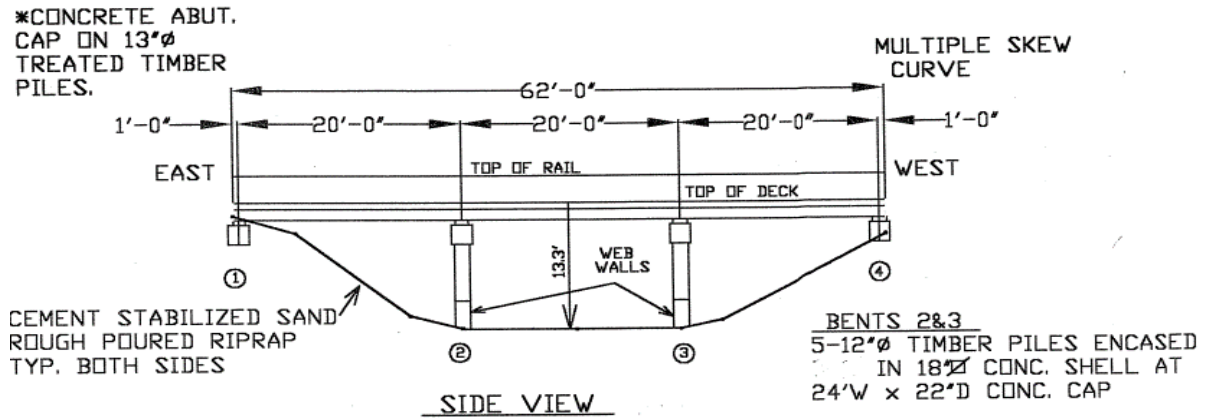
1. $f'_c = 3$ ksi per AASHTO MBE.
2. $f_y = 33$ ksi per AASHTO MBE.

C.1.22 Bridge CS-22

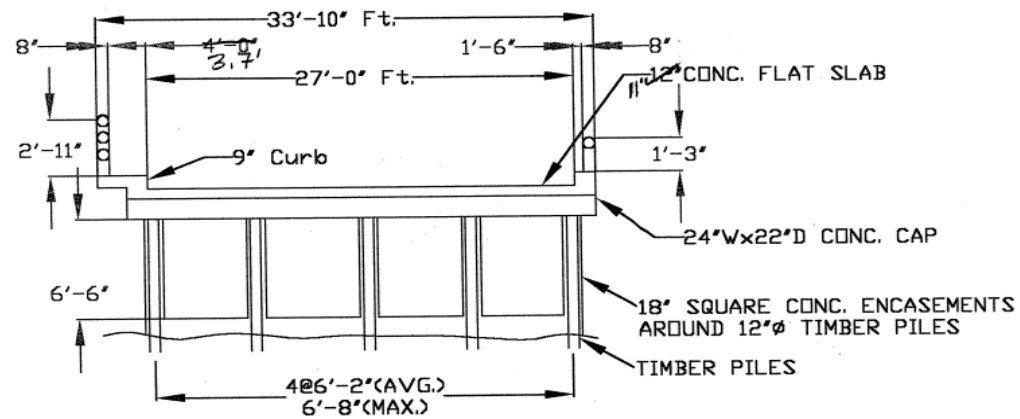
Bridge CS-22 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 62 ft and includes three simply supported spans. The controlling span for load rating is 20 ft and is 33 ft 10 in. wide, with a roadway width of 27 ft. The bridge carries two lanes, one in each direction, and has an ADT of 270 vehicles. The CS controls the rating of the bridge, which has an inventory gross loading of 18 US tons and an operating gross loading of 27 US tons. The bridge is posted for a 15,000-lbs axle.



Figure C.43. Bridge CS-22 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.44. Bridge CS-22 Details (TxDOT 2018c)

Table C.22. Characteristics and Load Ratings of Bridge CS-22

Bridge Characteristics		CS-22 Flexure Rating Factors			
Year Built	1962				
On/Off-System	Off				
ADT	270				
Controlling Span Length	20'-0"				
Deck Width	33'-10"				
Roadway Width	27'-0"				
Slab Thickness (in.)	11.0				
Top Reinforcement	N/A				
Bottom Reinforcement	Unknown				
Curb	36 in. x 9 in. high				
Curb Reinforcement	Unknown				
Wearing Surface Thickness (in.)	5.0				
TxDOT Inventory RF	0.50				
TxDOT Operating RF	0.75				
TxDOT Inventory Rating (HS)	10.0				
TxDOT Operating Rating (HS)	15.0				
Posted Axle Limit (lbs)	15,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft/ft)	-	C_n (kip-ft/ft)	-	C_n (kip-ft/ft)	-
C_{OP} (kip-ft/ft)	-				
M_{DL} (kip-ft/ft)	0.2	M_{DL} (kip-ft/ft)	0.2	M_{DL} (kip-ft/ft)	8.9
$M_{LL-HS20}$ (kip-ft/ft)	-	$M_{LL-HS20}$ (kip-ft/ft)	-	$M_{LL-HL93}$ (kip-ft/ft)	76.2
Inventory HS20 RF	-	Inventory HS20 RF	-	Inventory HL93 RF	-
Operating HS20 RF	-	Operating HS20 RF	-	Operating HL93 RF	-
Inventory Rating (HS)	-	Inventory Rating (HS)	-		
Operating Rating (HS)	-	Operating Rating (HS)	-		

Notes:

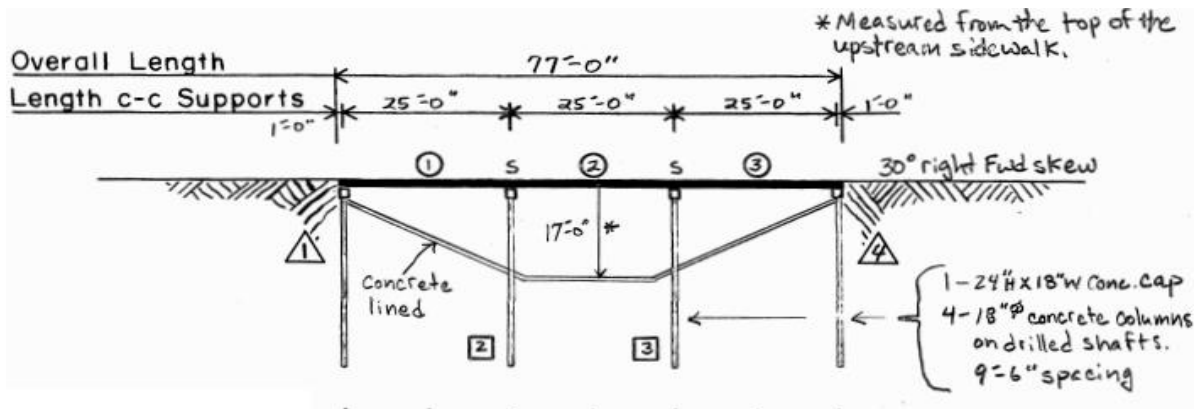
1. $f'_c = 3$ ksi per AASHTO MBE.
2. $f_y = 33$ ksi per AASHTO MBE.

C.1.23 Bridge CS-23

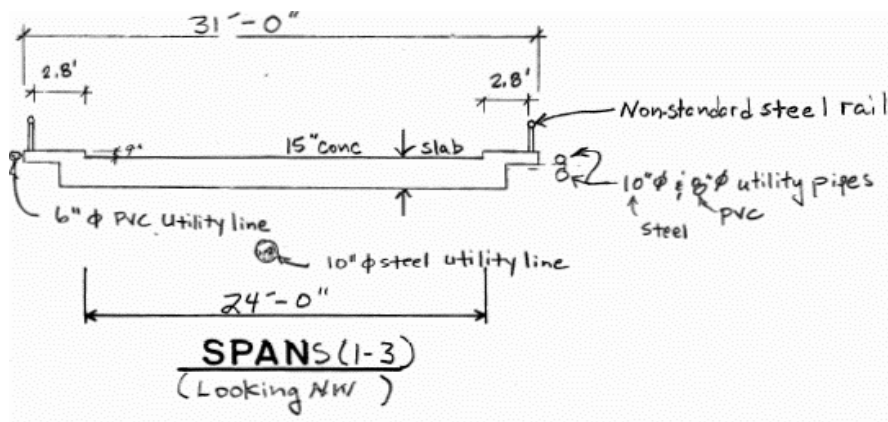
Bridge CS-23 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 5 (Fair). The bridge has a total length of 77 ft and includes three simply supported spans. The controlling span for load rating is 25 ft and is 31 ft wide, with a roadway width of 24 ft. The bridge carries two lanes, one in each direction, and has an ADT of 1980 vehicles. The CS and substructure control the rating of the bridge, which has an inventory gross loading of 18 US tons and an operating gross loading of 27 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure C.45. Bridge CS-23 (TxDOT 2018c)



(a) Elevation View



(b) Transverse Section



(c) Underside Superstructure View

Figure C.46. Bridge CS-23 Details (TxDOT 2018c)

Table C.23. Characteristics and Load Ratings of Bridge CS-23

Bridge Characteristics		CS-23 Flexure Rating Factors									
Year Built	1970	<p>CS-23 Flexure Rating Factors</p> <p>Legend: TxDOT (ASR)</p> <table border="1"> <thead> <tr> <th>Category</th> <th>TxDOT (ASR)</th> </tr> </thead> <tbody> <tr> <td>Inventory</td> <td>0.75</td> </tr> <tr> <td>Operating</td> <td>1.00</td> </tr> </tbody> </table>				Category	TxDOT (ASR)	Inventory	0.75	Operating	1.00
Category	TxDOT (ASR)										
Inventory	0.75										
Operating	1.00										
On/Off-System	Off										
ADT	1980										
Controlling Span Length	25'-0"										
Deck Width	31'-0"										
Roadway Width	24'-0"										
Slab Thickness (in.)	15.0										
Top Reinforcement	N/A										
Bottom Reinforcement	Unknown										
Curb	36 in. x 9 in. high										
Curb Reinforcement	Unknown										
Wearing Surface Thickness (in.)	5.0										
TxDOT Inventory RF	0.75										
TxDOT Operating RF	1.00										
TxDOT Inventory Rating (HS)	10.0										
TxDOT Operating Rating (HS)	15.0										
Posted Axle Limit (lbs)	21,000										
Calculated Values for RFs											
ASR		LFR		LRFR							
C_{IN} (kip-ft)	-	C_n (kip-ft)	-	C_n (kip-ft)	-						
C_{OP} (kip-ft)	-										
M_{DL} (kip-ft)	13.4	M_{DL} (kip-ft)	13.4	M_{DL} (kip-ft)	17.8						
$M_{LL-HS20}$ (kip-ft)	20.8	$M_{LL-HS20}$ (kip-ft)	20.8	$M_{LL-HL93}$ (kip-ft)	93.9						
Inventory HS20 RF	-	Inventory HS20 RF	-	Inventory HL93 RF	-						
Operating HS20 RF	-	Operating HS20 RF	-	Operating HL93 RF	-						
Inventory Rating (HS)	-	Inventory Rating (HS)	-								
Operating Rating (HS)	-	Operating Rating (HS)	-								

Notes:

1. $f'_c = 3$ ksi per AASHTO MBE.
2. $f_y = 33$ ksi per AASHTO MBE.

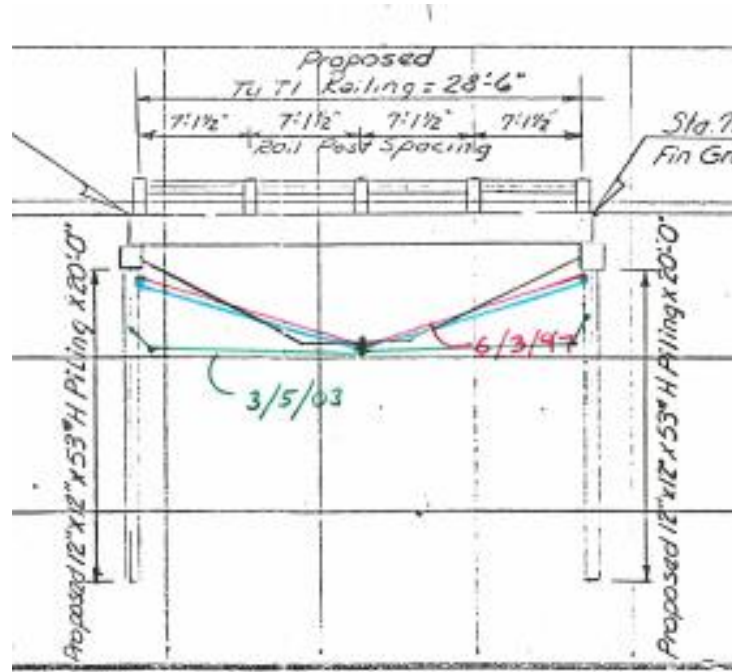
C.2 CONCRETE MULTI-GIRDER BRIDGES

C.2.1 Bridge CM-1

Bridge CM-1 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 7 (Good). The bridge has a total length of 30 ft, which is one simply supported span. The controlling span for load rating is 30 ft and is 35 ft 6 in. wide, with a roadway width of 33 ft 6 in. The bridge carries two lanes, one in each direction, and has an ADT of 170 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 18 US tons and an operating gross loading of 30 US tons. The bridge is posted for a 20,000-lbs single axle, a 28,000-lbs tandem axle, a 48,000-lbs single vehicle, and an 80,000-lbs combination vehicle.



Figure C.47. Bridge CM-1 (TxDOT 2018c)



(a) Elevation View



(b) Underside Superstructure View

Figure C.48. Bridge CM-1 Details (TxDOT 2018c)

Table C.24. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-1

Bridge Characteristics	
Year Built	1942
On/Off-System	On
ADT	170
Max Span Length	30'
Deck Width	35'-6"
Roadway Width	33'-6"
Slab + Beam Depth (in.)	24.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	0.5
TxDOT Inventory RF	0.50
TxDOT Operating RF	0.83
TxDOT Inventory Rating (HS)	9.9
TxDOT Operating Rating (HS)	16.6
Posted Axle Limit (lbs)	see note 1

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	0.50	0.45	0.50	0.28
Operating	0.83	0.87	0.83	0.36

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft)	92.4	C_n (kip-ft)	162	C_n (kip-ft)	-
C_{OP} (kip-ft)	128.3				
M_{DL} (kip-ft)	52.7	M_{DL} (kip-ft)	52.7	M_{DL} (kip-ft)	
$M_{LL-HS20}$ (kip-ft)	86.8	$M_{LL-HS20}$ (kip-ft)	86.8	$M_{LL-HL93}$ (kip-ft)	
Inventory HS20 RF	0.46	Inventory HS20 RF	0.50	Inventory HL93 RF	-
Operating HS20 RF	0.87	Operating HS20 RF	0.83	Operating HL93 RF	-
Inventory Rating (HS)	9.2	Inventory Rating (HS)	10		
Operating Rating (HS)	17.4	Operating Rating (HS)	16.6		

Notes:

1. Posting restrictions include a 20,000-lbs single axle, a 28,000-lbs tandem axle, a 48,000-lbs single vehicle and a 80,000-lbs combination vehicle.
2. $f'_c = 2.5$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.
3. $f_y = 33$ ksi per TxDOT Load Rating Calculations and AASHTO MBE.

C.2.2 Bridge CM-2

Bridge CM-2 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 5 (Fair). The bridge has a total length of 60 ft, consisting of two simply supported spans. The controlling span for load rating is 29 ft and is 21 ft 7.5 in. wide, with a roadway width of 21 ft. The bridge carries two lanes, one in each direction, and has an ADT of 100 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 20,000-lbs single axle, a 28,000-lbs tandem axle, a 48,000-lbs single vehicle, and an 80,000-lbs combination vehicle.



Figure C.49. Bridge CM-2 (TxDOT 2018c)



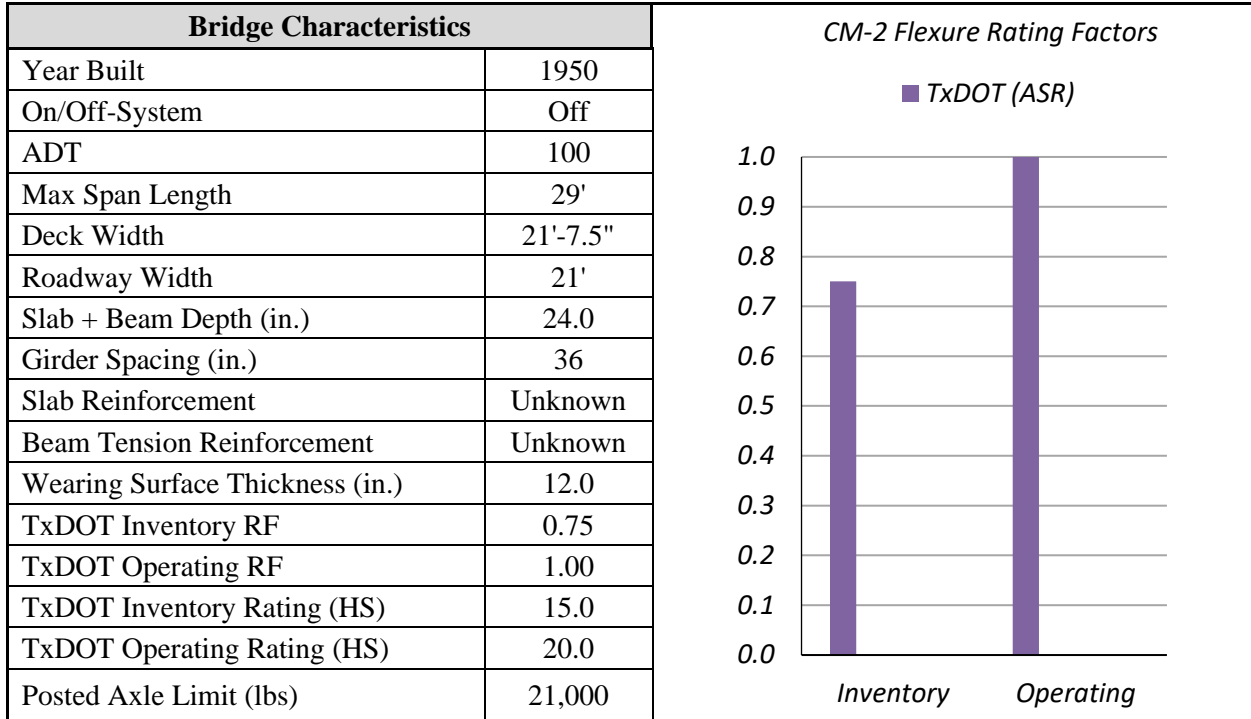
(b) Elevation View



(b) Underside Superstructure View

Figure C.50. Bridge CM-2 Details (TxDOT 2018c)

Table C.25. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-2



Notes:

1. $f'_c = 2.5$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.3 Bridge CM-3

Bridge CM-3 has a deck condition rating of 7 (Good), a superstructure condition rating of 5 (Fair), and a substructure condition rating of 5 (Fair). The bridge has a total length of 60 ft, consisting of two simply supported spans. The controlling span for load rating is 29 ft and is 21 ft 8 in. wide, with a roadway width of 21 ft. The bridge carries two lanes, one in each direction, and has an ADT of 250 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.

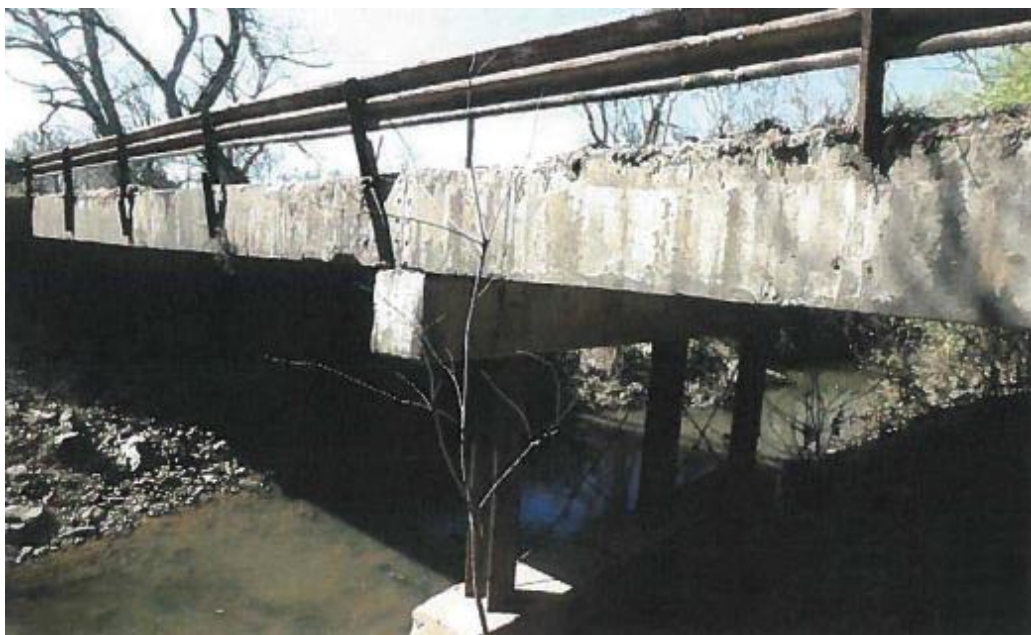
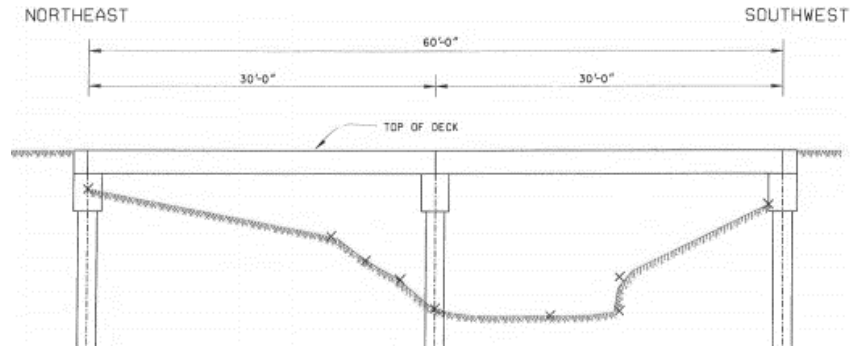
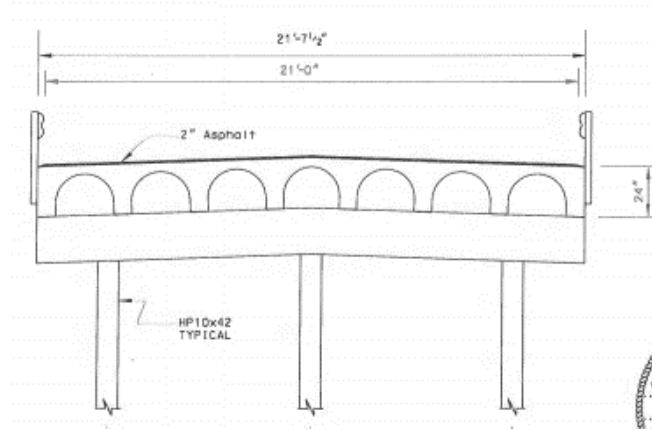


Figure C.51. Bridge CM-3 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.52. Bridge CM-3 Details (TxDOT 2018c)

Table C.26. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-3

Bridge Characteristics	
Year Built	1950
On/Off-System	Off
ADT	250
Max Span Length	29'-0"
Deck Width	21'-8"
Roadway Width	21'-0"
Slab + Beam Depth (in.)	24.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	2.0
TxDOT Inventory RF	0.75
TxDOT Operating RF	1.00
TxDOT Inventory Rating (HS)	15.0
TxDOT Operating Rating (HS)	20.0
Posted Axle Limit (lbs)	24,000

CM-3 Flexure Rating Factors

Condition	TxDOT (ASR)
Inventory	0.75
Operating	1.00

Notes:

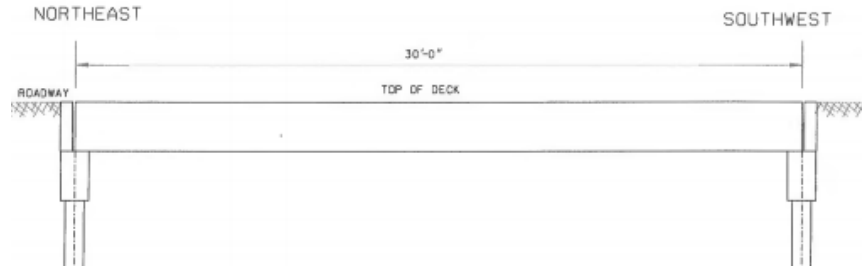
1. $f'_c = 2.5$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.4 Bridge CM-4

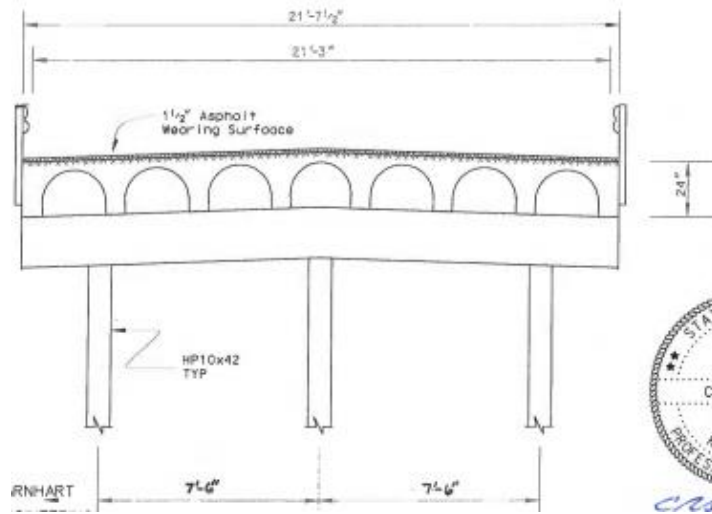
Bridge CM-4 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 5 (Fair). The bridge has a total length of 30 ft, which is one simply supported span. The controlling span for load rating is 29 ft and is 21 ft 8 in. wide, with a roadway width of 21 ft. The bridge carries two lanes, one in each direction, and has an ADT of 250 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure C.53. Bridge CM-4 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.54. Bridge CM-4 Details (TxDOT 2018c)

Table C.27. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-4

Bridge Characteristics		CM-4 Flexure Rating Factors			
Year Built	1950				
On/Off-System	Off				
ADT	250				
Max Span Length	29'-0"				
Deck Width	21'-8"				
Roadway Width	21'-0"				
Slab + Beam Depth (in.)	24.0				
Girder Spacing (in.)	36				
Slab Reinforcement	Unknown				
Beam Tension Reinforcement	Unknown				
Wearing Surface Thickness (in.)	1.5				
TxDOT Inventory RF	0.75				
TxDOT Operating RF	1.00				
TxDOT Inventory Rating (HS)	15.0				
TxDOT Operating Rating (HS)	20.0				
Posted Axle Limit (lbs)	24,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft)	-	C_n (kip-ft)	-	C_n (kip-ft)	-
C_{OP} (kip-ft)	-				
M_{DL} (kip-ft)		M_{DL} (kip-ft)		M_{DL} (kip-ft)	
$M_{LL-HS20}$ (kip-ft)		$M_{LL-HS20}$ (kip-ft)		$M_{LL-HL93}$ (kip-ft)	
Inventory HS20 RF	-	Inventory HS20 RF	-	Inventory HL93 RF	-
Operating HS20 RF	-	Operating HS20 RF	-	Operating HL93 RF	-
Inventory Rating (HS)	-	Inventory Rating (HS)	-		
Operating Rating (HS)	-	Operating Rating (HS)	-		

Notes:

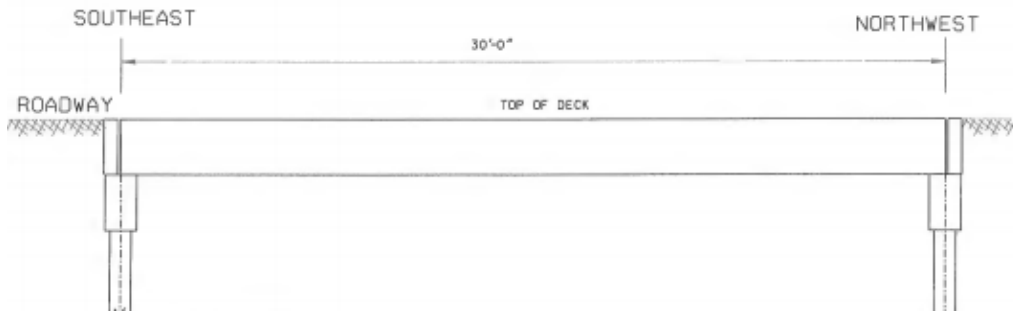
1. $f'_c = 2.5$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.5 Bridge CM-5

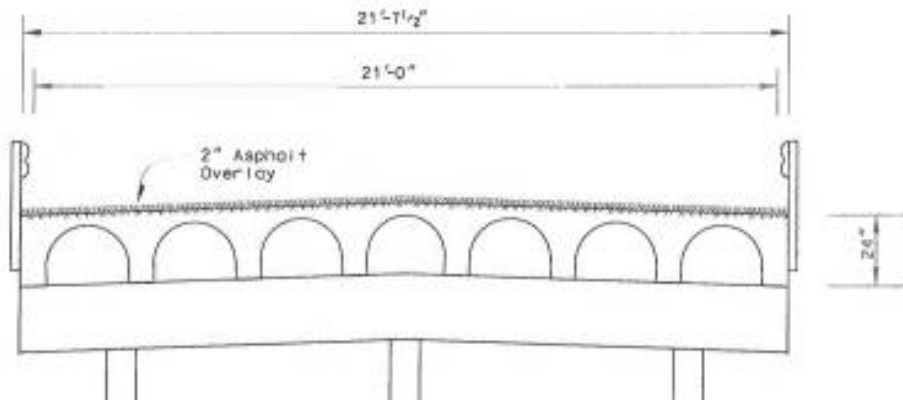
Bridge CM-5 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 5 (Fair). The bridge has a total length of 30 ft, which is one simply supported span. The controlling span for load rating is 29 ft and is 21 ft 8 in. wide, with a roadway width of 21 ft. The bridge carries two lanes, one in each direction, and has an ADT of 150 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 26 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 24,000-lbs tandem axle.



Figure C.55. Bridge CM-5 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.56. Bridge CM-5 Details (TxDOT 2018c)

Table C.28. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-5

Bridge Characteristics	
Year Built	1950
On/Off-System	Off
ADT	150
Max Span Length	29'-0"
Deck Width	21'-8"
Roadway Width	21'-0"
Slab + Beam Depth (in.)	24.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	2.0
TxDOT Inventory RF	0.75
TxDOT Operating RF	1.00
TxDOT Inventory Rating (HS)	14.3
TxDOT Operating Rating (HS)	20.0
Posted Axle Limit (lbs)	24,000

CM-5 Flexure Rating Factors

State	TxDOT (ASR)
Inventory	0.75
Operating	1.00

Notes:

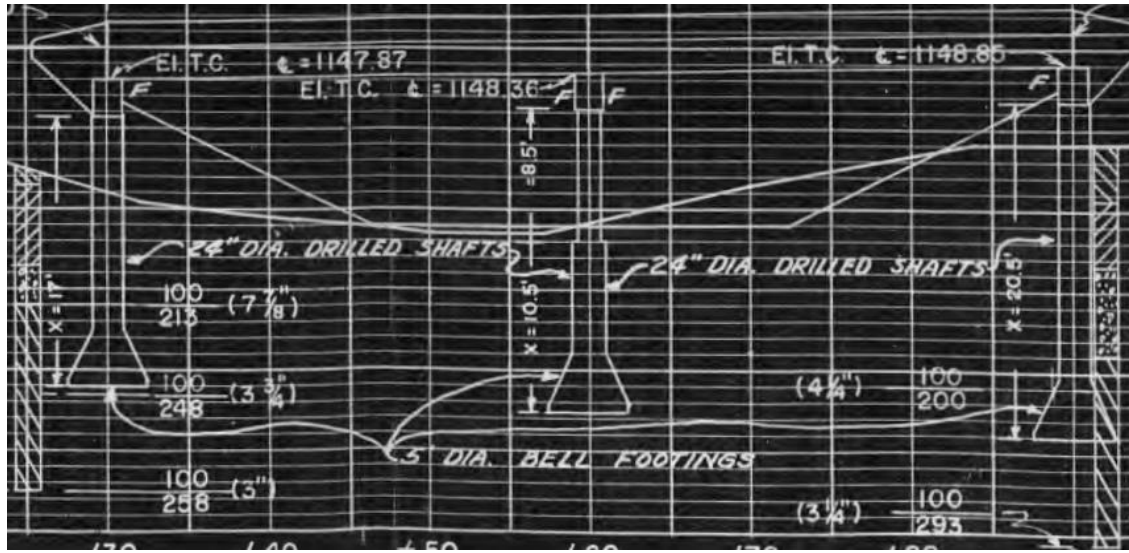
1. $f'_c = 2.5$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.6 Bridge CM-6

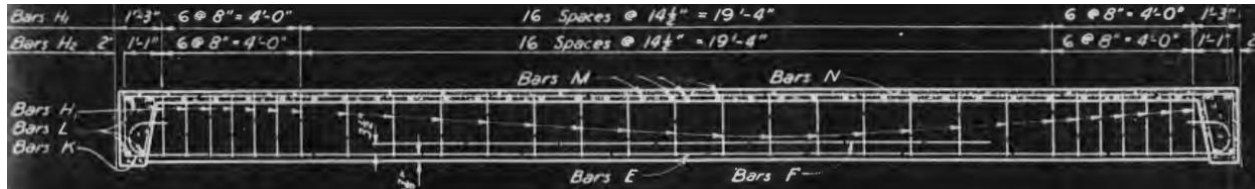
Bridge CM-6 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 7 (Good), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 60 ft and includes two simply supported spans. The controlling span for load rating is 30 ft and is 24 ft 8 in. wide, with a roadway width of 22 ft 8 in. The bridge carries two lanes, one in each direction, and has an ADT of 180 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 36 US tons and an operating gross loading of 49 US tons. The bridge is posted for a 20,000-lbs tandem axle.



Figure C.57. Bridge CM-6 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.58. Bridge CM-6 Details (TxDOT 2018c)

Table C.29. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-6

Bridge Characteristics					
Year Built	1957				
On/Off-System	On				
ADT	180				
Max Span Length	30'-0"				
Deck Width	24'-8"				
Roadway Width	22'-8"				
Slab + Beam Depth (in.)	24.0				
Girder Spacing (in.)	36				
Slab Reinforcement	Unknown				
Beam Tension Reinforcement	2-#6 and 2-#11				
Wearing Surface Thickness (in.)	1.5				
TxDOT Inventory RF	1.00				
TxDOT Operating RF	1.35				
TxDOT Inventory Rating (HS)	19.9				
TxDOT Operating Rating (HS)	33.2				
Posted Axle Limit (lbs)	20,000				
Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft)	98.4	C_n (kip-ft)	255.4	C_n (kip-ft)	255.4
C_{OP} (kip-ft)	137.8				
M_{DL} (kip-ft)	53.0	M_{DL} (kip-ft)	53.0	M_{DL} (kip-ft)	53.0
$M_{LL-HS20}$ (kip-ft)	86.8	$M_{LL-HS20}$ (kip-ft)	86.8	$M_{LL-HL93}$ (kip-ft)	196
Inventory HS20 RF	0.52	Inventory HS20 RF	0.99	Inventory HL93 RF	0.55
Operating HS20 RF	0.98	Operating HS20 RF	1.65	Operating HL93 RF	0.71
Inventory Rating (HS)	10.4	Inventory Rating (HS)	19.8		
Operating Rating (HS)	19.6	Operating Rating (HS)	33		

Notes:

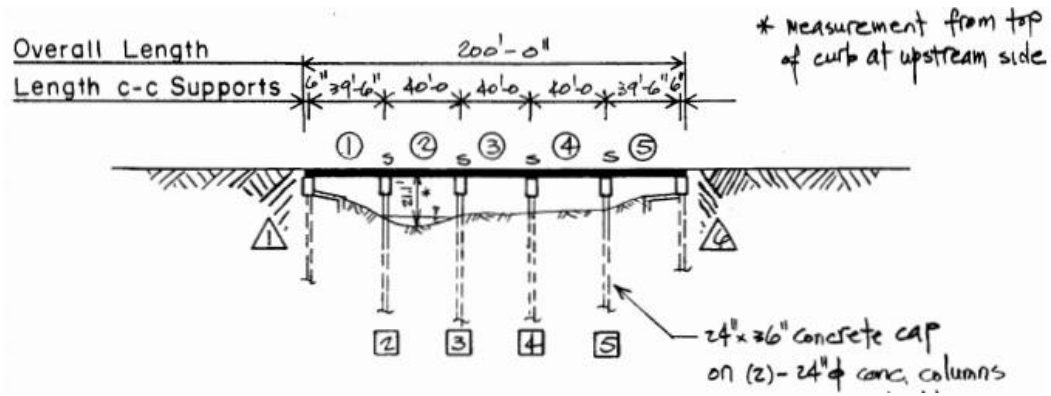
1. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
2. $f_y = 40$ ksi per TxDOT Load Rating Calculations.

C.2.7 Bridge CM-7

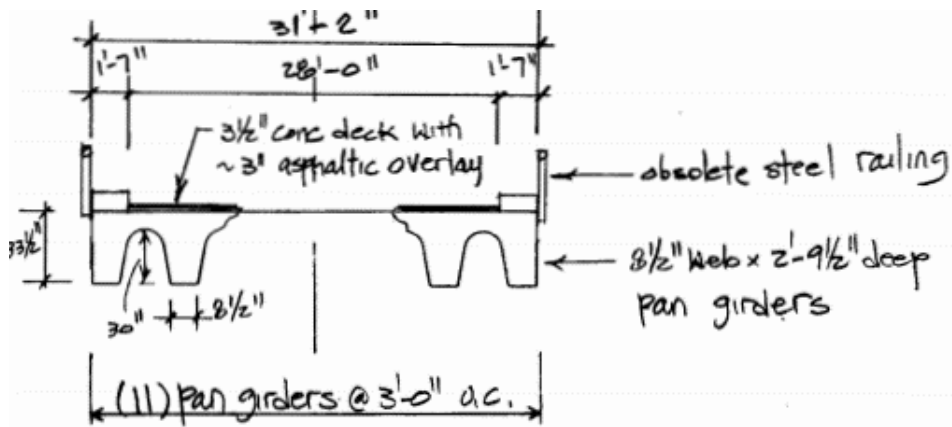
Bridge CM-7 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 5 (Fair), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 200 ft and includes five simply supported spans. The controlling span for load rating is 40 ft and is 31 ft 2 in. wide, with a roadway width of 28 ft. The bridge carries two lanes, one in each direction, and has an ADT of 2700 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 15,000-lbs axle or tandem axle and a 28,000-lbs gross vehicle.



Figure C.59. Bridge CM-7 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section

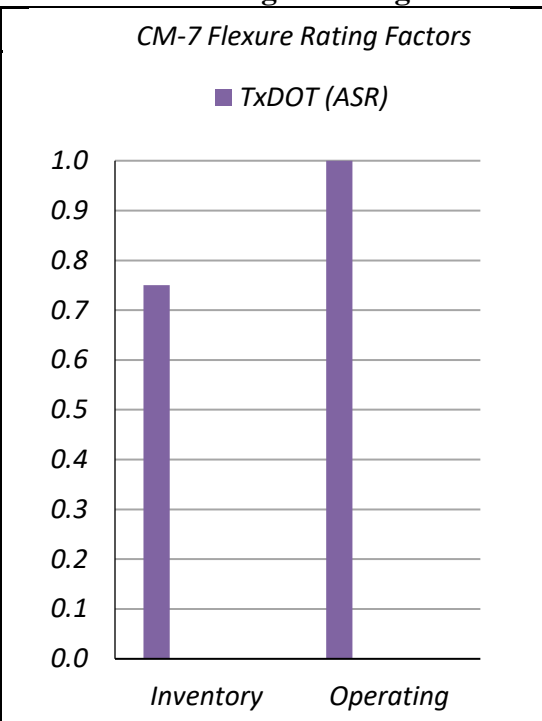


(c) Underside Superstructure View

Figure C.60. Bridge CM-7 Details (TxDOT 2018c)

Table C.30. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-7

Bridge Characteristics	
Year Built	1957
On/Off-System	Off
ADT	2700
Max Span Length	40'-0"
Deck Width	31'-2"
Roadway Width	28'-0"
Slab + Beam Depth (in.)	33.5
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	0.75
TxDOT Operating RF	1.00
TxDOT Inventory Rating (HS)	Unknown
TxDOT Operating Rating (HS)	Unknown
Posted Axle Limit (lbs)	see note 1



Notes:

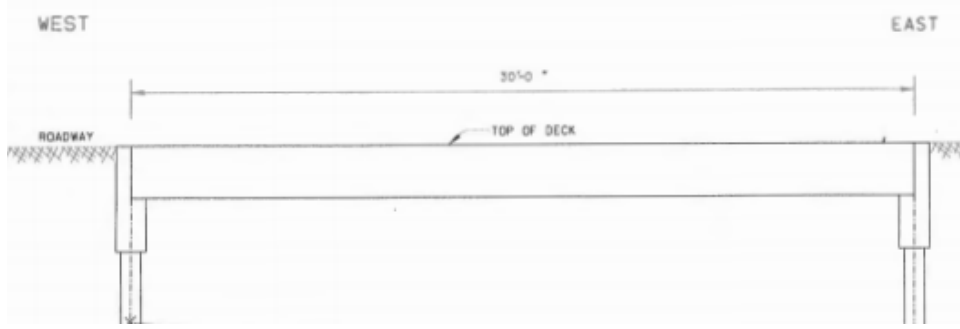
1. Posting restrictions include a 15,000-lbs single axle or tandem axle and a 28,000-lbs gross.
2. $f'_c = 2.5$ ksi per TxDOT AASHTO MBE.
3. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.8 Bridge CM-8

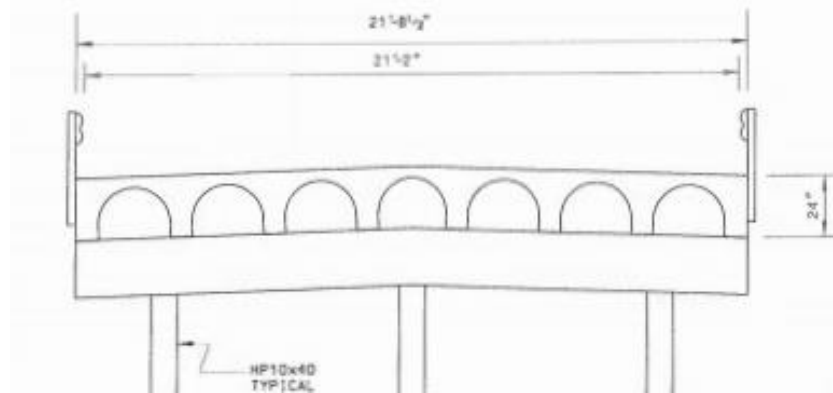
Bridge CM-8 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 5 (Fair). The bridge has a total length of 30 ft, which is one simply supported span. The controlling span for load rating is 29 ft and is 21 ft 9 in. wide, with a roadway width of 21 ft 2 in. The bridge carries two lanes, one in each direction, and has an ADT of 250 vehicles. The steel piles control the rating of the bridge, which has an inventory gross loading of 25 US tons and an operating gross loading of 34 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure C.61. Bridge CM-8 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section

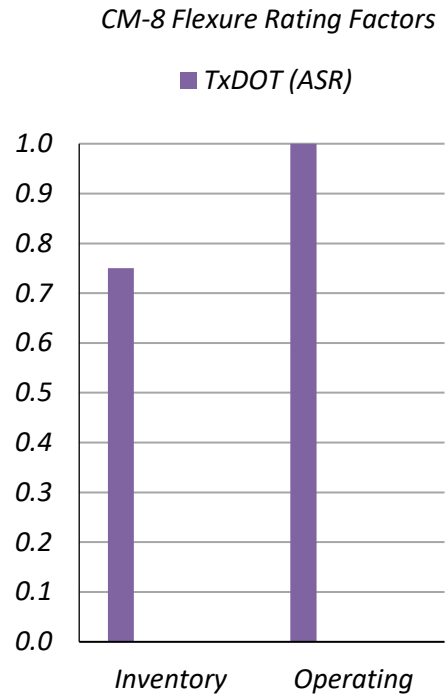


(c) Underside Superstructure View

Figure C.62. Bridge CM-8 Details (TxDOT 2018c)

Table C.31. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-8

Bridge Characteristics	
Year Built	1960
On/Off-System	Off
ADT	250
Max Span Length	29'-0"
Deck Width	21'-9"
Roadway Width	21'-2"
Slab + Beam Depth (in.)	24.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	0.75
TxDOT Operating RF	1.00
TxDOT Inventory Rating (HS)	13.8
TxDOT Operating Rating (HS)	18.9
Posted Axle Limit (lbs)	21,000



Notes:

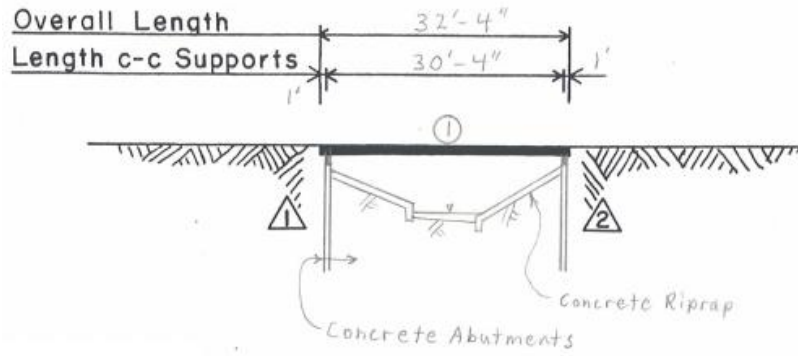
1. $f'_c = 3$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.9 Bridge CM-9

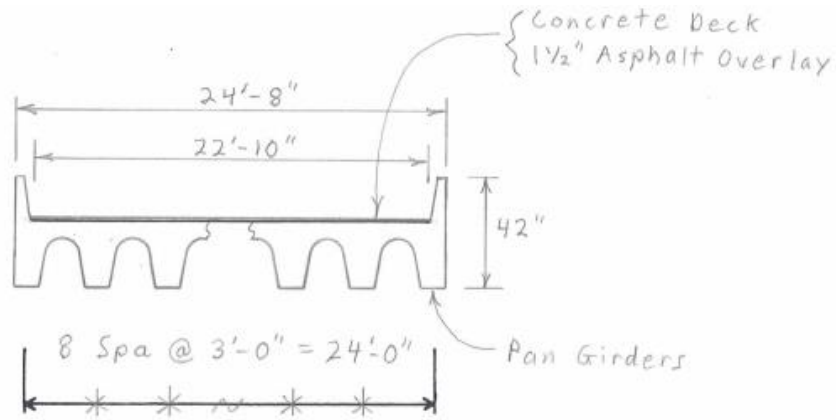
Bridge CM-9 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 5 (Fair). The bridge has a total length of 32 ft 4 in., which is one simply supported span. The controlling span for load rating is 30 ft 4 in. and is 24 ft 8 in. wide, with a roadway width of 22 ft 10 in. The bridge carries two lanes, one in each direction, and has an ADT of 950 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure C.63. Bridge CM-9 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section

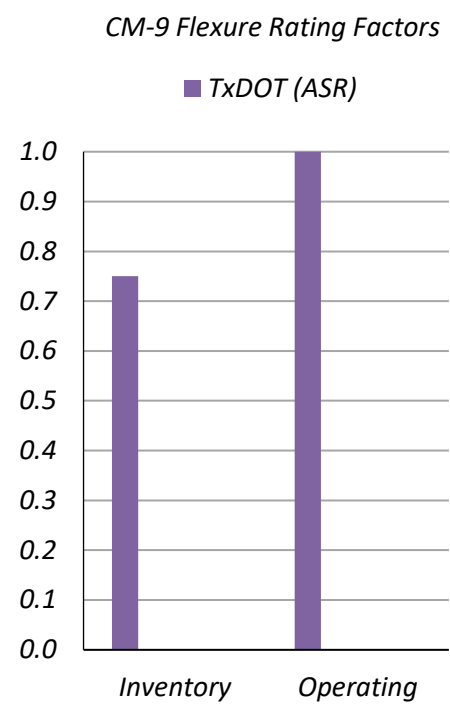


(c) Underside Superstructure View

Figure C.64. Bridge CM-9 Details (TxDOT 2018c)

Table C.32. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-9

Bridge Characteristics	
Year Built	1960
On/Off-System	Off
ADT	950
Max Span Length	30'-4"
Deck Width	24'-8"
Roadway Width	22'-10"
Slab + Beam Depth (in.)	33.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	Unknown
TxDOT Inventory RF	0.75
TxDOT Operating RF	1.00
TxDOT Inventory Rating (HS)	15.0
TxDOT Operating Rating (HS)	20.0
Posted Axle Limit (lbs)	21,000



Notes:

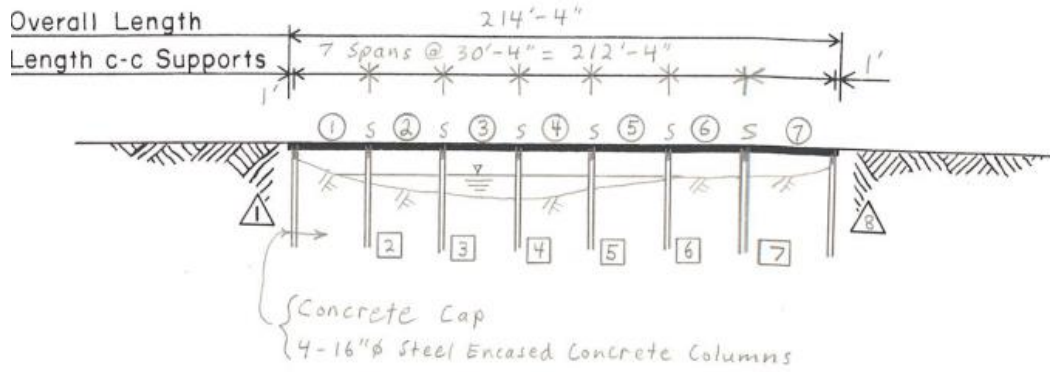
1. $f'_c = 3$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.10 Bridge CM-10

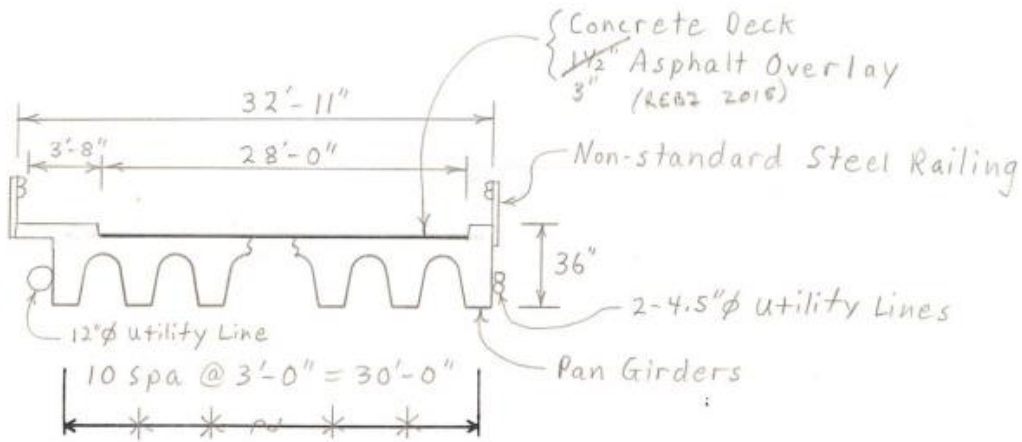
Bridge CM-10 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 5 (Fair). The bridge has a total length of 214 ft 4 in., consisting of 7 simply supported spans. The controlling span for load rating is 30 ft 4 in. and is 32 ft 11 in. wide, with a roadway width of 28 ft. The bridge carries two lanes, one in each direction, and has an ADT of 4920 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 21,000-lbs tandem axle.



Figure C.65. Bridge CM-10 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.66. Bridge CM-10 Details (TxDOT 2018c)

Table C.33. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-10

Bridge Characteristics	
Year Built	1960
On/Off-System	Off
ADT	4920
Max Span Length	30'-4"
Deck Width	32'-11"
Roadway Width	28'-0"
Slab + Beam Depth (in.)	33.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	0.75
TxDOT Operating RF	1.00
TxDOT Inventory Rating (HS)	15.0
TxDOT Operating Rating (HS)	20.0
Posted Axle Limit (lbs)	21,000

CM-10 Flexure Rating Factors

Condition	TxDOT (ASR)
Inventory	0.75
Operating	1.00

Notes:

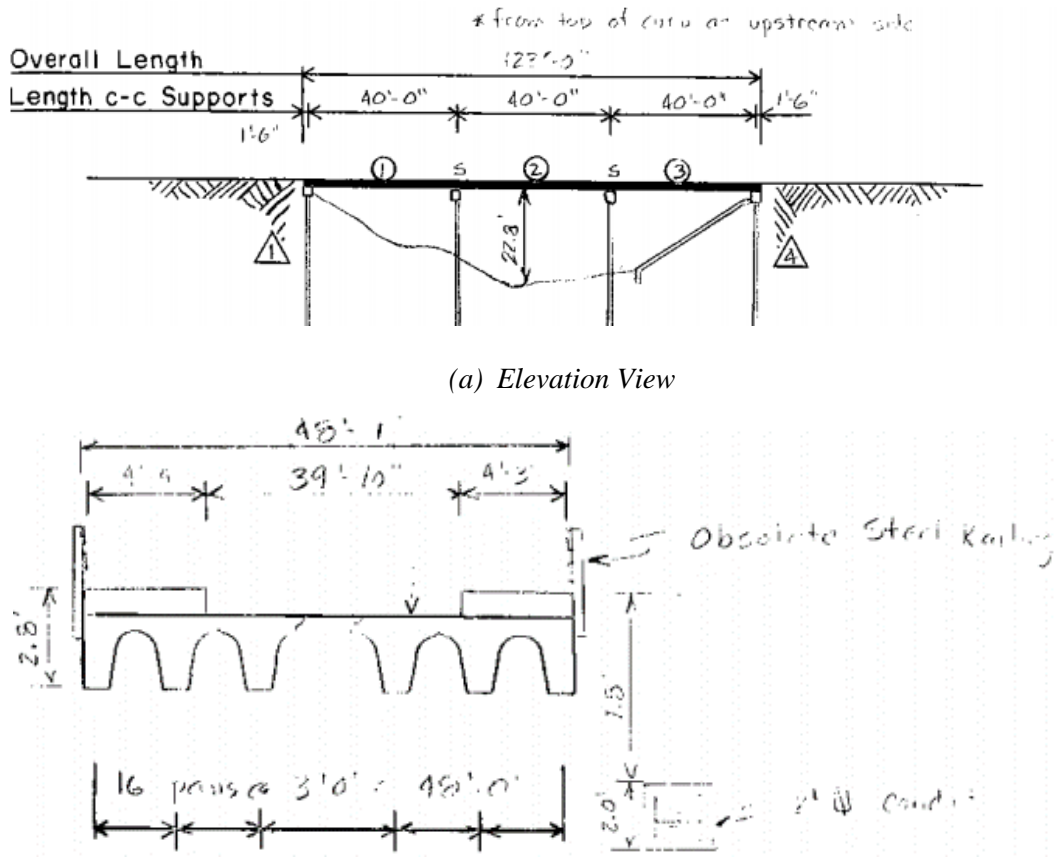
1. $f'_c = 3$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.11 Bridge CM-11

Bridge CM-11 has a deck condition rating of 6 (Satisfactory), a superstructure condition rating of 6 (Satisfactory), and a substructure condition rating of 7 (Good). The bridge has a total length of 123 ft, consisting of 3 simply supported spans. The controlling span for load rating is 40 ft and is 48 ft 1 in. wide, with a roadway width of 39 ft 10 in. The bridge carries two lanes, one in each direction, and has an ADT of 2650 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 28 US tons and an operating gross loading of 46 US tons. The bridge was posted but is now not posted.



Figure C.67. Bridge CM-11 (TxDOT 2018c)



(a) Elevation View

(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.68. Bridge CM-11 Details (TxDOT 2018c)

Table C.34. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-11

Bridge Characteristics	
Year Built	1960
On/Off-System	Off
ADT	2650
Max Span Length	40'-0"
Deck Width	48'-1"
Roadway Width	39'-10"
Slab + Beam Depth (in.)	33.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	Unknown
TxDOT Inventory RF	0.77
TxDOT Operating RF	1.29
TxDOT Inventory Rating (HS)	15.4
TxDOT Operating Rating (HS)	25.7
Posted Axle Limit (lbs)	Unknown

CM-11 Flexure Rating Factors

Condition	TxDOT (ASR)
Inventory	0.77
Operating	1.29

Notes:

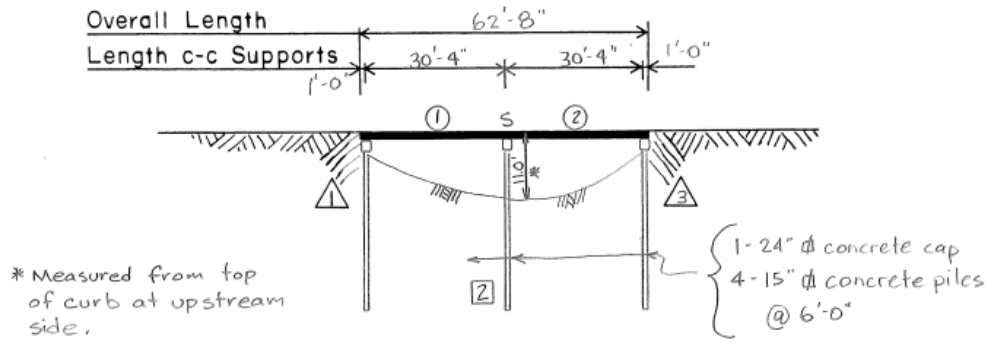
1. $f'_c = 3$ ksi per TxDOT AASHTO MBE.
2. $f_y = 33$ ksi per TxDOT AASHTO MBE.

C.2.12 Bridge CM-12

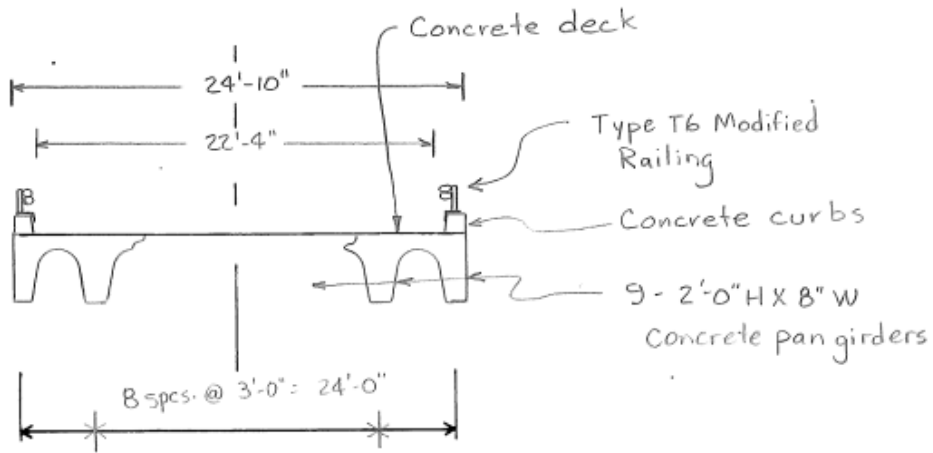
Bridge CM-12 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 6 (Satisfactory). The bridge has a total length of 62 ft 8 in., consisting of 2 simply supported spans. The controlling span for load rating is 30 ft 4 in. and is 24 ft 10 in. wide, with a roadway width of 22 ft 4 in. The bridge carries two lanes, one in each direction, and has an ADT of 70 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 27 US tons and an operating gross loading of 36 US tons. The bridge is posted for a 15,000-lbs tandem axle.



Figure C.69. Bridge CM-12 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section

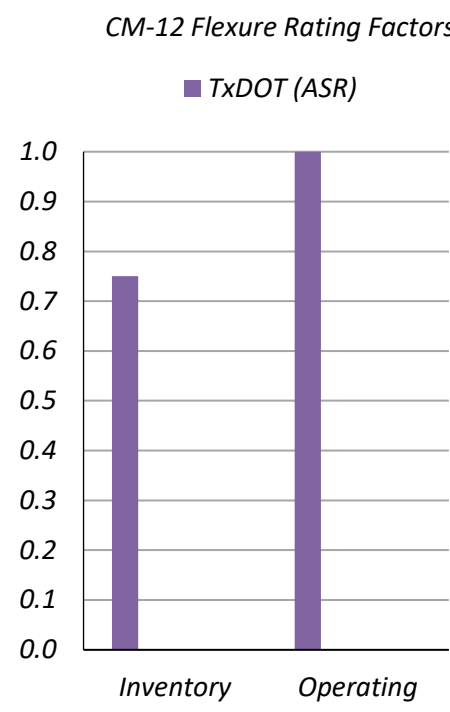


(c) Underside Superstructure View

Figure C.70. Bridge CM-12 Details (TxDOT 2018c)

Table C.35. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-12

Bridge Characteristics	
Year Built	1975
On/Off-System	Off
ADT	70
Max Span Length	30'-4"
Deck Width	24'-10"
Roadway Width	22'-4"
Slab + Beam Depth (in.)	24.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	Unknown
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	0.75
TxDOT Operating RF	1.00
TxDOT Inventory Rating (HS)	15.0
TxDOT Operating Rating (HS)	20.0
Posted Axle Limit (lbs)	15,000



Notes:

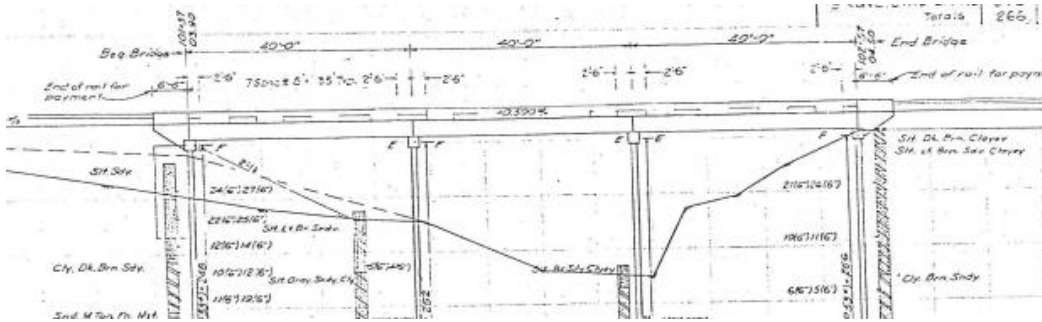
1. $f'_c = 3$ ksi per TxDOT AASHTO MBE.
2. $f_y = 36$ ksi per TxDOT AASHTO MBE.

C.2.13 Bridge CM-13

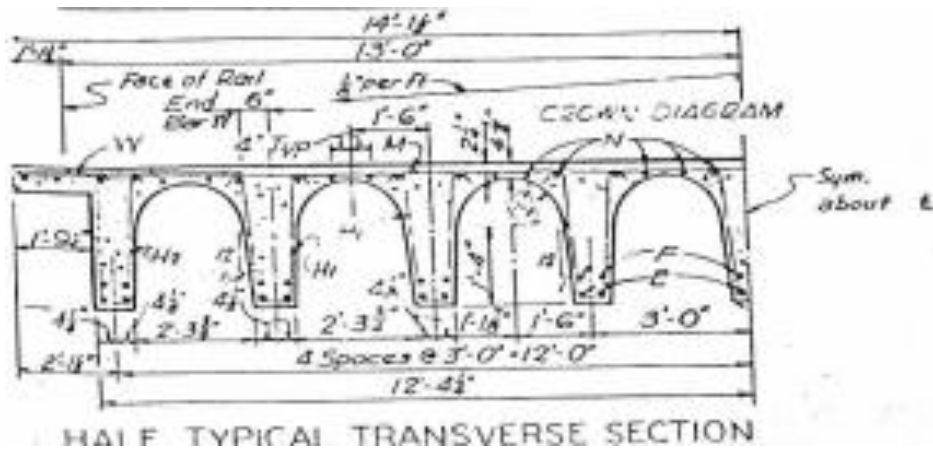
Bridge CM-13 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 5 (Fair). The bridge has a total length of 120 ft, consisting of 3 simply supported spans. The controlling span for load rating is 40 ft and is 28 ft 4 in. wide, with a roadway width of 26 ft. The bridge carries two lanes, one in each direction, and has an ADT of 80 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 24 US tons and an operating gross loading of 41 US tons. The bridge is posted for a 24,000-lbs tandem axle and a 52,000-lbs gross vehicle.



Figure C.71. Bridge CM-13 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.72. Bridge CM-13 Details (TxDOT 2018c)

Table C.36. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-13

Bridge Characteristics	
Year Built	1982
On/Off-System	Off
ADT	80
Max Span Length	40'-0"
Deck Width	28'-4"
Roadway Width	26'-0"
Slab + Beam Depth (in.)	33.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	2-#6 and 2-#10
Wearing Surface Thickness (in.)	0.0
TxDOT Inventory RF	0.68
TxDOT Operating RF	1.14
TxDOT Inventory Rating (HS)	13.6
TxDOT Operating Rating (HS)	22.7
Posted Axle Limit (lbs)	see note 1

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft)	119.3	C_n (kip-ft)	329	C_n (kip-ft)	329
C_{OP} (kip-ft)	167				
M_{DL} (kip-ft)	99.8	M_{DL} (kip-ft)	99.8	M_{DL} (kip-ft)	99.8
$M_{LL-HS20}$ (kip-ft)	140.4	$M_{LL-HS20}$ (kip-ft)	140.4	$M_{LL-HL93}$ (kip-ft)	272.9
Inventory HS20 RF	0.14	Inventory HS20 RF	0.65	Inventory HL93 RF	0.43
Operating HS20 RF	0.48	Operating HS20 RF	1.09	Operating HL93 RF	0.55
Inventory Rating (HS)	2.8	Inventory Rating (HS)	13.08		
Operating Rating (HS)	9.6	Operating Rating (HS)	21.8		

Notes:

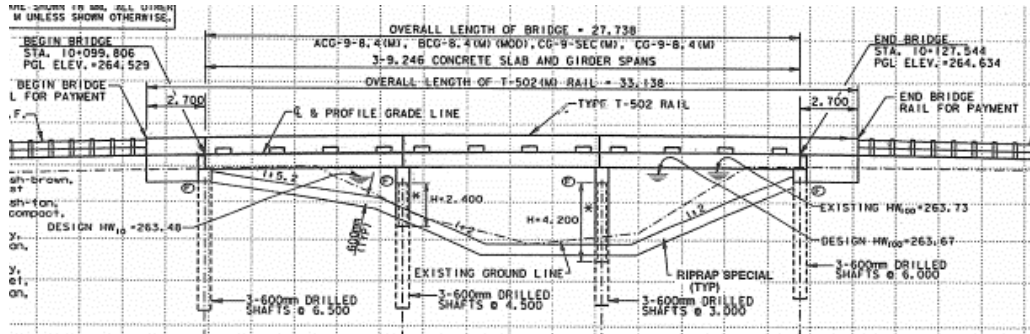
1. Posting restrictions include a 24,000-lbs tandem axle and a 52,000-lbs gross.
2. $f'_c = 3$ ksi per TxDOT Load Rating Calculations.
3. $f_y = 40$ ksi per TxDOT Load Rating Calculations.

C.2.14 Bridge CM-14

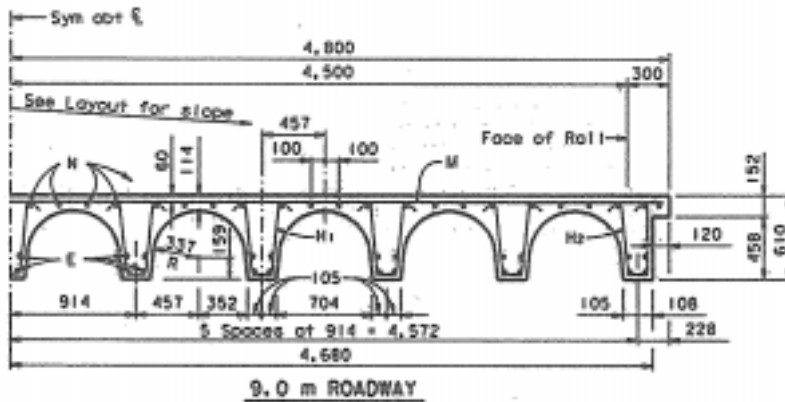
Bridge CM-14 has a deck condition rating of 7 (Good), a superstructure condition rating of 7 (Good), and a substructure condition rating of 8 (Very Good). The bridge has a total length of 90 ft, consisting of 3 simply supported spans. The controlling span for load rating is 30 ft and is 28 ft 4 in. wide, with a roadway width of 26 ft. The bridge carries two lanes, one in each direction, and has an ADT of 1890 vehicles. The concrete pan girders control the rating of the bridge, which has an inventory gross loading of 36 US tons and an operating gross loading of 49 US tons. The bridge is posted for a 19,500-lbs axle or tandem axle.



Figure C.73. Bridge CM-14 (TxDOT 2018c)



(a) Elevation View



(b) Longitudinal Section



(c) Underside Superstructure View

Figure C.74. Bridge CM-14 Details (TxDOT 2018c)

Table C.37. Characteristics and Interior Girder Flexure Load Ratings of Bridge CM-14

Bridge Characteristics	
Year Built	2000
On/Off-System	Off
ADT	1890
Max Span Length	30'-0"
Deck Width	28'-4"
Roadway Width	26'-0"
Slab + Beam Depth (in.)	24.0
Girder Spacing (in.)	36
Slab Reinforcement	Unknown
Beam Tension Reinforcement	2-#11 and 2-#11
Wearing Surface Thickness (in.)	3.0
TxDOT Inventory RF	1.00
TxDOT Operating RF	1.35
TxDOT Inventory Rating (HS)	20.0
TxDOT Operating Rating (HS)	27.0
Posted Axle Limit (lbs)	19,500

Condition	TxDOT (ASR)	ASR	LFR	LRFR
Inventory	1.00	0.46	1.46	0.80
Operating	1.35	0.90	2.44	1.04

Calculated Values for RFs					
ASR		LFR		LRFR	
C_{IN} (kip-ft)	94.3	C_n (kip-ft)	345.7	C_n (kip-ft)	196
C_{OP} (kip-ft)	132				
M_{DL} (kip-ft)	54.2	M_{DL} (kip-ft)	54.2	M_{DL} (kip-ft)	54.2
$M_{LL-HS20}$ (kip-ft)	86.8	$M_{LL-HS20}$ (kip-ft)	86.8	$M_{LL-HL93}$ (kip-ft)	345.7
Inventory HS20 RF	0.46	Inventory HS20 RF	1.46	Inventory HL93 RF	0.80
Operating HS20 RF	0.90	Operating HS20 RF	2.44	Operating HL93 RF	1.04
Inventory Rating (HS)	9.2	Inventory Rating (HS)	29.2		
Operating Rating (HS)	18.0	Operating Rating (HS)	48.8		

Notes:

1. $f'_c = 3$ ksi per TxDOT AASHTO MBE.
2. $f_y = 36$ ksi per TxDOT AASHTO MBE.

