Bridges Designed for Minimum Maintenance

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BRIDGES DESIGNED FOR MINIMUM MAINTENANCE

Final Report

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16. Abstract

Service-life based designs of highway bridges have gained significant interest from state DOTs in recent years. The primary purpose of these efforts is to provide durability-based designs for bridges and bridge elements that require minimal maintenance during the targeted service life of the structure. This is achieved by defining different target service lives for bridge design with the assumption that a longer target service life will result in a longer period with no or minimal maintenance. While the importance of durability-based design is highlighted in the report, which aims to achieve varying levels of target service lives that change based on the importance or the criticality of the bridge, the study acknowledges that most common bridges could be replaced due to functional improvements rather than reaching the end of their service life. Target service lives of 75 years (Normal), 100 years (Enhanced), and 125 years (Maximum) are considered in this report. The study focuses on identifying common types of bridge deterioration and maintenance activities, developing design recommendations suited to various traffic conditions and exposures, and validating the performance and cost-effectiveness of these designs through service life analysis. The report also discusses the challenges of predicting long-term degradation mechanisms and exposure conditions, and the importance of preventive maintenance strategies to prolong bridge life. Practical recommendations are presented for the considered bridge elements to achieve the different target service lives using economical design considerations. The immediate benefit of this research is implementing cost-effective bridge designs that require little to no maintenance, thereby reducing long-term costs and improving the overall performance of the transportation system in Iowa. The findings of this study could also be used by Iowa DOT to develop a guide for service life design of Iowa bridges.

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EXECUTIVE SUMMARY

Service-life based designs of highway bridges have gained significant interest from state Departments of Transportation (DOTs) in recent years. The primary purpose of these efforts is to provide durability-based designs for bridges and bridge components that require minimal maintenance during the targeted service life of the structure. Such designs usually lead to a better life cycle cost of the asset by avoiding costly maintenance during the targeted service life. The focus has been to design bridges to achieve at least 100-year service life, especially for signature bridges. However, an NCHRP report indicates that most bridges are replaced after 53 years, and for reasons other than having reached the end of their service life. Replacement is often primarily due to functional improvements needed for the transportation system rather than deterioration of bridge condition. For example, bridges designed in the 1950s may not be suitable to support traffic loads today due to the significant difference in vehicular weight, dimensions, and number of vehicles utilizing the bridge; even if these bridges are suitable to stay in service based on their condition. Therefore, it may not be economical to design common bridges to obtain a 100-year service life because they will need to be replaced before that time.

The primary purpose of this study is to provide durability-based designs for bridges and bridge elements in Iowa that result in minimal (or no) maintenance needs for the first 50 years of service, approximately the same time as the typical bridge age at replacement. The project focuses on developing design recommendations for various bridge elements to achieve target service lives of 75 years (Normal), 100 years (Enhanced), and 125 years (Maximum), with the understanding that an increased target service life is expected to correspond to a decreased risk of maintenance needs within the first 50 years of life. These strategies are expected to be more economical and sensible than consistently designing for a 100-year service life due to the availability of a shorter targeted service life and the flexibility given to the designer to select the target service life that best applies on a case-by-case basis.

This study identifies common types of bridge deterioration experienced by and maintenance activities conducted in Iowa, develops design recommendations suited to various traffic conditions and exposures across Iowa, and validates the performance and cost-effectiveness of these designs through service life analysis. Based on the current maintenance practices and needs of Iowa DOT, identified through review of Iowa DOT maintenance records and a survey of local Iowa DOT jurisdictions, the following bridge elements were identified as requiring the most maintenance:

1. Reinforced concrete members: concrete decks, concrete barriers, concrete girders, concrete pier caps, concrete pier columns
2. Steel superstructures
3. Joints
4. Bearings
5. Foundations
6. Approach systems
7. Berm erosion

Durable designs for target service lives—Normal, Enhanced, and Maximum—were developed and validated through service life modeling using WJE CASLE™ or quantitative performance prediction where

feasible, such as for reinforced concrete elements and steel superstructures. To support the service life modeling, chloride exposure assumptions suitable for Iowa sites were verified through chloride profile testing of cores collected from ten bridges across Iowa.

The service life design methodology could not be applied to all elements due to the lack of existing technology to reduce maintenance needs (e.g., joints and bearings) or insufficient knowledge to model the deterioration and expected service life of certain elements (e.g., foundations, approach systems, and berms). In these cases, the current standard designs and their performance were reviewed, and best design practices to avoid or minimize deterioration rates were identified for the Iowa DOT to consider.

This report serves as general guidelines for the Iowa DOT to select different design strategies to minimize bridge element maintenance based on a targeted service life approach. This will help reduce maintenance costs during the early stages of a bridge's life, with the risk of maintenance needs within this time period depending on the selected target service life during design. Portions of this report can be used by the Iowa DOT to create a guide for service life design for bridges, similar to the guidance provided by the Guide Specification for Service Life Design of Highway Bridges (AASHTO, 2020) and the MnDOT Service Life Design Guide for Bridges. Additional information in this report classifies exposure categories in Iowa based on salting practices along different road types. This information can be used alongside the design recommendations in this report to optimize the selection of design requirements for Iowa bridges.

A summary of the sections of this report that can be used to develop a guide for service life design for Iowa bridges is provided below.

- Classification of exposure zones can be found in Section 6.1, Section 6.2, and Appendix A.
- Guidance for service life design of reinforced concrete elements can be found in Section 6.1.
- Guidance for service life design of steel superstructures can be found in Section 6.2.
- Guidance for selection of joints and bearings can be found in Section 6.3 and Section 6.4, respectively.
- General consideration for foundations can be found in Section 6.5.
- General recommendations for approach slabs and berm erosion can be found in Section 6.6 and Section 6.7, respectively.

CHAPTER 1. INTRODUCTION AND BACKGROUND

1.1 Introduction

Service-life based designs of highway bridges have gained significant interest from state DOTs in recent years. The primary purpose of these efforts is to provide durability-based designs for bridges and bridge components that require minimal (or no) maintenance during the targeted service life of the structure. Such designs usually lead to better life cycle cost of the asset by avoiding costly maintenance during the targeted service life. The focus has been to design bridges to achieve at least 100-year service life, especially for signature bridges. However, an NCHRP report indicates that most bridges are replaced after 53 years and for reasons other than having reached the end of their service life (Bektas & Albughdadi, 2018). Replacement is often primarily due to functional improvements needed for the transportation system. For example, bridges designed in the 1950s may not be suitable to support traffic loads today due to the significant difference in vehicular weight, dimensions, and number of vehicles utilizing the bridge; even if these bridges are suitable to stay in service based on their condition. Therefore, it may not be economical to design common bridges to obtain a 100-year service life because they will need to be replaced before that time.

The focus of this study is to explore strategies that can be employed to design bridges to have minimum maintenance requirements during their targeted service lives. These strategies are expected to be more economical and sensible due to the shorter targeted service life. While a significant understanding of potential degradation mechanisms (corrosion of steel, corrosion of reinforced concrete elements, etc.) and hazards (traffic impact, scour, etc.) currently exists, there remains a lack of national industry standards that focus on durability-based designs for bridges and bridge components to achieve a targeted service life with minimal (or no) maintenance needs.

1.2 Background

An important mission of state departments of transportation (DOTs) is to maintain the bridge network in a state of good repair such that the system can reliably facilitate convenient travel for the public. The state of Iowa has over 23,000 bridges of which 4,168 are maintained by the Iowa DOT; the remainder are the responsibility of local or other agencies. To effectively manage and maintain the aging transportation infrastructure, Iowa DOT implemented a Transportation Asset Management Plan (TAMP) in lieu of the previously used worst-first management approach. Iowa DOT's TAMP provides a more balanced approach between reconstruction and preservation of transportation assets with the goal of minimizing long-term costs, extending the service life of transportation assets, and improving the overall transportation system performance. Iowa DOT's target for a state of good repair consists of maintaining a minimum of 46.8 percent of bridges (by deck area) in good condition and permitting no more than 6.5 percent of bridges (by deck area) to be in poor condition (IowaDOT, 2019). To consistently meet these metrics, the Iowa DOT must distribute limited funding and resources across its large number of bridge assets efficiently, resulting in a continual need for the development, identification, and implementation of more cost-effective bridge designs and maintenance strategies.

Traditionally, bridge management entailed permitting the bridge to deteriorate and conducting maintenance on an as-needed basis. Often bridge maintenance is only executed once distress of the structure or a component is evident or to maintain functionality of the bridge. Examples of these as-needed maintenance activities include replacement of deteriorated bearings or steel members, patching delaminations and spalls in reinforced concrete or prestressed concrete members, beam end repairs, and scour repairs. As bridges continue to age and deteriorate, these maintenance activities increase and can overwhelm maintenance departments that strive to keep the bridge and roadways open and safe. Maintenance, preservation, and rehabilitation activities are implemented by the DOTs to keep bridges in satisfactory condition. Examples of Iowa DOT's typical bridge maintenance activities are provided in Figure 1 (IowaDOT, 2019).

Work Type	Treatment Family	Project Treatment	Typical Unit Cost
Preservation	Paint steel	Routine painting of steel girders	$10/sq. ft.
Preservation	Wash weathering steel	Wash weathering steel girders on a regular basis	$4,000/ bridge
Maintenance	Strip seal joint repair	Replace glands	$100/ft.
Maintenance	Expansion joint replacement	Install new expansion joints	$2,000/ft.
Rehabilitation	Deck overlay	Dense concrete overlay	$50/sq. ft.
Rehabilitation	Deck overlay	Epoxy Polymer overlay	$30/sq. ft.
Preservation	Epoxy injection	Inject epoxy into delaminated areas under deck overlays.	$12/sq. ft.
Maintenance	Deck patching	Repair delaminated and spalled areas of a deck	$100/sq. ft.
Maintenance	Prestressed girder repair	Repair girder ends under joints	$1,500/ beam end
Rehabilitation	Deck replacement	Replace bridge deck	$75/sq. ft.
Reconstruction	Bridge replacement	Replace bridge	$325/sq. ft. of existing bridge deck area
Reconstruction	Culvert replacement	Replace culvert	$650/CY/ft.
Construction	New bridge	New bridge	$118/sq. ft.
Construction	New culvert	New culvert	$650/CY/ft.

Figure 1. Table 3-1 of Bridge Treatments and Unit Costs from Iowa DOT TAMP (2019).

Currently, there is general agreement that minimizing the frequency of bridge replacements is both cost-effective for the agency and desirable to the public, and as a result, transportation agencies have been implementing preventive maintenance strategies to prolong bridge life. As shown in Figure 1, preventive maintenance activities that the Iowa DOT commonly implements include routine painting, washing of steel members, joint repair, epoxy injection, and deck overlays. Other preventive strategies include application of crack sealers, concrete sealers, jackets, and wraps as well as joint replacement and cathodic protection of reinforced concrete, prestressed concrete, or steel members (FHWA, 2018). Preventive maintenance may be classified as cyclical, in which the frequency is predetermined (e.g., sealing concrete surfaces every 5 years regardless of condition), or condition-based, in which action is triggered due to the presence of distress (e.g., spot or zone painting a deteriorated steel coating). Condition-based preventive maintenance and maintenance applied on an as-needed basis are similar. The key difference between the two is that condition-based preventive maintenance is applied when the bridge is still in relatively good condition and with the primary purpose of prolonging the bridge's life by slowing deterioration, whereas as-needed maintenance is applied when bridge distress is noted and for the purpose of correcting conditions that might compromise bridge serviceability or functionality. While the types of activities may be the same, the scope and purpose of the work differs.

1.3 Overview of Service Life Design for Service Lives Exceeding 50 Years

Recently, state DOTs have been further reducing life cycle costs by requiring that bridges be designed for a prolonged service life with typical target service lives between 75 and 100 years or more, particularly when the bridge is a high-profile asset or signature structure for which replacement is expensive and disruptive. This has led to the implementation of robust, durable bridge designs and construction practices that either fully avoid, withstand, or at least slow deterioration such that minimal maintenance is required in the bridge's early age and rehabilitation is not to be expected within the targeted service life.

New construction projects with long service life requirements commonly require a separate durability consultant to develop corrosion control or durability plans. These plans review the design of each bridge component and assess if the design and pre-planned maintenance program have adequately addressed the deterioration mechanisms associated with many of the categories identified by Azizinamini et al., 2013 (discussed in a following section). The types of bridge deterioration given the most attention in these projects typically fall under the categories of concrete durability and structural steel corrosion protection. Examples of deterioration mechanisms considered and typical corresponding mitigation strategies are provided in Table 1. The strategies may be categorized as "Avoidance of Deterioration," in which the structure has been designed such that the degradation mechanism is physically infeasible; "Deemed to Satisfy," in which the structure is deemed to have sufficient protection to withstand the exposure conditions causing degradation; or "Probabilistic Approaches," in which the deterioration of the bridge is modelled probabilistically to determine when the extent of distress exceeds acceptable limits with reasonable confidence.

The durability plans are developed by the following steps:

- Identify bridge components and elements that may require maintenance during the target service life.
- Identify applicable types of deterioration and the exposure conditions of each element or component.
- Assess the ability of the as-designed elements or components to withstand the assumed exposure conditions, either by their deemed-to-satisfy strategies or modelling.
- Coordinate with the design and construction contractors to develop improved designs for elements that do not meet the service life requirements according to the analysis conducted in Step 3.
- Reiterate Steps 3 and 4 until all bridge elements or components meet the service life requirements of the project.

Design changes that are commonly recommended if reinforced concrete does not meet the service life requirements include increasing the concrete cover; replacing carbon steel reinforcement with corrosion-resistant reinforcement such as epoxy-coated rebar or stainless steel; selecting higher quality concrete mixtures that are less permeable; and requiring more stringent QA/QC requirements ensuring a higher quality as-built condition. For steel elements, increasing section size and consequently the sacrificial steel thickness, applying a protective coating, or specifying a more robust protective coating such as a duplex system may be recommended. Other design choices that can promote durability include mitigation of joint leakage by elimination of bridge deck joints and use of integral abutments, and mitigation of scour by the use of armoured slopes or incorporation of river stabilization techniques (Hoppe, Weakley, & Thompson, 2016).

Successful realization of cost savings relies heavily on the commitment of the agency to implement service life design for the bridge and the experience and expertise of the durability consultant and contractor. Making educated predictions of future conditions is essential, including accurate characterization of exposure conditions, bridge use, and deterioration mechanisms, which is difficult to extrapolate for periods of 50 to 100 years. For example, in 1920, gross vehicle weight limits for trucks were typically under 28,000 pounds while today, the gross vehicle weight limit is 80,000 pounds and overweight trucks may be permitted to weigh as much as 155,000 pounds (U.S. DOT, 2000). Sodium chloride de-icing chemicals were first used in the 1930s (but not widely used until the 1970s) and their effects on corrosion of reinforced concrete were not known or widely studied until the 1960s (Kelly et al., 2010; Fischel, 2001). Similarly, alkali-silica reaction (ASR) was not a known concrete degradation mechanism until the 1940s (Thomas et al., 2013). These examples illustrate the difficulty in accurately anticipating causes of degradation and predicting exposure loads long-term. Because of these challenges and their exacerbation due to the large timescale, prediction of a 100-year service life is challenging and may not be practical or desirable, despite advancements in our understanding and modelling of bridge deterioration.

Table 1. Bridge deterioration mechanisms considered when designing for 75 to 100-year service life and examples of corresponding mitigation or preventive strategies.

Degradation Mechanism	Types of Mitigation or Preventive Strategies		
	Avoidance of Deterioration	**Deemed to Satisfy**	**Probabilistic Approaches**
Concrete Elements			
Alkali-silica reaction	Use of non-reactive aggregates	Use of SCMs and demonstration of ASR resistance by prequalification testing	--
Sulfate attack	Confirmation that sulfate concentrations in soil and water are not aggressive	Use of sulfate-resistant cements	--
Freeze-thaw and salt scaling	Confirmation that the bridge location does not experience freeze-thaw cycles Use of non-corrosive deicing agents	Provision of sufficient entrained air systems Demonstration of salt scaling resistance by prequalification testing	--
Delayed ettringite formation	Implementation of temperature control plan preventing elevated concrete temperatures	--	--
Cracking	--	Sealing of visible cracks	--

Degradation Mechanism	Types of Mitigation or Preventive Strategies		
	Avoidance of Deterioration	**Deemed to Satisfy**	**Probabilistic Approaches**
Carbonation-induced corrosion	--	Use of stainless-steel reinforcement[1]	Identification of minimum concrete cover and quality (permeability) required
Chloride-induced corrosion	Use of non-corrosive deicing agents	Use of stainless-steel reinforcement[1]	Identification of minimum concrete cover and quality (permeability) required
Steel Elements			
Atmospheric corrosion	--	Application of a paint or galvanization layer	--
Galvanic corrosion	Isolation between dissimilar metals	--	--
Soil corrosion	--	Inclusion of sacrificial thickness	--
Fatigue	--	Use of fatigue-resistant connections	--

[1]AASHTO Guide HBSLD-1 considers the use of stainless-steel as an avoidance of deterioration approach; however, the chloride corrosion threshold of stainless-steel bars is a function of the specific type and composition of the alloy. Therefore, in certain severe/aggressive exposure conditions, the use of stainless-steel bars may not guarantee avoidance of corrosion.

Overly robust designs and preservation strategies needed for 100-year service life may not be the optimal solution for most bridges. Bektas and Albughdadi (2018) showed that a 100-year service life may not always be necessary as many bridges are replaced in approximately 50 years. Bridges are more commonly decommissioned due to the need for functional improvements, such as increased width, adjusted clearance dimensions, or other parameters, rather than reasons related to poor bridge condition. This indicates that the 100-year service life requirement results in structures for which protection against deterioration is overdesigned, and less costly designs may be adequate for the bridge systems' needs. As a result, the goal of this study is to identify opportunities for the Iowa DOT to further optimize its bridge investments by identifying optimal bridge designs that are suitable for various targeted service lives and that require minimal maintenance during their service to provide relief to maintenance forces and funding.

1.4 Objectives

The primary objective of this study is to implement cost-effective bridge designs and techniques applied that require minimal (or no) maintenance during the first 50 years; through a targeted service life design approach. The study is intended to address principal maintenance activities for various bridge components and subsystems that are typically completed for different bridge types owned and maintained by Iowa DOT and local jurisdictions. A list of supportive sub-objectives of the study are:

- To identify the types of bridge deterioration and maintenance activities most common within and costly to Iowa transportation agencies by analyzing the available bridge maintenance records;
- To reduce maintenance costs by developing designs and pre-planned maintenance activities suited to each of the different traffic conditions and exposures found within Iowa;

- To identify designs suitable for further research due to insufficient performance data and potential for large cost savings.

1.5 Report Organization

This report contains the following chapters and appendices:

- **Chapter 1. Introduction and Background**
  This chapter introduces the project and its objectives and scope.
- **Chapter 2. Literature Review**
  This chapter summarizes the review of literature including published studies and guides on service life design.
- **Chapter 3. Bridge Maintenance Practices of the Iowa DOT**

  This chapter summarizes the analysis of Iowa DOT bridge maintenance data conducted. Additionally, average daily truck traffic (ADTT) data from Iowa DOT's Structure Inventory and Inspection Management System (SIIMS) is analyzed to understand the traffic demands of state-owned bridges and investigate the relationship between bridge ADTT and recommended maintenance activities.
- **Chapter 4. Survey of Bridge Maintenance Needs Across Iowa**

  This chapter presents a summary of the results of a survey sent to local Iowa DOT jurisdictions to prioritize a list of maintenance needs for their bridges. This list is used to guide the selection of bridge elements in Iowa that require increased focus for durability design to reduce maintenance needs.
- **Chapter 5. Approach for Development of Durable Bridge Designs**

  This chapter summarizes the list of bridge elements to be addressed for service life design and presents the methodology and approach to develop element specific service life design recommendations.
- **Chapter 6. Element-Specific Service Life Design Considerations**

  This chapter provides a summary of the recommendations and considerations to achieve various target service lives for the bridge elements that are listed in Chapter 5.
- **Chapter 7. Summary and Recommendations**

  The chapter provides a summary of the report and proposed implementation strategy.
- **Chapter 8. Bibliography**
  The chapter lists the citation for all the used references in the report.
- **Appendix A. Concrete Core Sampling and Chloride Profile Fitting**
  This appendix provides data from cores extracted from ten Iowa bridges of different types.

CHAPTER 2. LITERATURE REVIEW

This chapter presents the results of a literature review of published studies and guides on service life design. The following sections provide summaries of the most relevant studies to this investigation.

2.1 NCHRP Project 12-108 and AASHTO Guide Specification for Service life Design of Highway Bridges (2020)

A recent NCHRP study (NCHRP Project 12-108, 2020), led to the publication of the Guide Specification for Service Life Design of Highway Bridges (AASHTO, 2020) which provides practical guidance to designers of highway bridges on how to implement service life design through a three-tiered approach of "good-better-best" practices tied to four service life categories: Renewable, Normal, Enhanced, and Maximum. The study included a comprehensive review of published literature, technical reports from state DOTs, and an extensive survey of industry professionals. Two of the main studies reviewed were part of the Second Strategic Highway Research Program (SHRP 2), i.e., Project R19A: Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems, and Components (Azizinamini, Power, Myers, & Ozyildirim, 2014) and Project R19B: Bridges for Service Life Beyond 100 Years: Service Limit State Design (Kulicki, et al., 2015).

Several knowledge gaps were identified over the course of NCHRP Project 12-108. Most notably, limitations to the current service life prediction models and a lack of supporting data needed to develop deterioration models for additional service life limit states. The primary challenge behind this obstacle is the slow deterioration rate of bridges that limits the rate of data collection and impedes the near-term impact of research efforts on this subject. Consequently, the full understanding of the behavior of a bridge throughout its service life will always suffer from the long-term scale of the bridge degradation process.

A concise summary of the SHRP 2 studies is provided next.

2.2 SHRP 2 Project R19A

The objective of Project R19A was to identify the problems that typically limit the service life of bridges and to develop design provisions to address them. To achieve these objectives, an extensive literature review and survey were conducted as part of this study. Nine categories of service life challenges and needs resulted from this effort, namely: concrete durability; bridge decks; substructure; bearings; expansion joints, joints, and jointless bridges; fatigue and fracture; structural steel corrosion protection; steel bridge systems; and concrete bridge systems. The study identified causes of deterioration for each category as well as approaches to mitigate deterioration or plan maintenance to achieve a 100-year service life design goal. For example, for steel bridge systems, the study identified fatigue and fracture, and corrosion as the main deterioration issues for these types of bridge components. In addition, other issues related to truck impact and fire were identified as hazard-related issues.

A second phase of the project focused on developing a methodology to design bridges for service life while focusing on four areas of highest priority, joints, bearings, enhancing corrosion resistance of concrete bridges, and bridge decks. This second phase consisted of fourteen research studies leading to the development of new details, concepts, and in some cases, associated design provisions. An example result of these research efforts is the development of design provisions for jointless bridges. Ultimately, SHRP 2 Project R19A led to the development and proposal of the Design Guide for Bridges for Service Life

(Azizinamini, et al., 2014), which aimed to define procedures for the systematic design of new and existing bridges for service life and durability.

2.3 SHRP 2 Project R19B

The goal of SHRP 2 Project R19B was to develop design provisions, provide detailing guidance, and develop calibrated service limit states (SLSs) to achieve a bridge service life of 100-years. Additionally, the project aimed to develop a framework for the further development of calibrated SLSs. Project R19B relied on the findings of the previous project R19A, a survey of bridge owners, and other published studies to identify bridge performance challenges and evaluate the state of the art regarding SLSs. The results of these efforts indicated that most of the serviceability issues are related to expansion joints and deck cracking. Some of the most significant issues included deterioration and section loss of beam ends, painting of steel members, problems with bearings, corrosion of reinforcement, and deck overlays. Despite the limited survey responses (16 responses), the findings were consistent with other studies. Regarding SLSs, the survey respondents indicated that despite the many serviceability issues the existing SLSs are adequate, nonetheless suggestions of additional limit states were provided. Based on the survey results and literature review, a set of potential SLSs was developed and reviewed to assess which of the potential SLSs could be calibrated based on reliability theory. A discussion on structural reliability and limit state calibration is beyond the scope of this summary; refer to Kulicki, et al., 2015 for further details. Although it was not possible to calibrate several of the identified SLSs (due to their deterministic or judgement/experience basis), calibrated, reliability-based load and/or resistance factors for the following SLSs were developed: foundation deformations; cracking of reinforced concrete components; live load defections; permanent deformations; cracking of prestressed concrete components; fatigue of steel; and reinforced concrete components. The study ultimately led to a draft of proposed design provisions and modifications to AASHTO LRFD design provisions.

2.4 Summary of NCHRP Project 12-108 Industry Practice Survey

To assess the state-of-practice and knowledge on service life design, a questionnaire was sent to State DOTs bridge engineers and other agencies. Included in the questionnaire were questions from the 2017 AASHTO Subcommittee on Bridges and Structures' annual state bridge engineers survey and an online questionnaire. A total of 45 and 36 agencies responded the AASHTO survey and online survey, respectively. The questions included in the survey were grouped into three subjects: bridge systems; bridge elements; and design for durability.

The survey provided insight on the general aspects of durability and the common challenges faced by State DOTs. For example, the application and frequency of application of de-icing salts as well as maintenance frequency were selected as the owner actions with the most significant impact on bridge service life. From a design perspective, the survey clearly indicated that joints/structural continuity is the most significant factor impacting the service life of a bridge (nearly 70% of respondents ranked joints as the impactful factor). Furthermore, respondents indicated that joints and bridge decks are the most frequently repaired bridge components; over 60% and approximately 33% of respondents ranked joints and bridge decks, respectively, as the most repaired component. The survey also inquired about the most common durability issues of bridge specific components and types of bridges (i.e., concrete or steel bridges). A summary of the responses is provided in Table 2 along with the percent of respondents that selected them as a durability issue. Note that percentages provided in Table 2 can add up to over 100% given that the respondents were able to select multiple answers to the same question.

A significant amount of the issues impacting the durability of bridges are related to expansion joints. The survey results indicate that the most common issues associated with joints are leakage (100% of respondents), debris accumulated in the joints (84% of respondents), and material failure and damage (76% of respondents). Leaking joints in turn lead to deterioration of the components directly beneath the joint, most frequently, the bearings. The survey results indicate that the most common issues with bridge bearings are leakage induced deterioration (86.5% of respondents), steel corrosion (78% of respondents), and freezing or locking (60% of respondents).

Clearly, corrosion and joints are the primary factors impacting the durability of bridges. According to the survey, the two most common strategy to address reinforcement corrosion in bridge decks are the use of corrosion resistant reinforcement (64% of respondents) such as, stainless steel, epoxy coated bars, and galvanized reinforcement, and the specification of special concrete mixtures (61% of respondents), e.g., high performance concretes and low permeability concrete. The use of protective systems such as, sealants, membranes, overlays, and latex-modified concretes, was also reported as one of the common strategies to address corrosion; 40% of respondents indicated the use of one or more of these protection system against corrosion in bridge decks. Joint issues on the other hand, are by far addressed by eliminating the joints/jointless designs (66% of respondents). For further details regarding the survey questions and results, refer to the NCHRP Project 12-108 report (2020).

Table 2. NCHRP Project 12-108 survey summary - Most common durability issues affecting bridge components

Deterioration Mechanism	Concrete Bridges	Steel Bridges
Corrosion*	Deck (77%)	-
	Superstructure (71%)	Superstructure (87%)
	Pier, Walls, and Abutments (82%)	-
	Foundations (45%)	Foundations (89%)
Freeze-Thaw	Deck (20%)	-
	Superstructure (11%)	
	Pier, Walls, and Abutments (11%)	
	Foundations (7.5%)	
Alkali-Aggregate	Deck (2.2%)	-
	Superstructure (8.9%)	
	Pier, Walls, and Abutments (8.9%)	
	Foundations (2.5%)	
Fatigue	-	Superstructure (13%)

2.5 Overview of AASHTO Guide Specification for Service Life Design of Highway Bridges

As a direct result of the research studies within the Second Strategic Highway Research Program and NCHRP Project 12-108, the American Association of State Highway and Transportation Officials (AASHTO) published the *Guide Specification for Service Life Design of Highway Bridges*, henceforth referred to by its publication code HBSLD-1 (AASHTO, 2020). The objective of this HBSLD-1 guide is to "provide practical guidance to designers and owners on design decisions that affect the durability of highway bridges" within a single specification. Some of the key features of the HBSLD-1, are the definition of target service life categories, exposure zones and classes, provision of calibrated design requirements for reinforced concrete for the chloride-induced corrosion limit state, the inclusion of a framework for the implementation of a full probabilistic service life design method, and the inclusion of case studies exemplifying the application of the guide specification.

According to the HBSLD-1 guide, the service life categories are defined and associated to good-better-best practices as shown in Table 3.

Table 3. Service Life Category Definitions per AASHTO Guide Specification for Service Life Design of Highway Bridges

Category	Bridge Component Type	Bridge Description	Qualitative Practice Level
Renewable	Bearings, joints, strip seals, guardrails, barriers, sign structures, coating systems, approach slabs, sleeper slabs, deck overlays	All	Replaceable
Normal	All other components	Typical bridges	Good
Enhanced		Bridges with high cost, high ADT, social context, etc.	Better
Maximum		Bridges with higher cost, higher ADT, social context, etc.	Best

Source: Adapted from AASHTO Guide Specification for Service Life Design of Highway Bridges (AASHTO, 2020).

In general, there are two strategies for service life design, i.e., 1) providing means for the structure to withstand both environmental and load demands without reaching defined limit states during its service life, and 2) removing the structure's vulnerability to deterioration by eliminating vulnerable details and/or the use of non-reactive materials, e.g. stainless steel reinforcement. The HBSLD-1 guide draws upon *fib* Bulletin 34: Model Code for Service Life Design (fib, 2006) and the ISO standard ISO 16204 Durability - Service life design of concrete structures (ISO, 2012) to define four different approaches for service life design; these are:

- Full Probabilistic Design – Based on validated, probabilistic deterioration models to compute reliability indices for specific limit states.
- Partial Factor Design (semi-probabilistic) – Deterministic approach that relies on partial safety factors (calculated using the full probabilistic method) for the applied actions and material resistance to allow designers to evaluate specific limit states during the design.
- Deemed-to-Satisfy – Provides prescriptive requirements that should lead to a bridge service life above minimum specified service life
- Avoidance-of-Deterioration – Assumes a given deterioration mechanism is not occurring, e.g., due to the use of non-reactive materials and/or separating the environmental action from the structure using protective systems.

Both ISO 16204 and *fib* Bulletin 34 have identified various limit states to consider in the full probabilistic and semi-probabilistic approaches. Limit states for the probabilistic and semi-probabilistic approaches are summarized in Table 4. However, except for carbonation- and chloride-induced depassivation, there are no generally accepted deterioration models for most of the identified service life limit states. Therefore, the most used approaches to service life design are the Deemed-to-Satisfy and Avoidance-of-Deterioration methods. This is a consequence of the limitations of current service life prediction models and a lack of supporting data needed to develop deterioration models for additional service life limit states, as well as difficulties associated with the definition of service life limit states.

Table 4. Service Life Limit States for the Full Probabilistic and Partial Factor design methods.

Full Probabilistic Design		Partial Factor Design (Semi-Probabilistic)
1.	Chloride-induced depassivation (ISO 16204 / *fib* Bulletin 34)	Chloride-induced depassivation (uncracked concrete)
2.	Carbonation-induced depassivation (ISO 16204 / *fib* Bulletin 34)	Carbonation-induced depassivation (uncracked concrete)
3.	Corrosion-induced cracking, spalling, and collapse (ISO 16204)	-
4.	Freeze-thaw damage no de-icing agents or salt water (ISO 16204 / *fib* Bulletin 34)	Freeze-thaw damage (no deicing agents or salt water)
5.	Freeze-thaw damage with de-icing agents or salt water (ISO 16204 / *fib* Bulletin 34)	-
6.	Freeze-thaw induced deflection and collapse (ISO 16204)	-

Sources: ISO 16204, fib Bulletin 34, and AASHTO (2020).

The HBSLD-1 guide provides guidance primarily focused on the Deemed-To-Satisfy and Avoidance-of-Deterioration design methods. However, appendix A of the HBSLD-1 guide provides a framework and guidance for the implementation of the Full Probabilistic method for the design of concrete structures subjected to chloride-induced corrosion. For the Deemed-to-Satisfy approach, design requirements and mitigation strategies are provided as a function of predefined exposure classes and service life categories.

Exposure classes are defined based on the type of deterioration mechanism (e.g., corrosion and freeze-thaw) and environment exposure zones. Environment exposure zones are further subdivided into macro and micro exposure zones to be considered for the overall bridge as well as individual bridge elements or regions. Four macro environment exposure zone are defined within the HBSLD-1 guide as follows:

- Rural/Mild/Nonaggressive – "Little to no exposure to airborne or applied (i.e., deicing) salts. Low pollution from sulfur dioxide, low humidity and precipitation, and no exposure to chemical fumes. Typically, inland locations".
- Industrial/Moderate – "Occasional exposure to airborne salts or deicing salt runoff. Noncoastal bridges with irregular deicing salt application. Industrial areas with airborne contaminants, polluted urban areas, areas with moderate to high humidity".
- Marine – "Coastal environments with exposure to airborne salts or direct contact with sea water or brackish water. Typically defined by a limiting distance from a coast. State and local transportation specifications should be consulted in determining the extent of a marine

environment. In lieu of other guidance, bridges located within 0.5 miles of a body of salt water maybe considered to be in a marine environment".

- Deicing – "Region where deicing salts are used on a regular basis during the winter months".

In addition to the macro exposure zones, eight micro exposure zones are defined in section 2.2 of the HBSLD-1, namely Buried Zone, Submerged Zone, Tidal Zone, Low Water Zone, Direct Deicing Zone, Indirect Deicing Zone, Splash/Spray Zone, and Atmospheric Zone. The micro exposure zones are best illustrated by Figure 2 (AASHTO, 2020), which has been adapted by the AASHTO HBSLD-1 guide from (Morley & Bruce, 1983), (West, Laroshce, Koester, Breen, & Kreger, 1999), (Caltrans, 2010), (UFGS, 2012), and (Hannigan, Rausche, Likins, Robinson, & Becker, 2016).

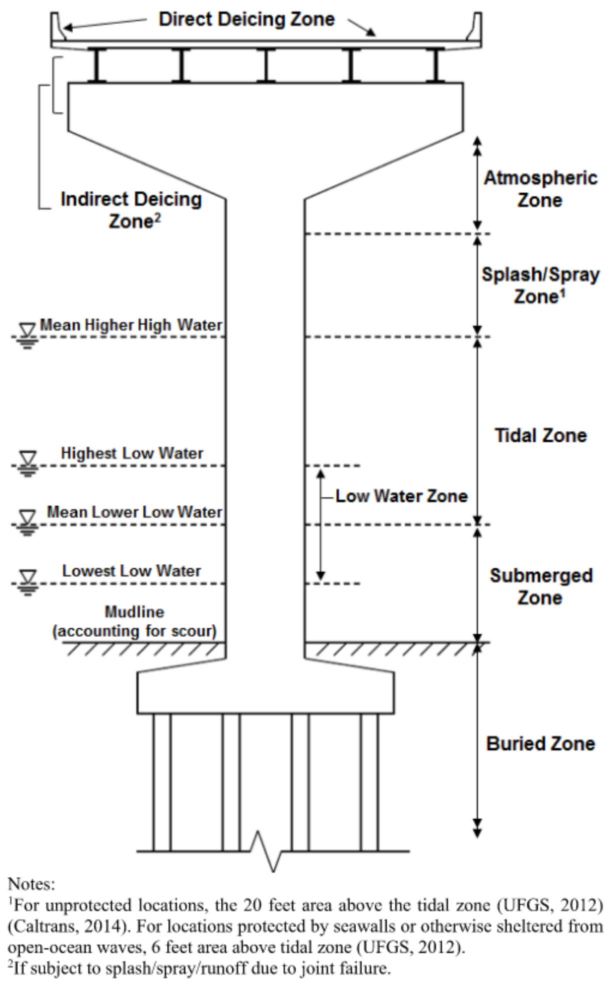

Figure 2. Micro Environment Exposure Zones as defined in the HBSLD-1 guide (AASHTO, 2020)

Ultimately, the exposure classes are grouped by the type of deterioration mechanism and are defined based on the exposure zones. Exposure classes for concrete structures are defined for corrosion, freeze-thaw, sulfate attack, and concrete in contact with water, whereas exposure classes for steel structures are defined for corrosion and fatigue. For further details refer to Sections 4 and 5 of the HBSLD-1 guide for the provisions for concrete and steel structures, respectively.

Deemed-to-Satisfy guidance and prescriptive requirements are provided in the form of concrete material specifications, concrete cover dimensions, crack control approaches, coatings and other protective systems (e.g., membranes and overlays for concrete), guidance for replaceable elements, as well as element-specific guidance. Moreover, Deemed-To-Satisfy requirements are a function of the desired/target service life, exposure of the element in consideration, and the consequences of failing to achieve the targeted service life.

In general, the framework for service life design presented in the AASHTO HBSLD-1 guide follows the steps outlined in the previous section. However, detailed case studies illustrating the implementation of the service life design methodology are provided in Appendix B of the HBSLD-1 guide.

CHAPTER 3. BRIDGE MAINTENANCE PRACTICES OF THE IOWA DOT

This chapter summarizes the analysis of Iowa DOT bridge maintenance data conducted. Additionally, average daily truck traffic (ADTT) data from Iowa DOT's Structure Inventory and Inspection Management System (SIIMS) is analyzed to understand the traffic demands of state-owned bridges and investigate the relationship between bridge ADTT and recommended maintenance activities. The purpose of this chapter is to identify the most common maintenance actions and bridge durability issues that the Iowa DOT most commonly encounter in its efforts to maintain the bridges in a state of good repair. The results of this chapter and Chapter 4 will be a prioritized list of the bridge components that require the most maintenance by Iowa DOT.

3.1 Analysis Scope

The study analyzed the "recommended maintenance data set" and the "program work data set" obtained from Iowa DOT SIIMS to identify the most common types of distress/deterioration and bridge maintenance activities within the Iowa DOT bridge inventory. The analysis scope included:

1. Quantifying bridge maintenance activities based on Iowa DOT maintenance codes.
2. Identifying most common maintenance activities for bridge decks, superstructure, substructure, and pavements.
3. Analyzing truck traffic demands for Iowa bridges for a sample of the state bridge inventory.
4. Investigating the relationship between truck traffic demand and most common bridge maintenance activities.
5. Evaluating the sensitivity of the top maintenance activities relative to the ADTT.

3.2 Maintenance Recommendations Data Set

The SIIMS maintenance recommendations data set included 9,046 maintenance items covering 3,108 interstate and state-owned bridges across Iowa. The data set included repair dates from 2011 to 2021. Iowa DOT maintenance records are organized by numerical codes that group maintenance recommendations into several categories. Of the categories defined within SIIMS, the most relevant to this study are:

- Deck Maintenance – codes 100 to 199
- Superstructure Maintenance – codes 200 to 299
- Bearings and Substructure Maintenance – codes 300 to 399
- Miscellaneous Maintenance – codes 400 to 499
- Pavement Maintenance – codes 500 to 599

3.2.1 Truck Traffic Statistics

The maintenance records included average daily traffic (ADT) estimates and average daily truck traffic (ADTT) estimate as a percentage of the ADT estimates. ADTT percentages were used to calculate average daily truck traffic counts for each bridge within the data set (total of 3108 bridges). A histogram of the number of bridges with ADTT at or below units of a given value and the cumulative frequency is shown in

Figure 3. Average daily truck traffic values ranged from zero to 15,498 trucks per day, with an average ADTT of 966 trucks per day for all bridges analyzed. As shown in Figure 3, about 76% of the bridges in the data set have ADTT at or below 1,000 trucks per day. The 50th, 75th, and 90th percentile ADTT correspond to bridges with 441, 943, and 2,520 trucks per day, respectively, as shown in Table 5.

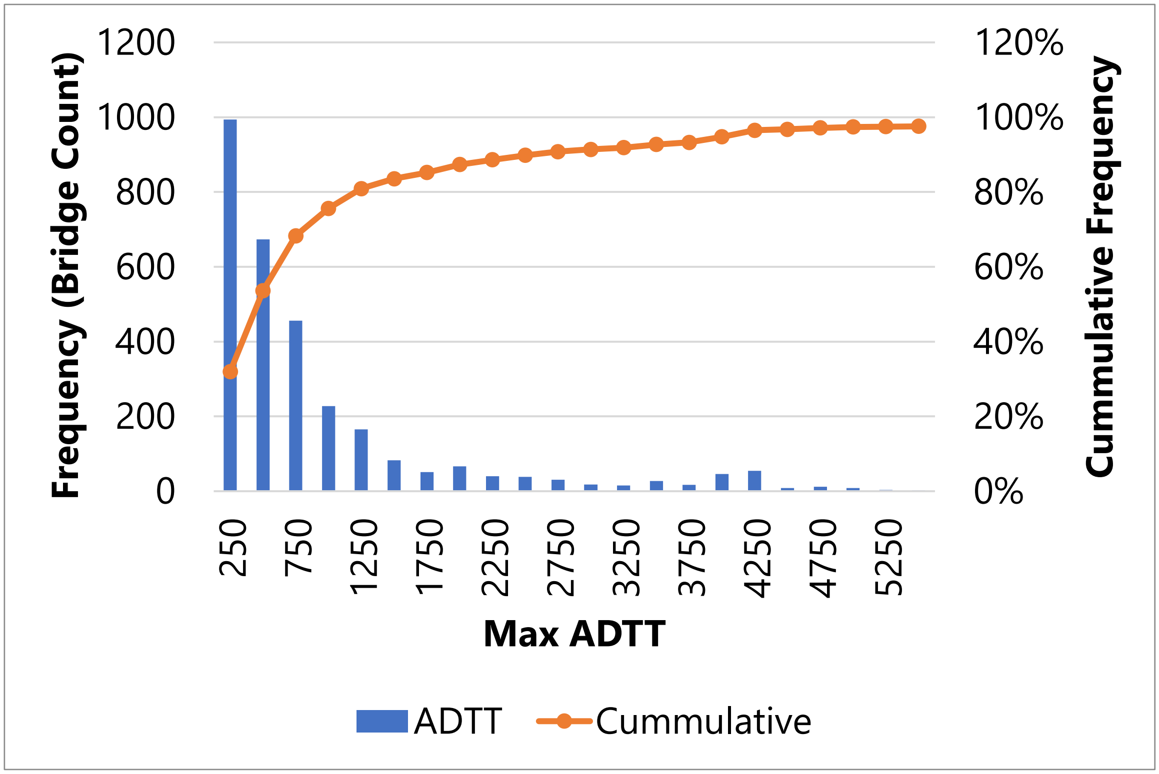

Figure 3. Distribution of Iowa bridges by ADTT. Iowa DOT SIIMS data set, sample size 3108 bridges.

Table 5. Average Daily Truck Traffic Statistics Summary

Count	Average ADTT	Minimum ADTT	Maximum ADTT	Standard Deviation	50th Percentile	75th Percentile	90th Percentile
3108	966	0	15498	1516	441	943	2520

Currently, Iowa DOT does not have a quantitative measure to differentiate between normal truck traffic demand and high-volume truck traffic demands for their bridges. Based on the statistical analysis, five (5) different ADTT ranges were considered to differentiate between "normal" and "high" truck traffic. These ADTT values were also used to investigate the top maintenance activities for bridges with high truck traffic and how sensitive these top maintenance items are to the selected definition of high truck traffic. The defined ADTT values are:

- 500 to 1,000 Trucks per day - per Iowa DOT LRFD Bridge Design Manual Section 5.2.4.1.2 which corresponds to 54th Percentile.
- 1,000 to 1,750 Trucks per day - 76th percentile and approximately the average ADTT among the 3108 bridges.
- 1,750 to 2,200 Trucks per day - 85th percentile

- 2,200 to 2,500 Trucks per day - Candidate Traffic Critical project for bridges with 11,000 & 20% Trucks or more per Iowa DOT design manual, Traffic Critical Projects Program.
- Greater than 2,500 Trucks per day - 90th percentile

3.3 Program Work Recommendations Data Set

A second maintenance data set (program work recommendations) was included in our analysis of top maintenance activities. The data set contains 1,978 entries covering “candidate” dates from 2001-2021. However, the program work recommendations data does not utilize the same maintenance codes used within the maintenance recommendations data set; rather, a description of the maintenance activity is provided instead. To compare and combine the two data sets, the description of each program work recommendation entry was used to categorize the maintenance activity and group them (whenever possible) with the maintenance items from the maintenance recommendation data set.

3.4 Breakdown of Maintenance Activities

Maintenance entries on each data set were grouped in five categories based on the maintenance codes used within the maintenance recommendation data set. These maintenance categories were Bridge Deck, Pavement & Rails, Bearings & Substructure, Superstructure, and Miscellaneous maintenance. Both the maintenance recommendations and program work recommendations data sets were combined to understand the categories that are most frequently in need of repair. Note that maintenance entries related to culverts, bridge removal, widening, and bridge replacement were excluded from the analysis.

As shown in Figure 4, most bridge maintenance is related to deck repairs (46% of total maintenance) and miscellaneous maintenance (28%); miscellaneous maintenance includes activities such as, berm erosion, tree and brush removal, and flood debris removal. Maintenance related to pavement & rails, bearings & substructure, and bridge superstructure accounted for 18%, 7%, and 5% of all the maintenance entries, respectively.

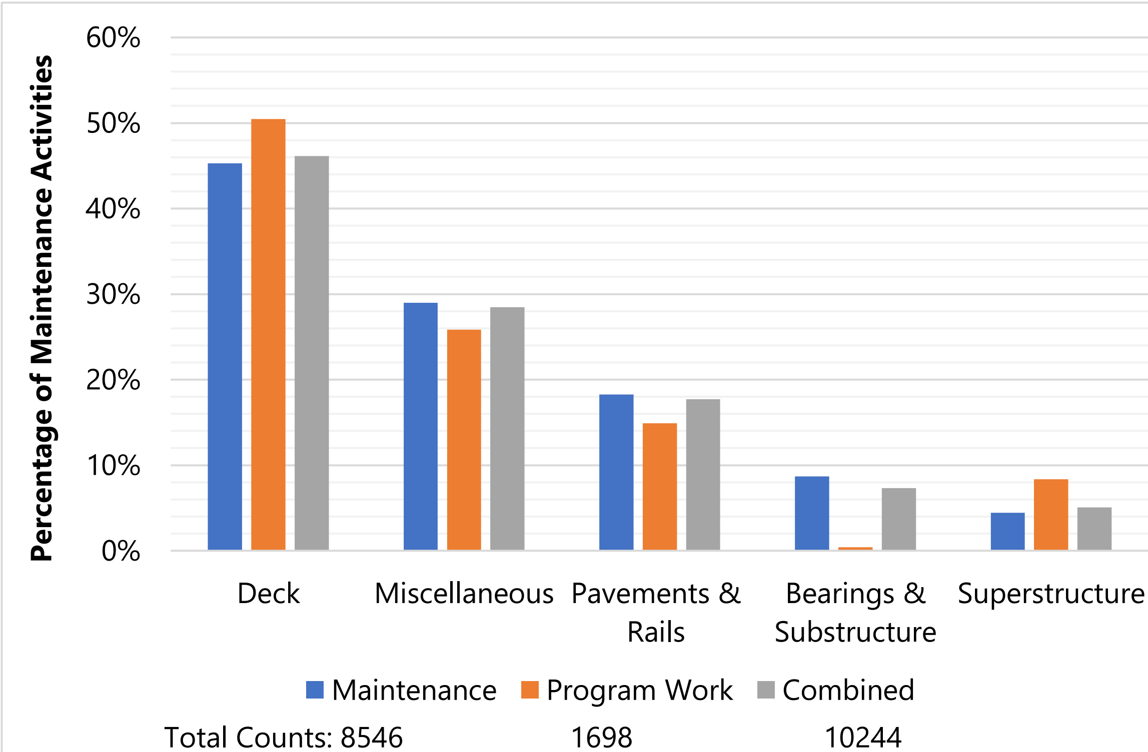

Figure 4. Maintenance Activities by Category

3.5 Top Maintenance Activities for Bridges with High Truck Traffic

Several of the maintenance codes and thus, maintenance entries included in the SIIMS dataset, refer to the same type of deterioration or maintenance activity. For example, under the bearings and substructure category, codes 313 to 315 refer to backwall repairs and the only difference among the codes is which backwall needs repair. Therefore, to determine which kinds of maintenance activities are most common, while accounting for the type of deterioration (and indirectly the underlying deterioration mechanism), several of the maintenance codes were combined under a single maintenance item before ranking them from most common to least common.

As previously mentioned, the top maintenance activities for bridges with low to high truck traffic were investigated for the five definitions of truck traffic volumes based on our selected ADTT ranges. To determine the top maintenance activities, the total number of entries in the data sets for each type of maintenance activities were used. The data is reported as a percentage of the total number of maintenance items for bridges with an ADTT equal to or greater than the selected ADTT value for truck traffic. Table 6 shows the total number of entries for each of the selected definitions for truck traffic for the maintenance recommendations and combined data sets.

Table 6. Total count of maintenance entries considered in the analysis of top maintenance activities

Heavy Truck Traffic Definition	Maintenance Recommendations Data Set	Combined Maintenance Recommendations & Program Work Recommendations Data Sets
ADTT ≥ 500	4199	5039
ADTT ≥ 1000	2178	2665
ADTT ≥ 1750	1299	1594
ADTT ≥ 2200	1053	1291
ADTT ≥ 2500	913	1111

Figure 5 and Figure 6 show how maintenance activities are distributed between the five general maintenance categories for the different ADTT values defining high truck traffic volumes. As previously discussed, most maintenance activities fall within the deck maintenance category and, as shown in Figure 5 and Figure 6, the top maintenance categories did not change based on the selected ADTT. However, the number of deck, bearings & substructure, and superstructure maintenance activities increase with an increasing ADTT. A similar analysis was conducted to investigate the sensitivity of the top maintenance activities relative to our definition of high truck traffic volumes. Figure 7 through Figure 16 show the top maintenance activities for the different maintenance categories and high truck traffic definitions defined in this study. Plots of the maintenance recommendations data set and the combined data sets are included. Table 7 provides a summary of the most common maintenance activities for the different categories for the combined data sets.

Table 7. Summary of most common maintenance activities (combined data sets)

Rank	Bridge Deck		Miscellaneous		Pavement & Rails		Bearings & Substructure		Superstructure	
	Activity	%	Activity	%	Activity	%	Activity	%	Activity	%
1	Repair spalls and hollows	18-28	Berm erosion repairs	7.0-8.3	Approach pavement repairs	6.0-9.4	Clean/paint seats & bearings	4.0-5.1	Zone or complete paint	1.7-2.5
2	Repair or replace joints	10	Trees & brush removal	1.1-5.8	Shoulder panel repairs	1.5-2.7	Backwall repairs	1.6-2.7	Repair/seal spalls	1.1-2.0
3	Deck overlays or new decks	6.0	Slope protection	2.2-3.3	Re-cut or re-install approach joints	1.9-2.3	Misc.	<1.0	Misc.	<1.0

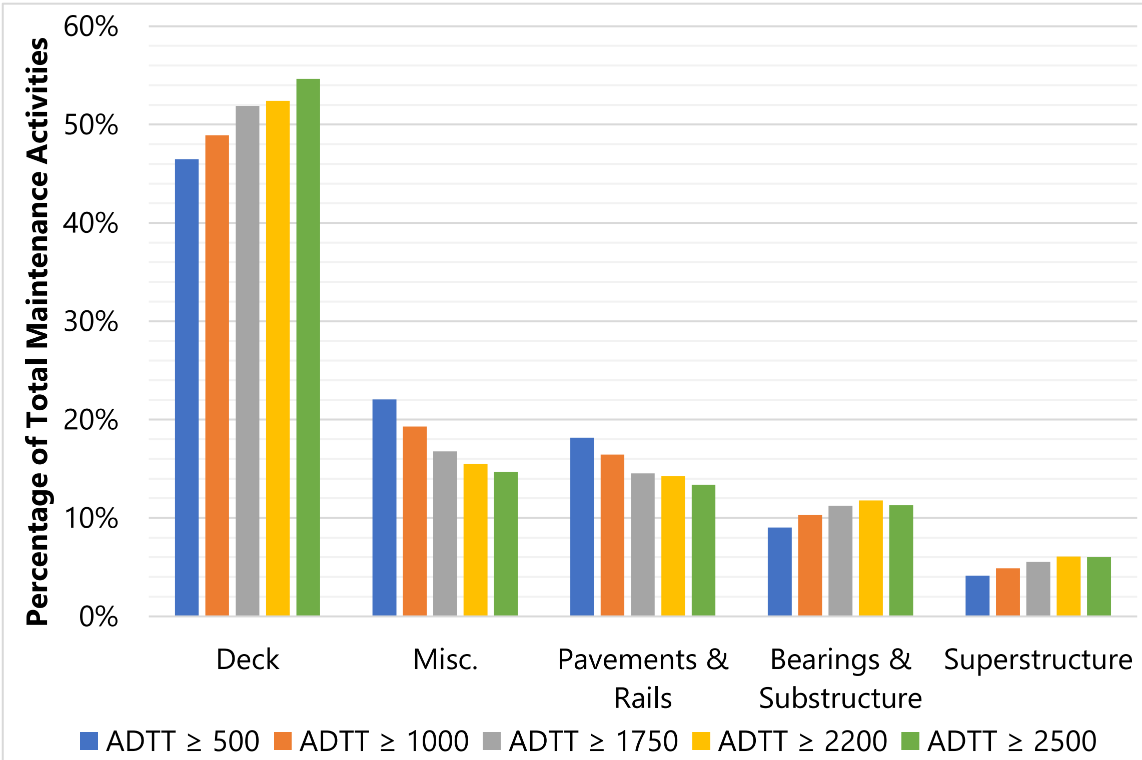

Figure 5. Fraction of Total Maintenance for Various Definitions of Heavy Truck Traffic.

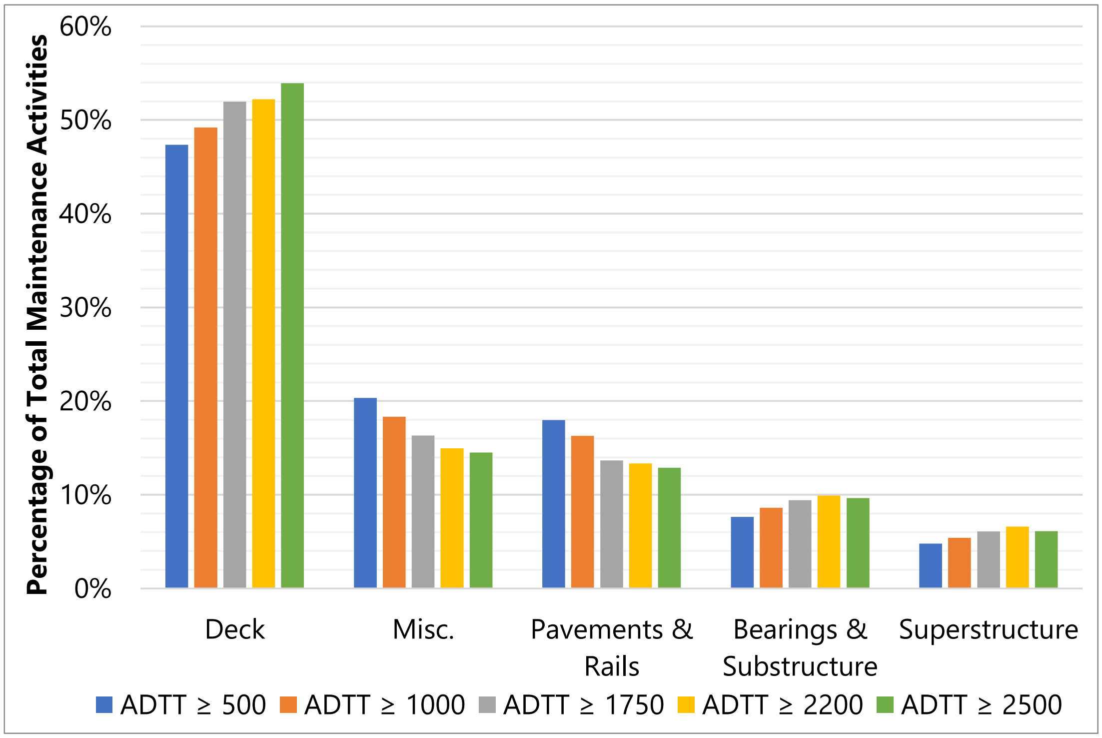

Figure 6. Fraction of Total Maintenance for Various Definitions of Heavy Truck Traffic (Combined Data Sets).

3.6 Discussion of Maintenance Activities for Bridges with High Truck Traffic

The analysis of bridge maintenance records allowed for the identification of the most common maintenance actions that Iowa DOT performs, recommends, and/or plans to implement to maintain their bridge inventory in a state of good repair. Based on both the maintenance recommendations and program work recommendations records, bridge deck repairs account for approximately 46% of all bridge maintenance efforts. The following discussion is focused on the top maintenance activities for bridges with high truck traffic, as defined based on the selected ADTT.

3.6.1 Deck Maintenance

The common different types for deck maintenance activities are summarized in Figure 7 and Figure 8, for the maintenance recommendations and the combined data sets, respectively. For the combined data sets (Figure 8), within the broad category of deck maintenance, repairs of spalls and hollows, joint replacement or repair, and deck overlay or new deck are the most common maintenance activities for bridges with high truck traffic. The occurrence of repair of spalls and hollows increases in frequency for bridges with higher ADTT, accounting for 18 to 28% of maintenance efforts for ADTT values of 500 and 2,500, respectively. Therefore, spalls and hollows appear to be 55% more common in bridges with ADTT of 2,500 than those with ADTT of 500. Deck joint repairs account for approximately 10% of total maintenance activities and showed little sensitivity to variation in ADTT. Furthermore, consideration of the program work maintenance recommendations revealed that deck overlays or new decks are the third most common maintenance activity and account for approximately 5.5 to 6.0% of maintenance efforts for ADTT values of 2,500 and 500, respectively.

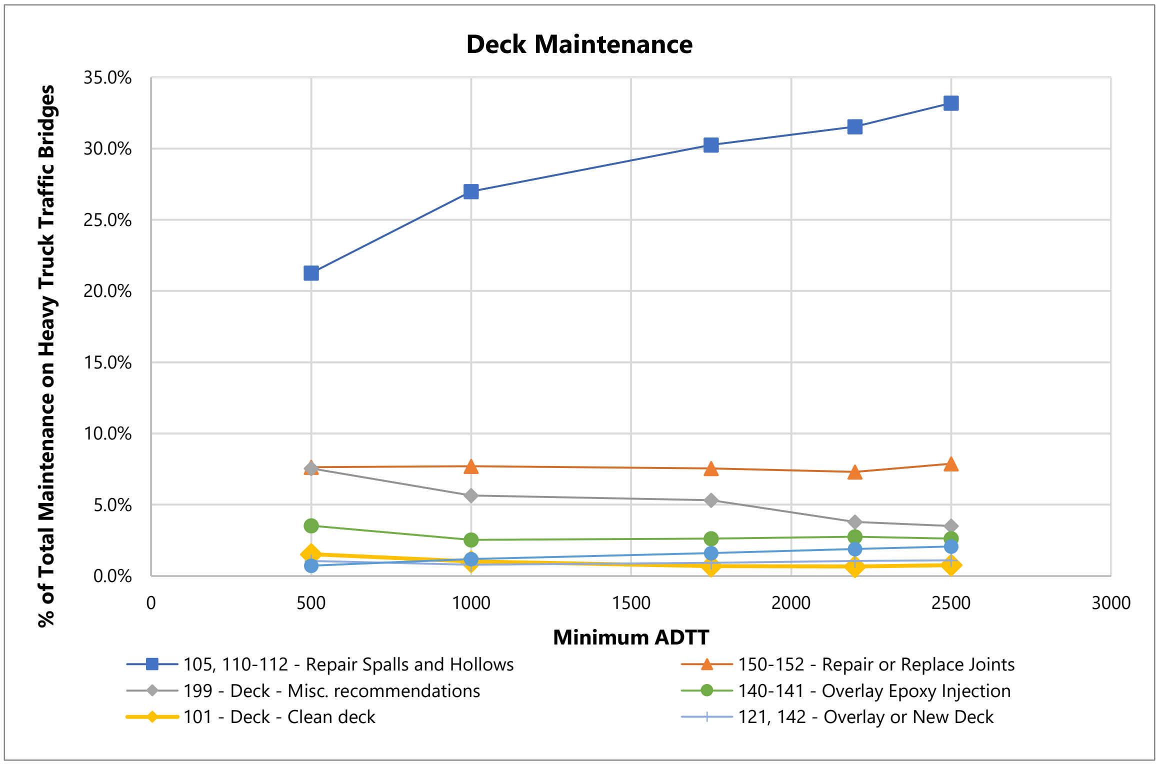

Figure 7. Top Deck Maintenance Activities for Varying Definitions of High Truck Traffic

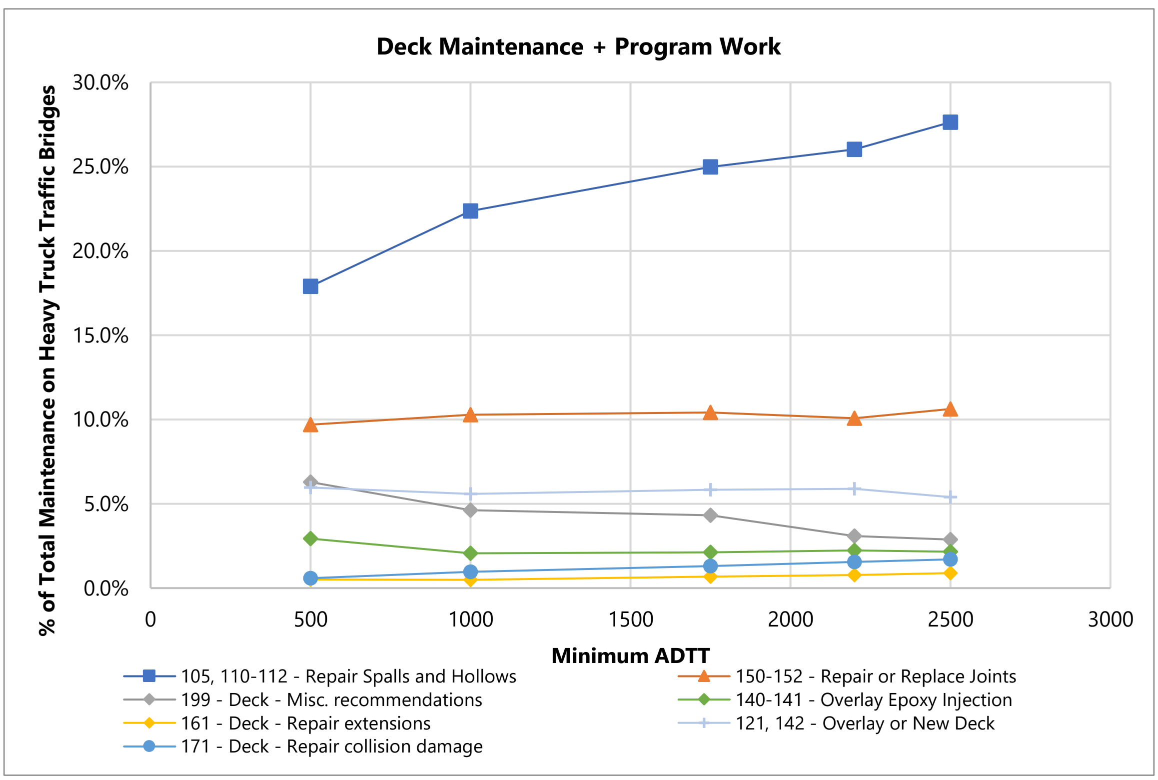

Figure 8. Top Deck Maintenance Activities for Varying Definitions of High Truck Traffic (Combined Data Sets)

3.6.2 Pavement & Rail Maintenance

The common different types for pavement and rail maintenance activities are summarized in Figure 9 and Figure 10, for the maintenance recommendations and the combined data sets, respectively. As shown in Figure 10, within the pavement category the most common maintenance activity is the repair of approach pavements which account for 6.0 to 9.0% of maintenance efforts for ADTT values of 2,500 and 500, respectively. Approach pavement repair records exhibit a decreasing trend with ADTT which is the opposite of the expected trend. Approach pavement repairs for bridges with ADTT of 2,500 or more appear to be 37% less common than for bridges with ADTT greater than or equal to 500. The reason for this apparent decrease in relative frequency of repair with increasing truck traffic may be partially attributed to variations in data collection and reporting. However, it is more likely that the reason for the observed trend is that approach pavements for bridges with higher truck traffic are built following more robust designs and, thus, require less maintenance than bridges with lower truck traffic demands; this hypothesis, however, has not been confirmed.

The second most common maintenance activity is shoulder panel repairs which account for approximately 1.5 to 2.7% of maintenance efforts for ADTT values of 2,200 and 500, respectively. A close third is re-cutting/re-installing approach joints which on average, account for 2.1% of maintenance efforts. Guardrail collision damage repairs generally account for 1.0 to 1.7% of bridge maintenance for ADTT values of 500 and 2,500, respectively.

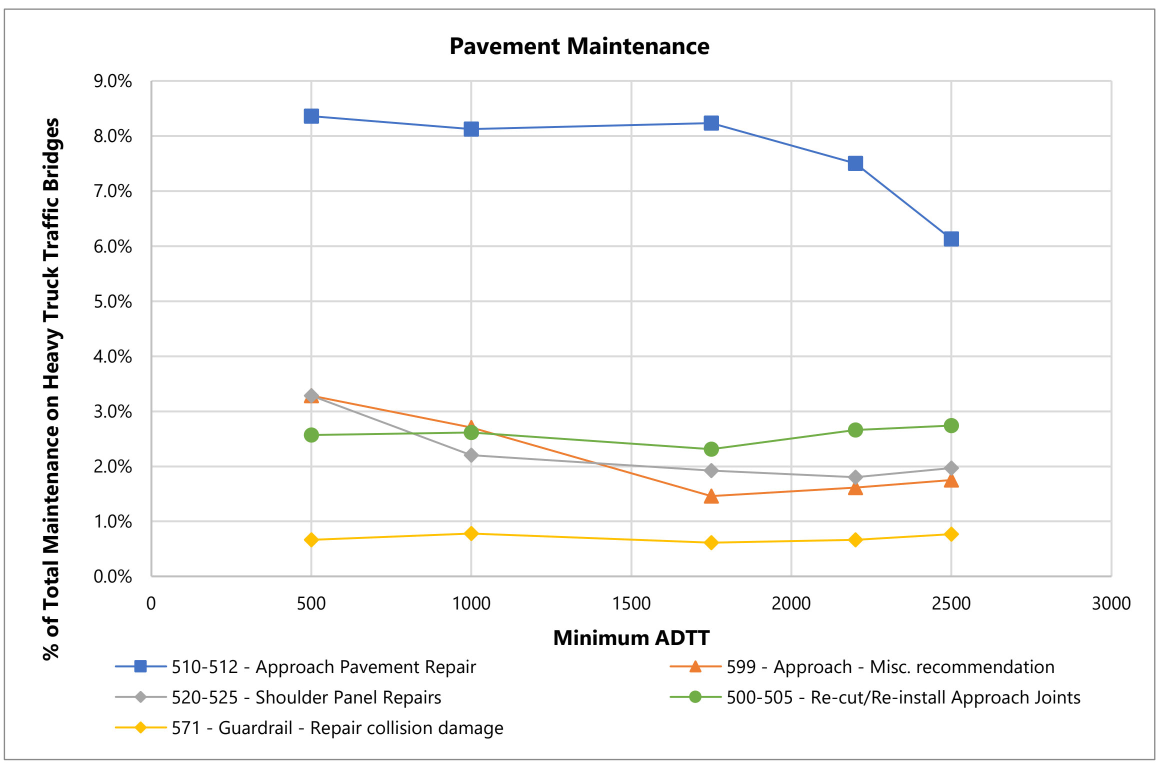

Figure 9. Top Pavement Maintenance Activities for Varying Definitions of High Truck Traffic

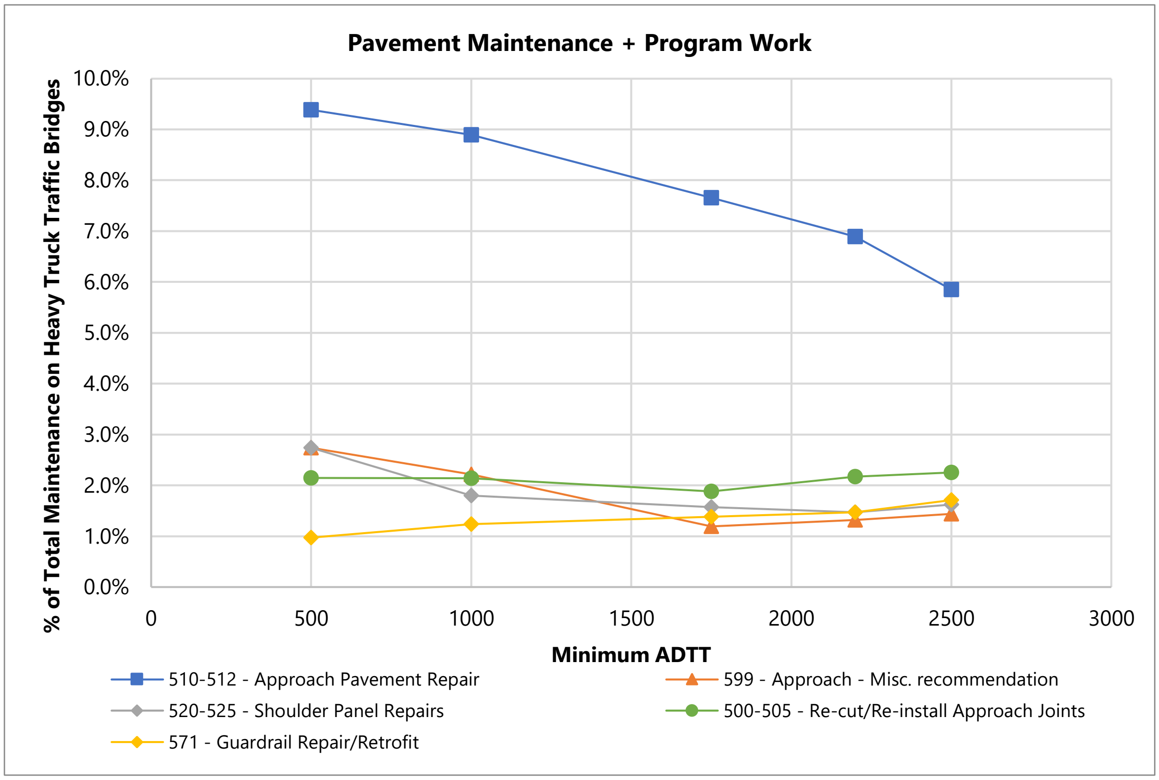

Figure 10. Top Pavement Maintenance Activities for Varying Definitions of High Truck Traffic (Combined Data Sets)

3.6.3 Bearings & Substructure Maintenance

The common different types for bearings and substructure maintenance activities are summarized in Figure 11 and Figure 12, for the maintenance recommendations and the combined data sets, respectively. As shown in Figure 12, within the bearings & substructure category the most common maintenance activity is cleaning and painting seats and bearings, which account for 4.0 to 5.1% of total maintenance for ADTT values of 500 and 2,200, respectively. The second most common maintenance is backwall repairs, which account for 1.6 to 2.7% of total maintenance for ADTT values of 500 and 2,500, respectively. Other maintenance activities generally account for 0.5% or less of all maintenance. No significant changes were observed when considering both data sets. However, cleaning and painting of bearings and seats appears to be about 29% more common for bridges with ADTT greater than or equal to 2,200, relative to bridges with ADTT of 500 or greater. Similarly, backwall repairs are 64% more common for bridges with ADTT greater than or equal to 2,500, relative to bridges with ADTT of 500 or greater.

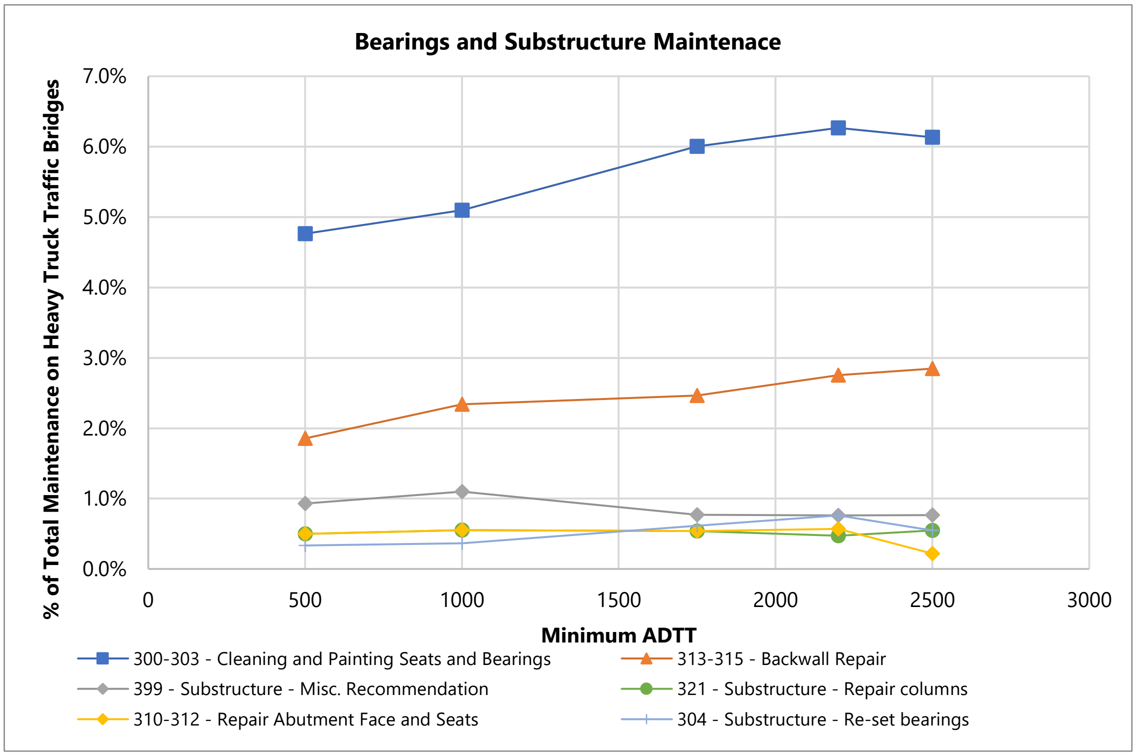

Figure 11. Top Bearings and Substructure Maintenance Activities for Varying Definitions of High Truck Traffic

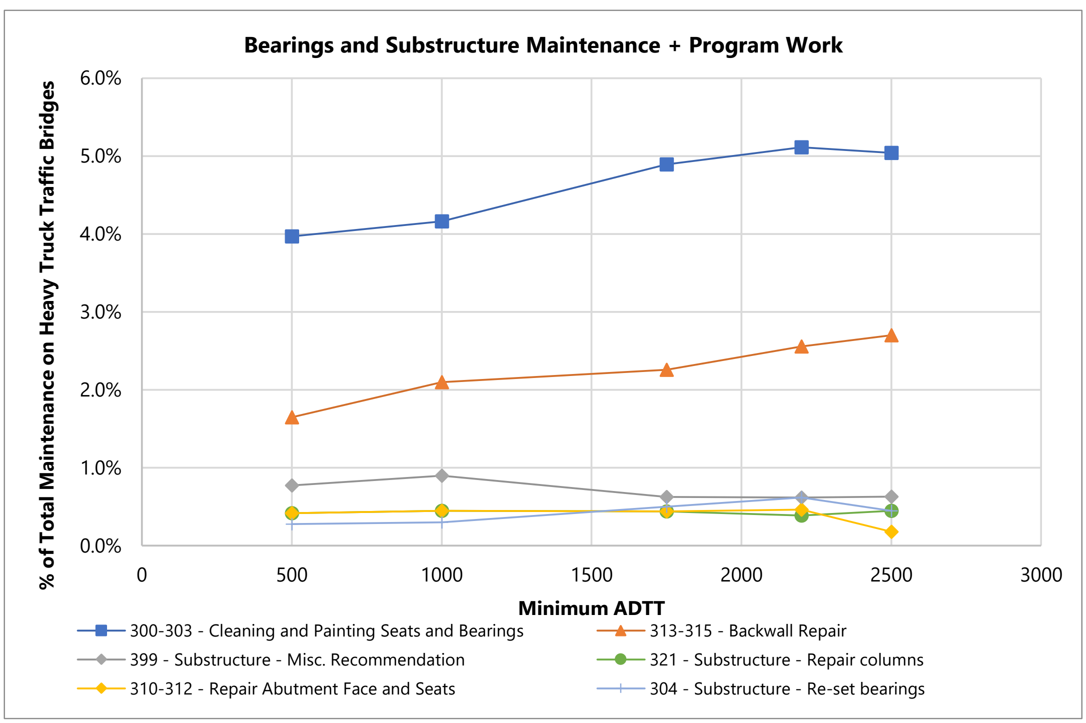

Figure 12. Top Bearings and Substructure Maintenance Activities for Varying Definitions of High Truck Traffic (Combined Data Sets)

3.6.4 Superstructure Maintenance

Superstructure maintenance is generally less common accounting 4.0 to 6.0% of all maintenance efforts. The common different types for superstructure maintenance activities are summarized in Figure 13 and Figure 14, for the maintenance recommendations and the combined data sets, respectively. The most common maintenance activity is "zone or complete painting" of steel structures, which accounts for 1.7 to 2.5% of all maintenance for ADTT values of 2,500 and 2,200, respectively (see Figure 14). Painting of concrete structures was also included within the programed work; however, there were less than 10 instances of concrete bridge paint within the program work records. Following painting of steel structures, the second most common maintenance is repairing and sealing spalls (approximately 1.0 to 2.0% of all maintenance); spall repairs become more prevalent for bridges with higher ADTT.

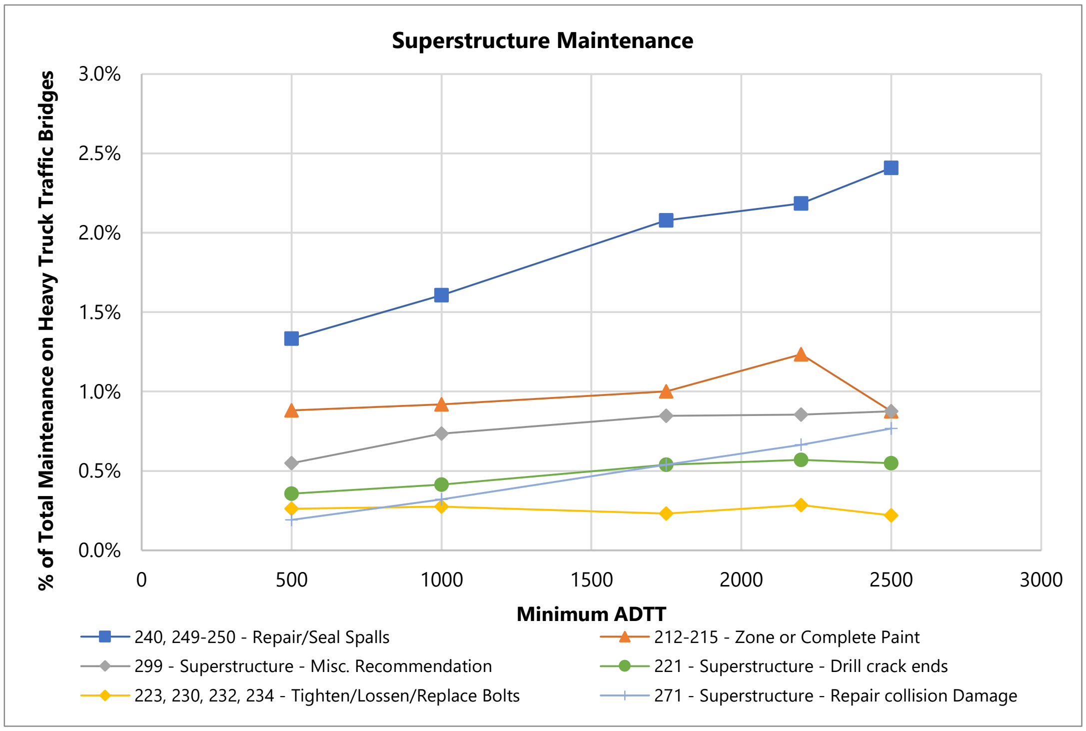

Figure 13. Top Superstructure Maintenance Activities for Varying Definitions of High Truck Traffic

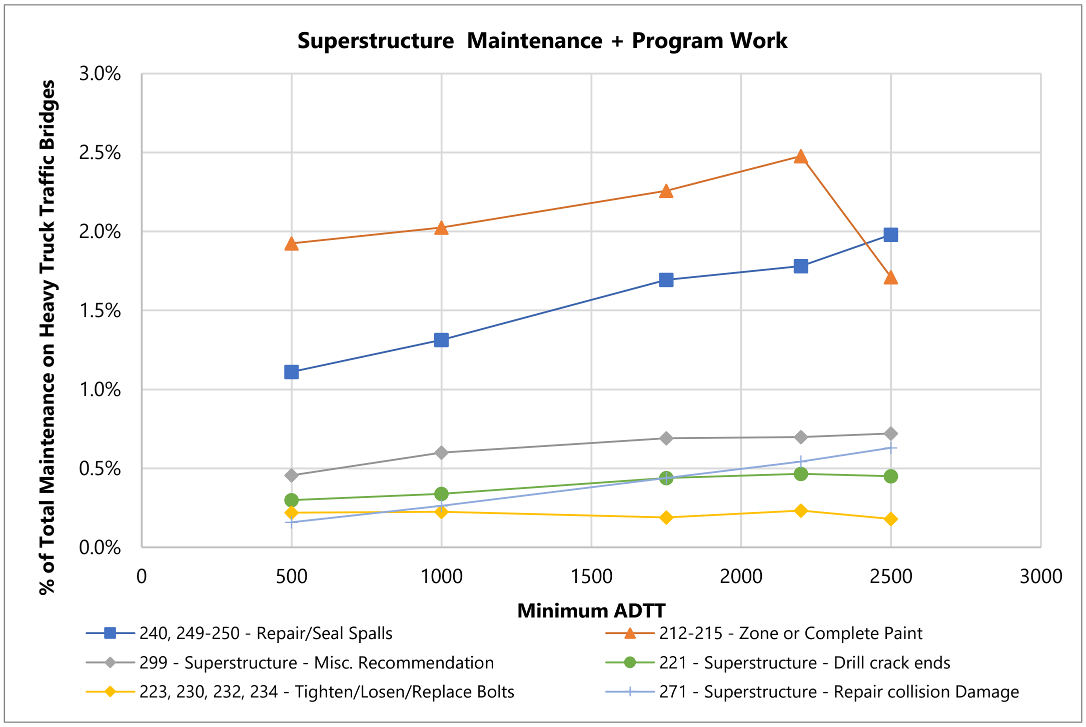

Figure 14. Top Superstructure Maintenance Activities for Varying Definitions of High Truck Traffic (Combined Data)

3.6.5 Miscellaneous Maintenance

Miscellaneous maintenance is the second most common category of maintenance and accounts for 14 to 20% of all maintenance. The common different types for miscellaneous maintenance activities are summarized in Figure 15 and Figure 16, for the maintenance recommendations and the combined data sets, respectively. The most common activities within the miscellaneous category are berm erosion repair, and "tree and brush removal" which account for 7.0% (ADTT of 2,500) to 8.0% (ADTT of 500), 1.0% (ADTT of 2,500) to 6.0% (ADTT of 500) of all maintenance, respectively (see Figure 16). Tree and brush removal is over 5 times more common for bridges with ADTT of 500 and greater than for bridges with ADTT greater than or equal to 2,500. A potential explanation for this observation is that bridges with the highest ADTT (2,500 or more) are likely located in urban areas and inherently require less vegetation control than bridges in rural areas, which can be reasonably assumed to have lower truck traffic demands.

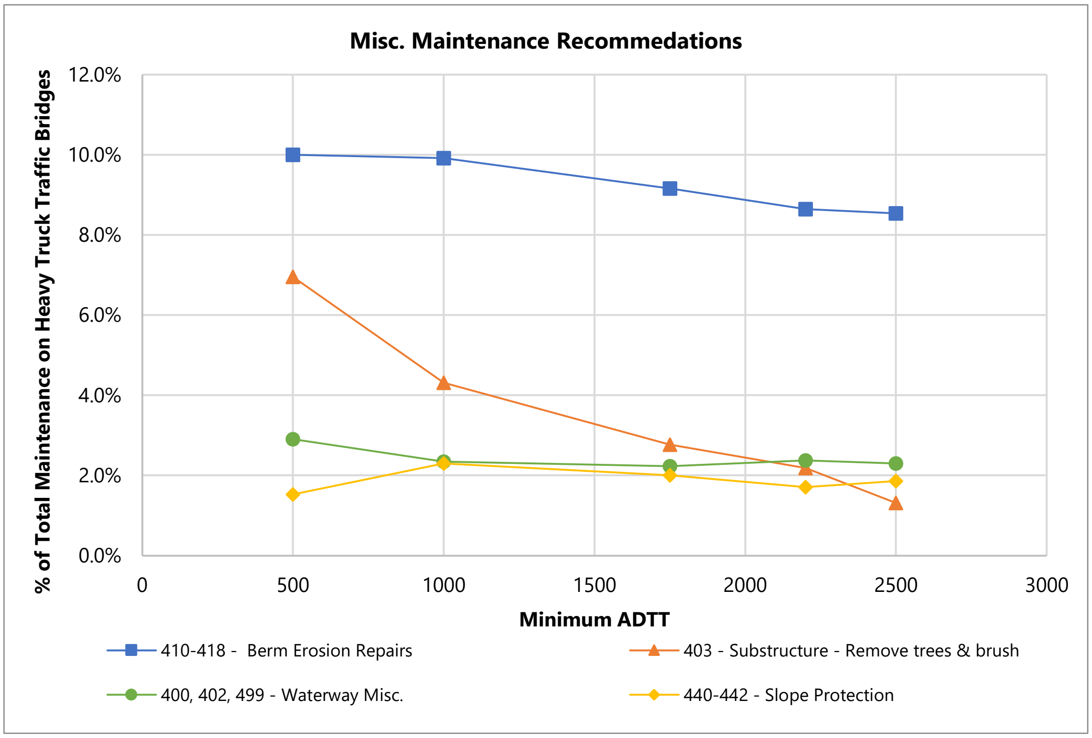

Figure 15. Top Miscellaneous Maintenance Activities for Varying Definitions of High Truck Traffic

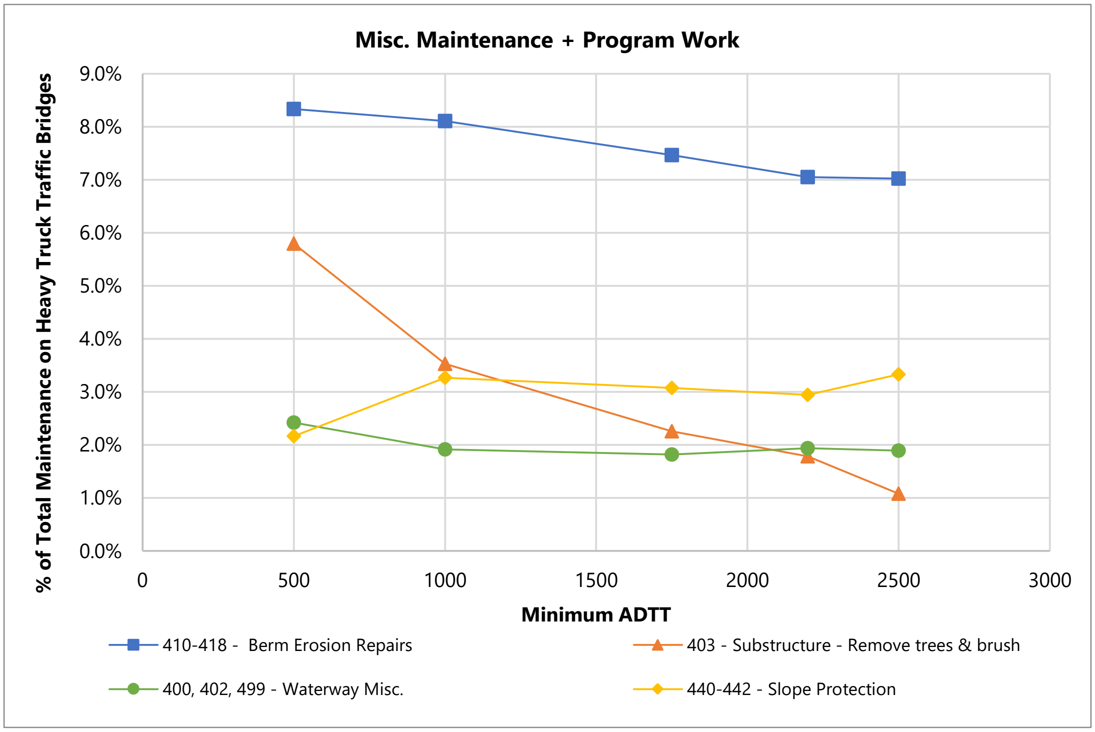

Figure 16. Top Miscellaneous Maintenance Activities for Varying Definitions of High Truck Traffic (Combined Data Sets)

3.7 Closing

In general, the ranking of most maintenance activities appears to be insensitive to ADTT values between 500 and 2,500. Per Iowa DOT design manual, Traffic Critical Projects Program, 2,200 trucks per day are Candidate Traffic Critical project for bridges with 11,000 & 20% Trucks or more. The larger variations in results when traffic is considered were observed for deck spall repairs, approach pavement repairs, and tree and brush removal. Deck spall repairs were found to be approximately 55% more common in bridges with ADTT greater than or equal to 2,500 than for those with ADTT of 500 or greater, suggesting higher truck volumes cause increased deck spalling and distress. Additionally, the deck, bearings & substructure, and superstructure maintenance categories generally account for a higher proportion of total maintenance effort with increasing ADTT.

Approach pavement repairs were found to be 37% less common in bridges with ADTT greater than or equal to 2,500 compared to bridges with ADTT greater than or equal to 500. The most plausible explanation for the observed trend is that approach slabs are built following more robust construction or design standards for bridges with higher truck traffic demands (e.g., ADTT ≥ 2,500). A decreasing trend with ADTT was also observed for tree and brush removal, which is over five times more common on bridges with ADTT of 500 to 1000 than those with higher ADTT (≥2,500). This may be attributable to the location of the bridges; the hypothesis is that bridges with higher ADTT (e.g., ≥ 2,500) are likely located in urban areas and inherently require less vegetation control than bridges in rural areas with lower truck traffic demands.

CHAPTER 4. SURVEY OF BRIDGE MAINTENANCE NEEDS ACROSS IOWA

A survey of local Iowa DOT jurisdictions was conducted that contained 23 questions grouped into four topics:

- General information on maintenance and deterioration,
- Bridge decks, joints, and railings,
- Superstructures, substructures, and bearings, and
- Foundations, pavements, and approaches.

A total of 38 completed surveys were received which included responses from 24 different zip codes across the state. All blank responses were omitted from the data analysis. Figure 17 shows the 24 different areas of the state from which a response was received, which shows a good geographical distribution across the state.

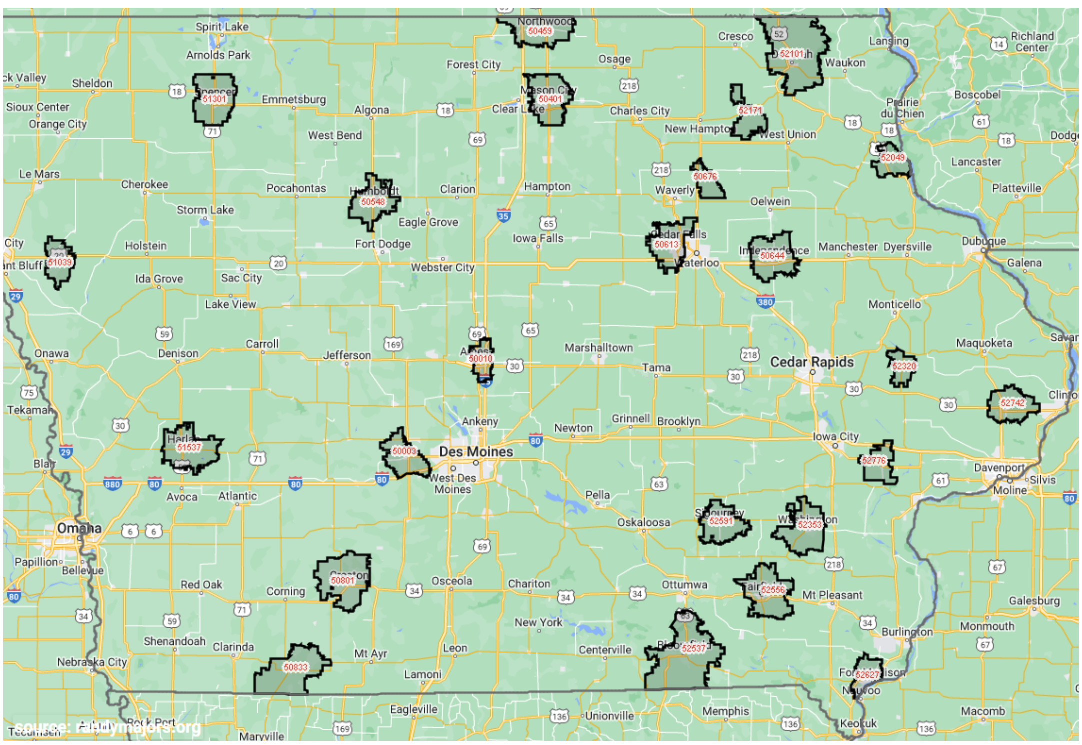

Figure 17. Map of the state of Iowa highlighting the locations that responded to the survey.

4.1 Summary of Findings

In general, winter maintenance activities were selected as the most impactful maintenance actions to bridge durability. Specifically, the application of de-icing salts, frequency of application, and type of de-icing salts were selected by 50%, 45% and 32% of respondents as the most impactful activities to bridge durability. Outside of winter maintenance, the application of sealants, protective coatings, and paints was selected as highly impactful to bridge durability by approximately 24% of respondents. These findings are consistent with the most common deck deterioration mechanisms as identified by Iowa DOT on this survey, which are, reinforcement corrosion and freeze-thaw. Moreover, the findings are also consistent with the components that most frequently need maintenance or repair, bridge decks, joints, and parapets/railings. The aggressive corrosive environment created by the use and frequency of application of de-icing salts is clearly one of the major causes of deterioration. These findings are also consistent with the maintenance data from Iowa DOT's Structure Inventory and Inspection Management System (SIIMS). It is therefore no surprise, that the three bridge components for which Iowa DOT desires to eliminate or minimize maintenance are bridge decks, joints, and railings which 55%, 32%, and 16% of respondents, respectively, identified as the most desirable components to eliminate or reduce maintenance (see responses to Question 3 in the following section). Notably, 13% and 16% of respondents indicated that the superstructure and bearings never need repairs.

Iowa DOT respondents were asked to suggest protective design strategies to help reduce bridge maintenance. Among these were: the use of impermeable concrete mixes and slower cement hydration mixes; steel protective systems such as galvanized steel; elimination of deck surface grinding that may lead to cracking and thus reduce deck durability; the use of non-corrosive de-icing chemicals; elimination of wood/timber bridge components; and removing joints near abutments, among others.

The most common deck maintenance activities appear to be cleaning/washing drains, repairing spalls and hollows, and replace/repair joints which were identified as the most frequent maintenance activities by 42%, 26%, and 10.5% of respondents, respectively. As previously mentioned, reinforcement corrosion and freeze-thaw deterioration are the most common issues affecting the bridge decks. To mitigate reinforcement corrosion, 82% of respondents indicated that they use epoxy coated bars for the top and bottom mats of deck reinforcement.

The most common types of joints across Iowa DOT bridges appear to be strip seal joints, compression joints, and jointless designs, which 26%, 10.5% and 24% of respondents indicated are most common. Note however, that jointless designs responses were evenly split with approximately 24% of respondents indicating they are the most common joint design and that they are not applicable. One potential explanation for the even split is that most of the responses indicating that jointless designs are most common appear to correspond to rural areas and likely smaller, single span or concrete slab bridges, whereas several of the responses indicating that jointless designs are rare or not applicable were received from larger urban areas of the state. The most common issues with joints are accumulation of debris, failed compression or strip seal joints leading to leaks.

Regarding to bridge railings, the most common issues are reinforcement corrosion, freeze-thaw deterioration, and railing anchor corrosion, with 40%, 10.5%, and 8% of respondents, respectively, indicating these are often an issue. According to 34% and 24% of respondents, the most common issues with steel railings are impact damage and steel corrosion, respectively.

Superstructure maintenance is primarily focused on the repair/seal of spalls, repair of collision damage, and zone or complete paint, as indicated by 24%, 16% and 8% of respondents, respectively. The most common superstructure durability issues are steel corrosion, reinforcement corrosion, and freeze-thaw deterioration, as indicated by 34%, 21%, and 10.5% of respondents, respectively. Note that despite steel corrosion being the most common issue, zone or complete painting was not as frequently needed as repairs of spalls and collision damage. The reason for the mismatch in frequency of maintenance and frequency of occurrence is not clear.

The most needed substructure maintenance action, as indicated by 21% of respondents is abutment backwall repairs, followed by column repairs (10.5% of respondents), and abutment face and seats repairs (5% of respondents). The main deterioration mechanisms affecting the substructure, specifically the piers, walls, and abutments are reinforcement corrosion and freeze-thaw, which were identified by 18% and 13% of respondents, respectively, as being often an issue. Approximately 13% of respondents also indicated alkali-aggregate is often an issue affecting the substructure; however, approximately 53% of respondents indicated alkali-aggregate, is rarely or never an issue. Therefore, alkali-aggregate is likely not a prevalent issue among Iowa DOT bridges.

The survey revealed that the most common type of bearings among Iowa DOT bridges is the fixed bearing, as indicated by 21% of respondents. However, elastomeric, sliding plate, and rocker bearings appear to also be common, as indicated by 13%, 8%, and 8% of respondents, respectively. The most prevalent issues affecting bearing durability were reported to be steel corrosion and deterioration due to joint leaks with 32% and 34% of respondents indicating these are often an issue affecting bearings. As part of their repairs and new designs, Iowa DOT most commonly specifies the use of fixed bearings (32% of respondents); however, elastomeric and sliding plate bearings are also commonly specified according to 26% and 21% of respondents, respectively (see Table 11).

The most frequent pavement maintenance actions are spalls/pothole repairs at bridge approaches and slope erosion, as reported by 45% and 42% of respondents. Pavement repairs at the bridge ends was also reported to be often needed by 37% of respondents. Note that the most common issue affecting bridge approaches is settlement, as reported by 81%of respondents (see Table 12). Other common issues are potholes/spalls, cracking, erosion, and undermining.

Regarding bridge foundations, timber decay is, by far, the most common problem as reported by 61% of respondents. Based on several of the survey answers, Iowa DOT appears to have many older bridges with wood piles which are often in need of repair or replacement due to pile deterioration. Other common issues were foundation undermining and scour, as reported by 26% and 21% of respondents, respectively.

The survey summarized herein confirmed the findings of the analysis of the SIIMS maintenance data set and provided insight into specific issues faced by Iowa DOT engineers in their maintenance efforts. Notably the most impactful actions to bridge durability are associated with winter maintenance (de-icing) and the most critical components to bridge durability are decks, joints, and railings. Note however, that approach pavement repairs were among the most frequent maintenance actions based on SIIMS dataset. Unfortunately, this option was not included among the possible answers to Question 2 and thus, a direct comparison to other bridge components is not possible. Another insight obtained from this survey is that there appear to be a large inventory of timber pile supported bridges which have deteriorated and are often in need of repair or replacement.

4.2 Summary of Survey Responses

4.2.1 General Information on Maintenance and Deterioration

***Question 1**: Identify the factors that your agency considers to have a significant impact on bridge durability and service life within your jurisdiction. Please indicate whether the item listed has low, moderate, or high impact in your experience.*

Seven different actions were provided for the local jurisdictions to consider and rate how impactful these actions are to bridge durability and service life. Figure 18 summarizes the answers received showing the number of answers corresponding to Negligible, Low, Moderate, and High impact to bridge durability.

The top three most impactful actions were the application of de-icing salts, the frequency of application of de-icing salts, and the type of de-icing salts. Approximately 95% of the respondents indicated that application of de-icing salts is highly or moderately impactful to bridge durability. Similarly, approximately 84% and 76% of respondents indicated that the frequency of application and type of de-icing salts, respectively, are highly or moderately impactful to bridge durability. Figure 19 shows the distribution of responses for these three actions. Notice that all three of these actions are related to winter maintenance activities. The fourth most impactful action was the application of sealants, protective coatings, or paints, which 71% of the respondents indicated was either highly or moderately impactful to bridge durability.

Survey respondents were able to add to the list of impactful actions and several of the responses where related to bridge decks such as, deck cracking and deck permeability (2 respondents). The type of reinforcement and reinforcement coating was also reported. All these added responses were deemed as highly impactful to bridge durability and service life.

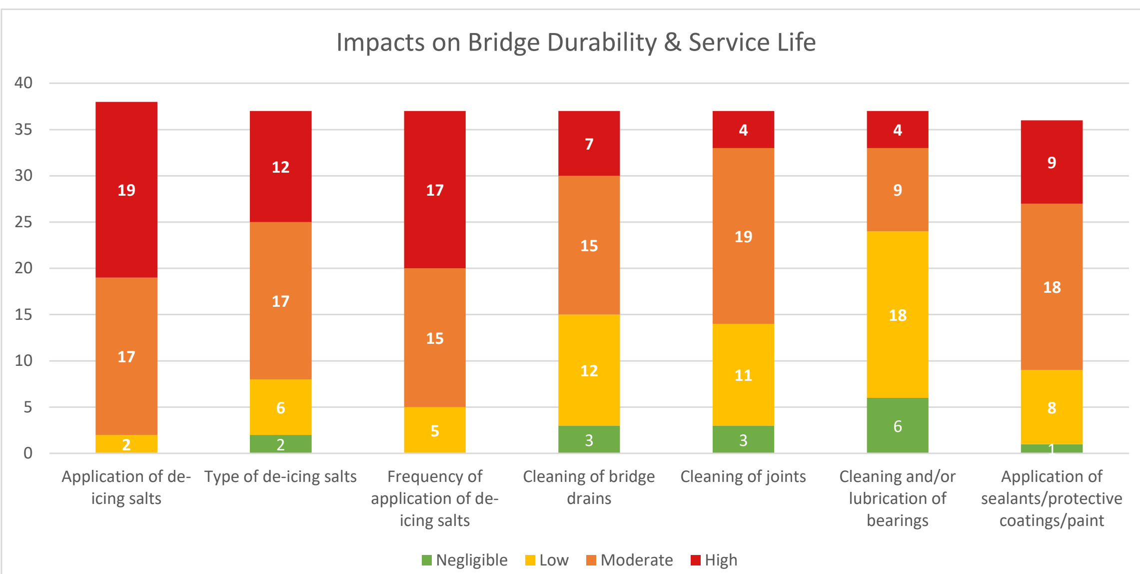

Figure 18. Actions affecting bridge durability and service life based on Iowa DOT local jurisdictions

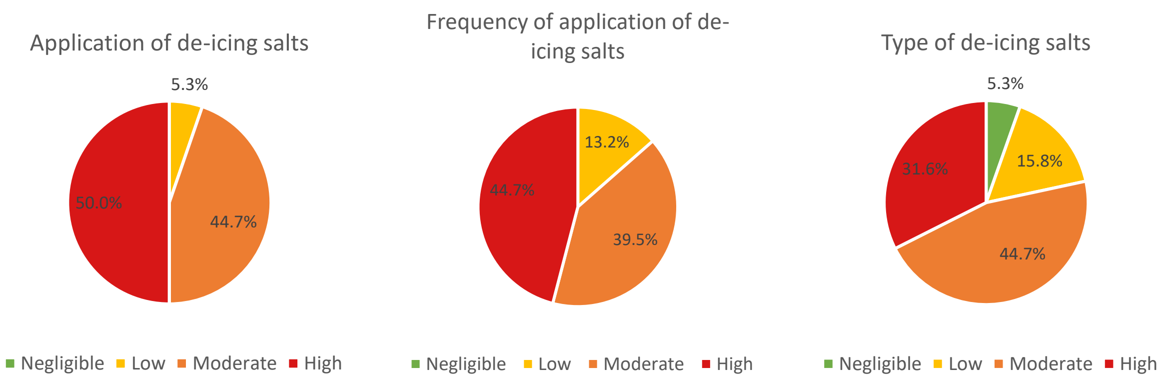

Figure 19. Most impactful actions affecting bridge durability and service life

***Question 2**: Indicate how frequently repair or service of each bridge component is needed.*

The purpose of this question was to assess maintenance needs; thus, respondents were requested to indicate how frequently such components require repair, regardless of whether repairs are performed by the agency. Seven bridge components were included in the survey as well as the ability to add up to three other components. Figure 20 summarizes the answers received showing the number of answers corresponding to Never, Rarely, Occasionally, and Often needs of maintenance/repair.

Bridge decks and joints were reported as the components that most often need repair, consistent with the data from the Iowa DOT SIIMS maintenance recommendations data set. Approximately 26% and 29% of the respondents (92% and 84% for occasionally and often combined) indicated that bridge decks and joints, respectively, are often in need of repair. Railings and parapets appear to be the third type of component that most frequently needs repairs with approximately 47% of the respondents indicating that these occasionally or often need repairs. Note that 13% and 16% of respondents indicated that the superstructure and bearings never need repairs. Figure 21 shows the distribution of the responses for the top three most often repaired bridge components.

At least one respondent indicated that approaches, piles, H-Pile encasements, or the substructure are often in need of repair.

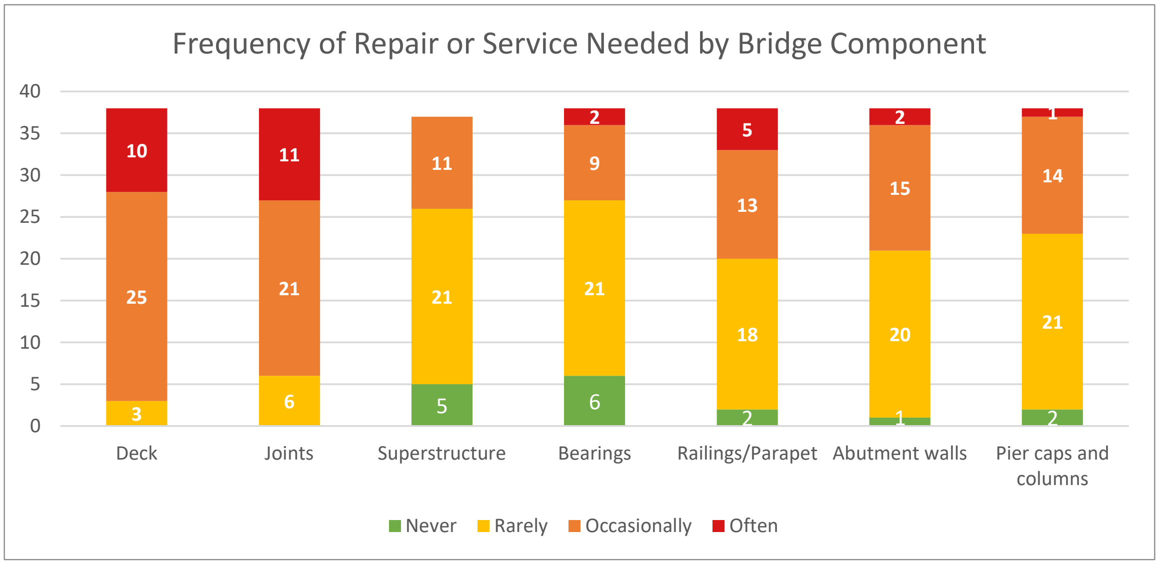

Figure 20. Summary of component maintenance and repair needs

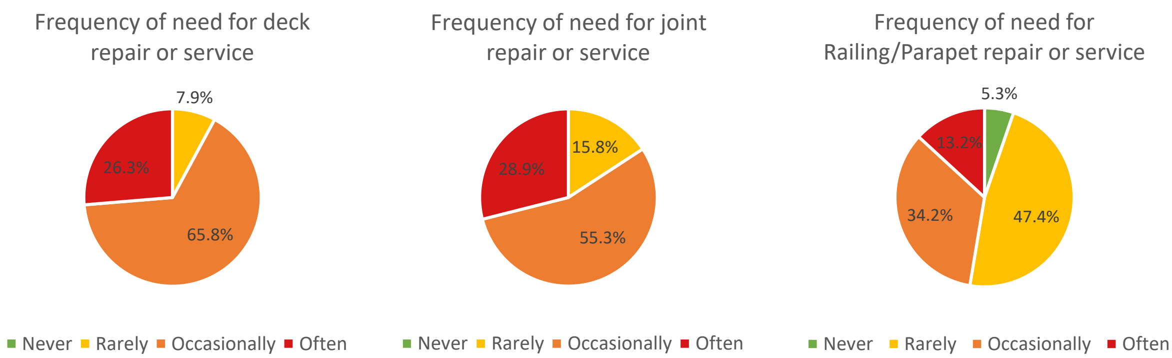

Figure 21. Most frequently repaired bridge components.

Question 3: *For which bridge components is it most desirable to your agency to minimize or eliminate maintenance? Please describe the components or design details and the degradation mechanism you would like to be addressed, and why a no-maintenance design is important to your agency (e.g., lower costs, less demand on maintenance personnel, less traffic closure time, etc.).*

This was an open question for which each respondent had the opportunity to elaborate on which components is it most desirable to minimize or eliminate maintenance. Table 1 summarizes the bridge components that were most mentioned as desirable for minimizing or eliminating maintenance, as well as how many respondents mentioned the same component, reasons for the desire of maintenance elimination, and (when included) the deterioration mechanisms mentioned for each component. Bridge deck repairs and joint repairs were the top components for which minimum or no maintenance is desired.

Table 8. Summary of most desirable components for maintenance reduction or elimination.

Bridge Component	Count	Reasons	Deterioration Mechanism
Deck	17	Limit repair crew exposure to traffic; minimize or eliminate traffic disruptions/road closures and associated costs and economic impact; deck repairs account for most of the DOT's time and repair costs.	Spalling and corrosion
Joints	10	Account for most of the DOT's time and repair costs; repair of joints is costly and difficult; not clear how to repair and address joint failures.	Joint leaks and failures lead to drainage issues, bearing corrosion, and allow de-icing salts to reach piles
Rails	5	Significant time spent on rail repairs; minimize traffic disruptions and associated costs/economic impacts	-
Approaches	4	Minimize or eliminate traffic disruptions/road closures and associated costs and economic impact	Approach settlement relative to bridge deck leads to accelerated deterioration
Piers	4	Piers are difficult to access and repair on small bridges; pier encasements have been problematic, solutions are needed	-
Abutment walls	3	-	-
Piles	3	Pile deterioration is the cause of most bridge replacements; difficult to repair/rehabilitate timber piles; cost effective repairs for timber piles are needed.	Degradation and rotting of timber piles

** Language paraphrased from the respondent answers. 31 of 38 respondents provided an answer to this question.*

Question 4: *Are there any particular protective design strategies that you think should be considered to reduce routine maintenance activities? If so, please describe.*

Table 9. Protective Design Strategies Recommended by Local Iowa DOT Jurisdictions

Answers
Slower cement hydration. Mixtures that are largely impenetrable. Design for impermeable surfaces and runoff for salt application. Then have an aggressive program of sealing to keep the salt out. Steel items well galvanized and easily replaceable by design.
No wood in the structure and ensure the bridge is no less than 28ft wide.
Serviceable Bearings
Elimination of guardrail requirements on low volume gravel roadways.
Improved rebar such as galvanized rebar
Stop grinding deck surfaces. I understand it's for a smoother ride because our contractors can't or don't worry about smooth surface any more since we allow them to grind. It opens up the top of our deck to allow more penetration of deicers in.
Application of a high-quality deck sealant immediately following bridge deck construction. Roadway design in which bridge approaches are level with bridge deck.

Answers
Concrete additives that are the most effective. Deck and barrier rail coatings are invasive budgetarily and from a time standpoint. I would be willing to pay more upfront to avoid these maintenance costs.
Reduction of chloride intrusion - having a tighter or impermeable deck surface
Epoxy coating the deck to eliminate deicing chemical corrosion or look at a non-corrosive deicing chemical. Bridge decks on gravel roads seldom require patching and/or deck overlays
Need to look into better concrete sealers deck protection.
Not have the joint at the abutment. Pour the deck solid over the abutment and have the joint 20' or 30' away on a sleeper slab.
The concrete deteriorates. Something resistant to deterioration like a poly would be helpful.
Frequent sealing of concrete rails and outside of bridge deck on continuous concrete slabs and beam bridges with open rail systems.

4.2.2 Bridge Decks, Joints, and Railings

Question 5: *Identify the most frequent deck maintenance actions. Please indicate how frequently each is needed.*

This question was aimed to understand which deck maintenance activities are most frequent throughout the state of Iowa. For this purpose, six maintenance actions were provided for local DOT jurisdictions to indicate how often are each of the maintenance actions needed; respondents were able to add up to three maintenance actions beyond those provided. Figure 22 summarizes how often are each of the six maintenance actions needed.

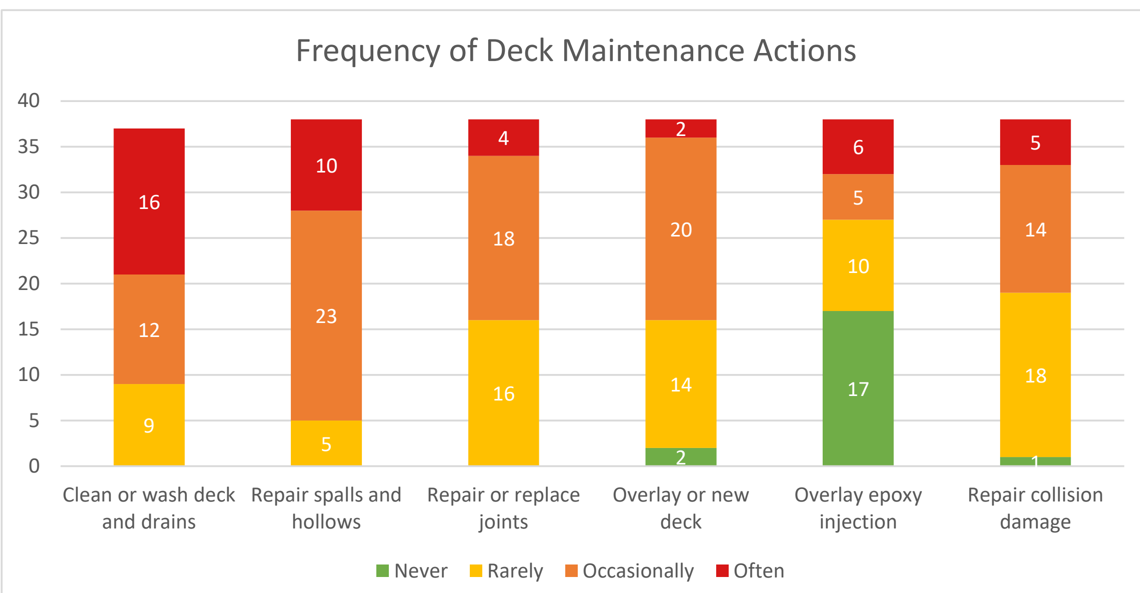

Figure 22. Frequency of deck maintenance actions

As shown in Figure 22, the three most common deck maintenance actions are the repair of spalls and hollows, cleaning or washing deck and drains, and repairing or replacing joints. Approximately 87% of respondents indicated that repair of spalls and hollows are occasionally or often needed; 74% indicated that cleaning deck or drains are occasionally or often needed; and 58% indicated that replacing or repairing joints are occasionally of often needed. Note however, that cleaning/washing decks and drains appears to be the most needed maintenance action with 42% of the respondents indicating it is needed often. Figure 23 shows the distribution of responses for the top three most common deck maintenance actions. A close fourth was new deck or overlays with 53% and 5% of respondents indicating that it is occasionally and often needed, respectively.

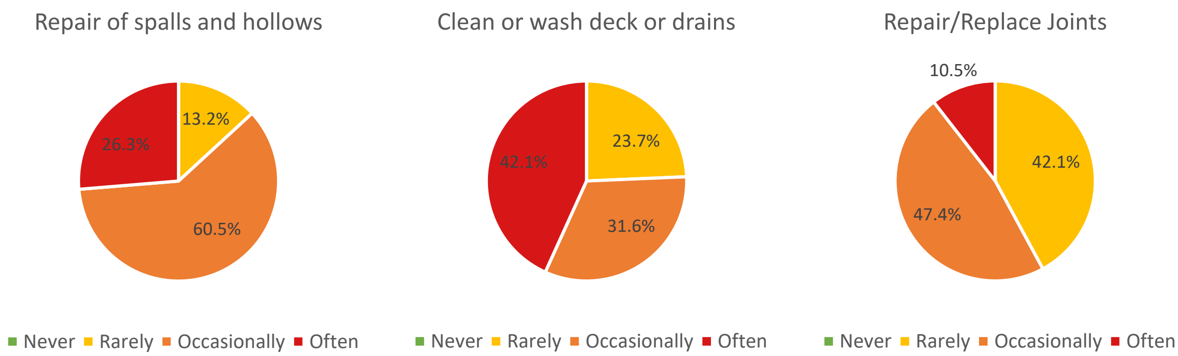

Figure 23. Most frequent deck maintenance actions

Question 6: *What are the most commonly observed durability issues affecting bridge decks? Please indicate how common each issue is.*

This question aimed to understand which deck deterioration mechanisms are most prevalent throughout the state of Iowa. Four common deterioration mechanisms were provided for local DOT jurisdictions to indicate how frequently are these encountered in their bridges. Respondents were able to add up to three deterioration mechanisms beyond those provided; however, none were added by the respondents. Figure 24 summarizes how prevalent is each deterioration mechanism.

Approximately 82% of respondents indicated that reinforcement corrosion occasionally or often affects their bridge decks; 68% indicated that freeze-thaw occasionally or often affects their bridge decks; and 55% indicated that Alkali-aggregate reaction occasionally or often affects their bridge decks. Note however, that impact damage to joints and approaches appears to be more often an issue than Alkali-aggregate reaction. Figure 25 shows the distribution of responses for the top three most prevalent deterioration mechanisms affecting bridge decks.

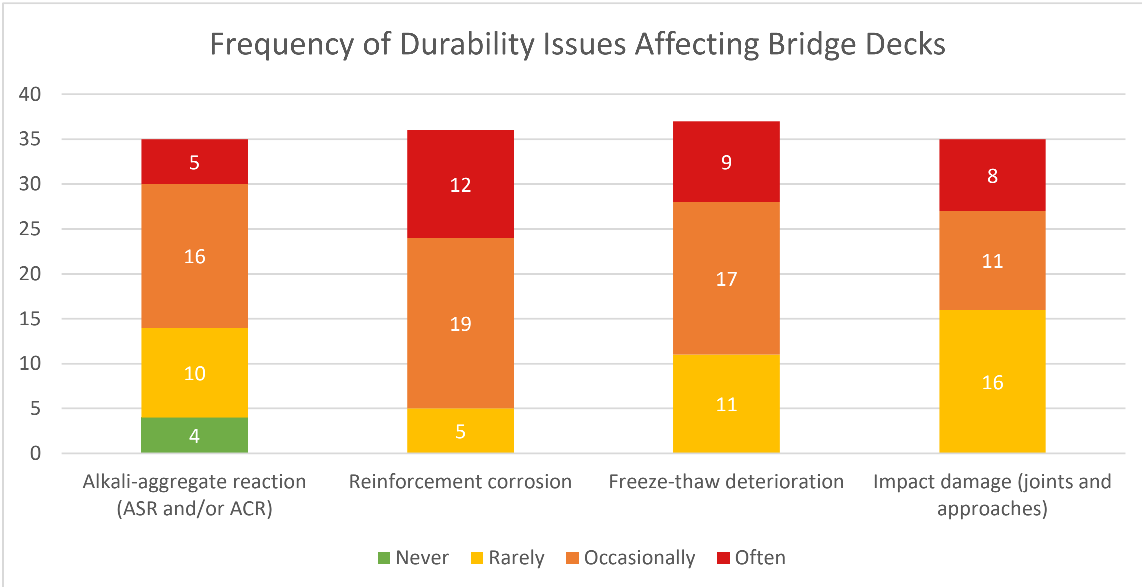

Figure 24. Frequency of deterioration mechanism affecting bridge decks

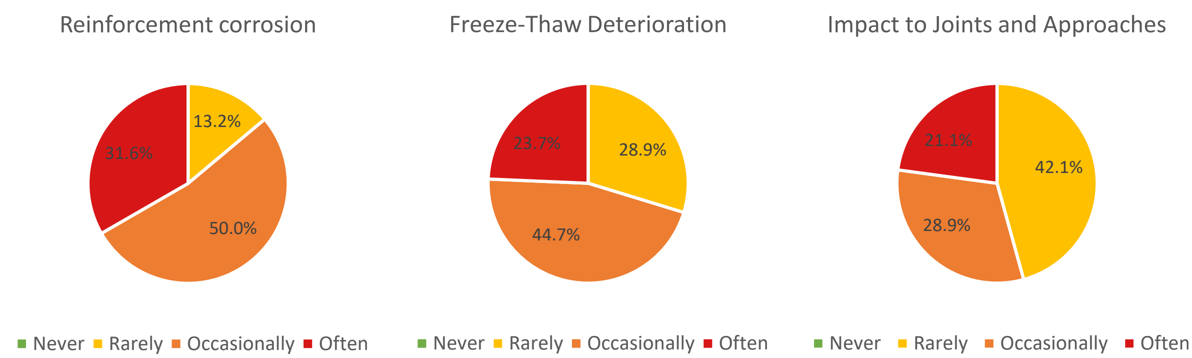

Figure 25. Most common bridge deck deterioration mechanisms

Question 7: *What are the strategies/materials most commonly used in your jurisdiction to reduce the likelihood of reinforcement corrosion in bridge decks? Please indicate the frequency of their use.*

This question aimed to understand what the current state of practice for reinforcement corrosion mitigation is in the state of Iowa. Nine strategies were provided for local DOT jurisdictions to indicate how frequently are these used in their bridge deck designs. Respondents were able to add up to three strategies beyond those provided. In addition to the provided options, the use of Ipanex® waterproofing admixture was reported by one respondent. Figure 26 summarizes the most prevalent practices for reinforcement corrosion mitigation among local Iowa DOT jurisdictions. Note that 68% to 71% of respondents indicated that stainless steel reinforcement is never used for bridge decks, likely due to the higher cost.

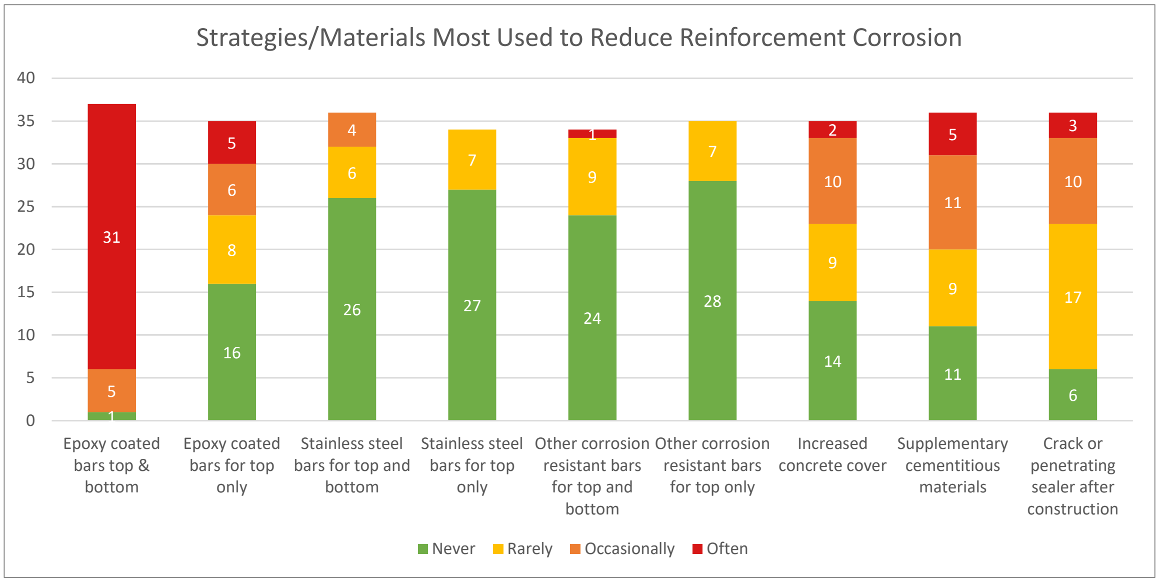

Figure 26. Strategies or materials for reducing reinforcement corrosion

Epoxy coated bars for both top and bottom reinforcement mats is by far the most prevalent strategy for reinforcement corrosion mitigation and approximately 82% of respondents indicated that epoxy coated bars are often used. Supplementary cementitious materials and crack or penetrating sealers were the second and third most often used strategies. Figure 27 shows the distribution of responses for the top three most common practices for reinforce corrosion mitigations.

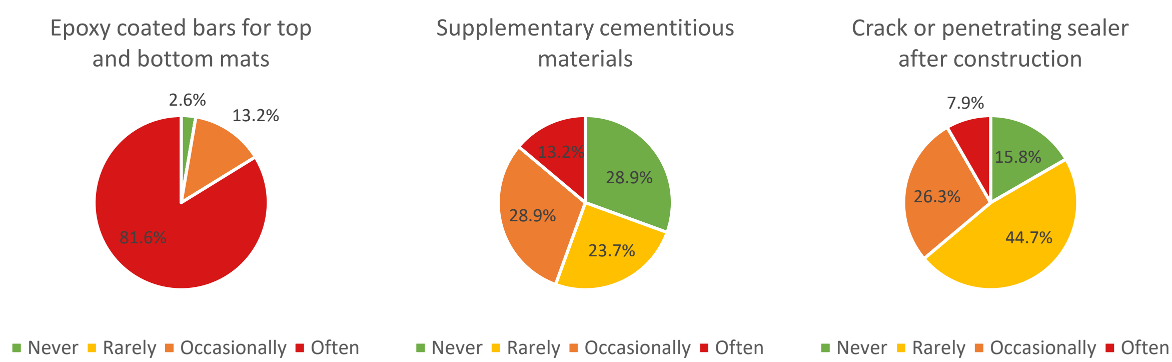

Figure 27. Most commonly used strategies or materials for reduction of reinforcement corrosion

Question 8: *Indicate how common the following types of expansion joints are among the existing bridges managed by your agency.*

This question aimed to understand what the current state of practice regarding the design of joints in the state of Iowa. Six types of joints were provided for local DOT jurisdictions to indicate how common is each type of joint among their bridge inventories. Respondents were able to add up to two other types of joints beyond those provided. However, none were added. Figure 28 summarizes the most common types of joints encountered among Iowa DOT bridges.

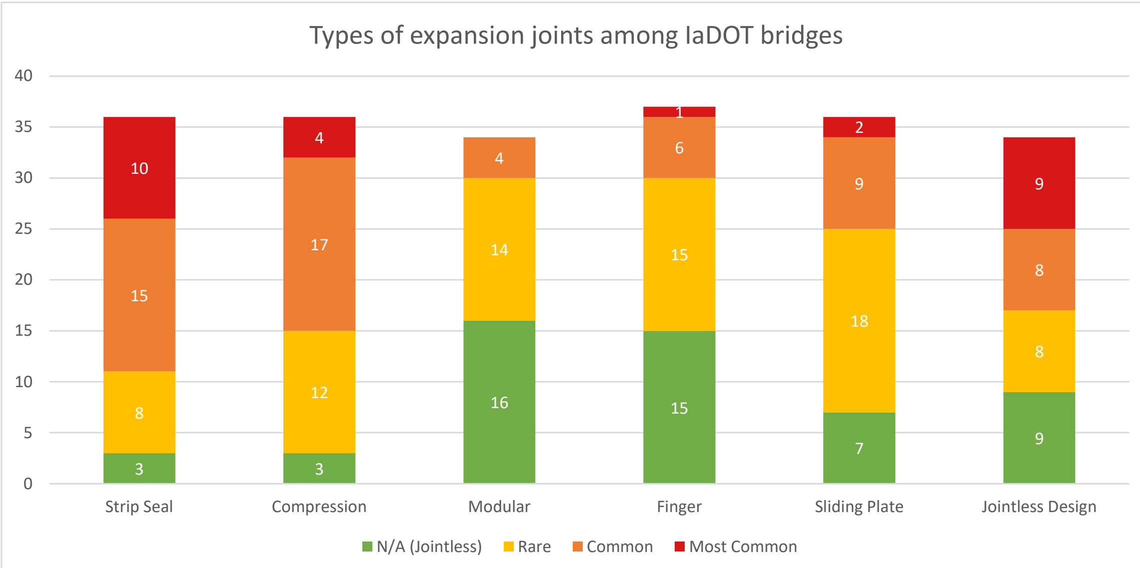

Figure 28. Types of expansion joints among Iowa DOT bridges.

Approximately 66% of respondents indicated that strip seal joints are common or the most common type of joints; 55% indicated that compression joints are common or the most common type of joints; and 45% indicated that jointless designs are common or the most common design. Note that jointless designs are evenly split with approximately 24% of respondents indicating they are the most common joint design and that it is not applicable. Furthermore, most of the responses indicating that jointless designs are most common appear to correspond to rural areas and likely smaller, single span or concrete slab bridges. Six responses from Ames and one from Cedar Falls indicate that jointless designs are not applicable or rare. Figure 29 shows the distribution of responses for the top three most common types of joints.

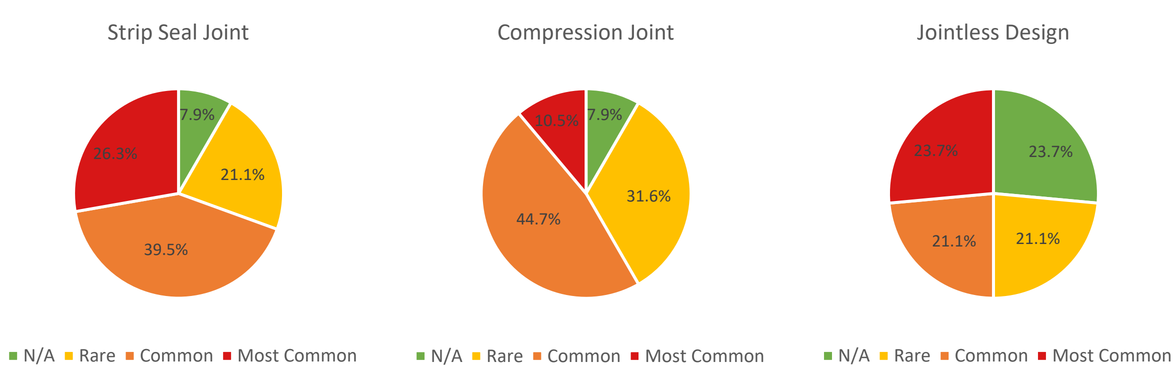

Figure 29. Most common types of joints among Iowa DOT bridges

Question 9: *List the types of expansion joints most commonly specified by your agency currently for repairs or new design or identify if a jointless design such as a link slab is specified. Please list strategies from most common to least common.*

Of the 38 respondents only 20 provided answers to this question. Based on the survey responses, strip seal joints and compression joints are tied as the most commonly specified joints by Iowa DOT with 30% of respondents indicating these are the most commonly specified joint. Additionally, 20% of respondents indicated that jointless designs are most commonly specified. Table 10 presents a summary of the types of joints specified by Iowa DOT and the count of respondents that included each joint type in their responses. Most common, common, and least common designations were assigned based on the order each respondent included the joint type in their response.

Table 10. Summary of expansion joint types most commonly specified by Iowa DOT for new designs and repairs

Joint Type	Most Common	Common	Least Common	Total
Compression Joint	6	5		**11**
Strip Seal Joint	6	4	2	**12**
Jointless Design	4		1	**5**
Neoprene Gland Joint	2			**2**
Sliding Plate Joint	2	2	1	**5**
Finger Joint		1	1	**2**
Modular Joint			2	**2**
Preformed rubber/composite expansion material		1		**1**
Total	**20**	**13**	**7**	**40**

Question 10: *What are the most commonly observed durability issues in your jurisdiction affecting bridge joints? Please indicate how common each issue is.*

Respondents were provided a list of seven common joint issues affecting bridge durability. Additionally, respondents were given the option to add up to three other issues that may be relevant to joint. However, none were added. Figure 30 summarizes common joint issues affecting bridge durability and how often local Iowa DOT jurisdictions encounter these in their bridges.

Approximately 95% of respondents indicated that debris on the joints is occasionally or often an issue; 84% indicated that deteriorated or failed compression seal are occasionally or often and issue; and 82% indicated that deteriorated or failed strip seals are occasionally or often and issue. Furthermore 80% of respondents indicated that deteriorated or missing joint fillers are occasionally or often an issue affecting bridge durability. As expected, deterioration of joint fillers and seals leading to leaky joints are the most common joint-related durability issue that Iowa DOT faces while maintaining their bridges. Figure 31 shows the distribution of responses for the top three most common joint issues affecting bridge durability.

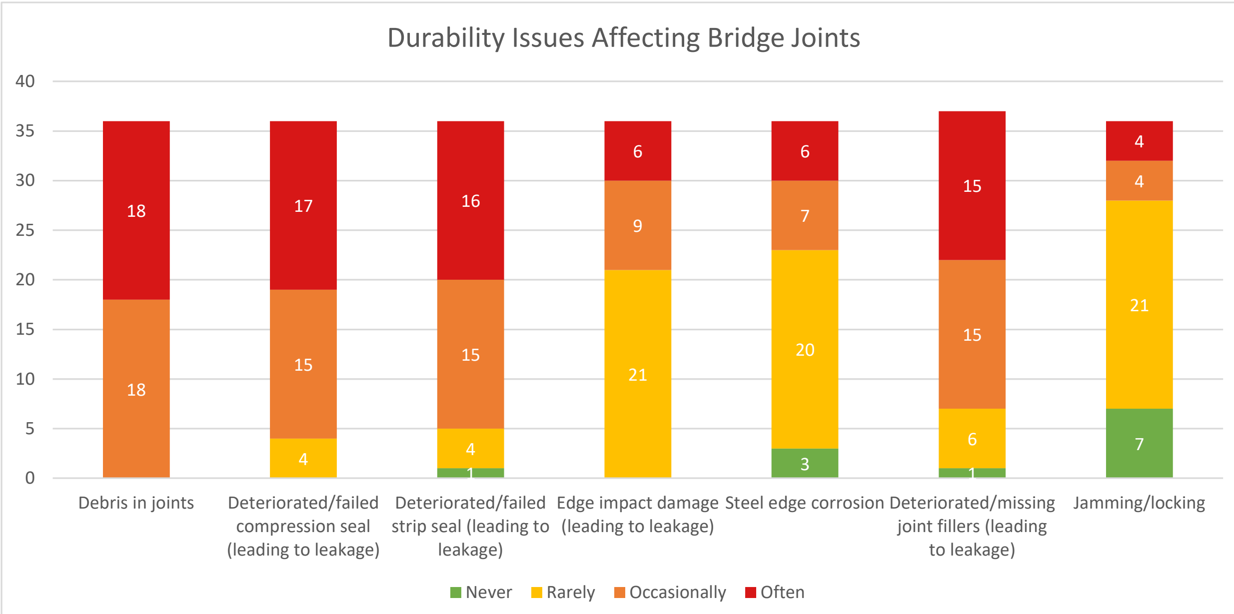

Figure 30. Common durability issues affecting bridge joints

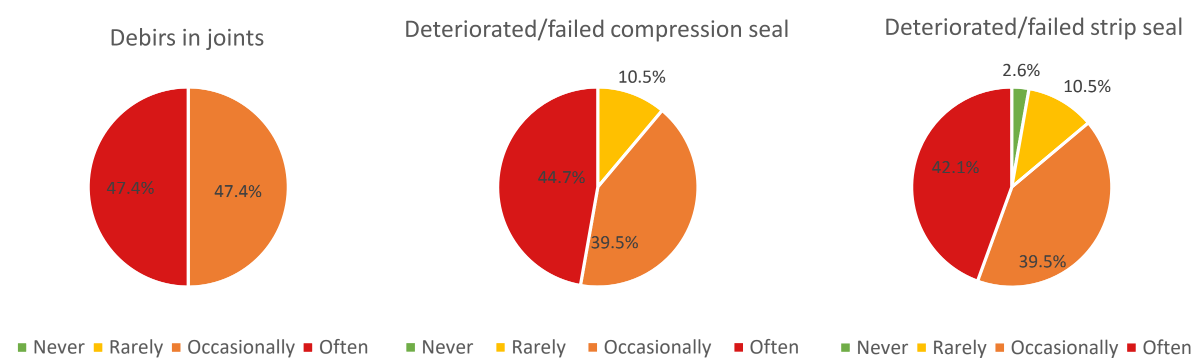

Figure 31. Most common joint issues affecting bridge durability.

Question 11: *What are the most commonly observed durability issues in your jurisdiction affecting concrete railings/parapets? Please indicate how common each issue is.*

Respondents were provided a list of seven railings issues affecting bridge durability. Additionally, respondents were given the option to add up to three other issues that may be relevant to railings or parapet deterioration. However, none were added. Figure 32 summarizes common railing issues affecting bridge durability and how often local Iowa DOT jurisdictions encounter these in their bridges.

Reinforcement corrosion on the inside face of the railings (splash affected face) is the most common railing issue with 68% of respondents identifying it as occasionally or often affecting their bridges. Freeze-thaw deterioration and corrosion of anchors between railing and bridge deck were also common with 55%

and 50% of respondents, respectively, indicating these are occasionally or often an issue. Figure 33 shows the distribution of responses for the top three most common railing issues.

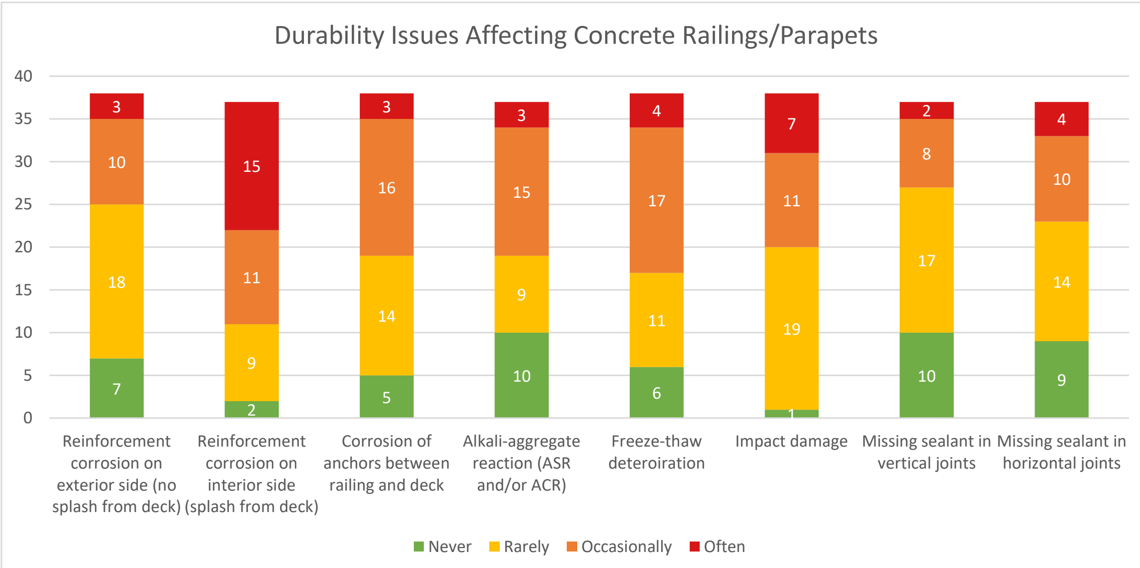

Figure 32. Durability issues affecting concrete railings and parapets

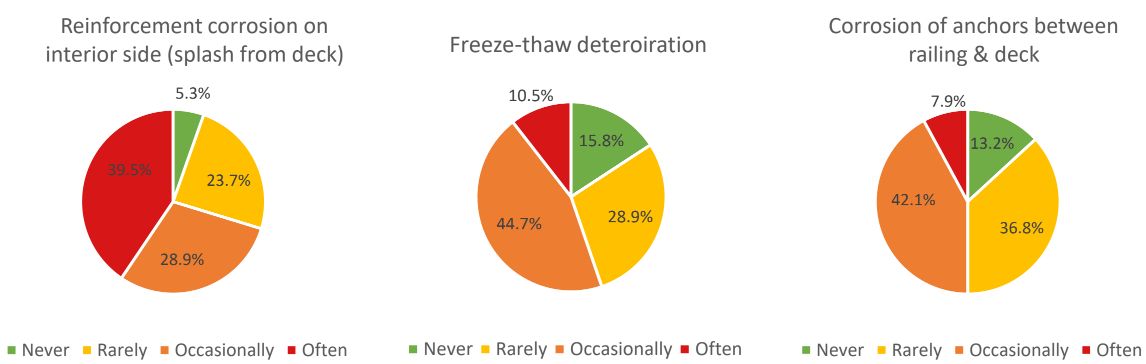

Figure 33. Most common durability issues affecting bridge railings and parapets

Question 12: *What are the most commonly observed durability issues in your jurisdiction affecting steel railings? Please indicate how common each issue is.*

Respondents were provided a list of three steel railings issues affecting bridge durability. Additionally, respondents were given the option to add up to three other issues that may be relevant to steel railing deterioration. However, none were added. Figure 34 summarizes the railing issues affecting bridge durability and how often local Iowa DOT jurisdictions encounter these in their bridges.

As shown in Figure 34, impact damage appears to be the most common issues affecting steel railings followed by railing corrosion. Connection deterioration, however, does not appear to be a common issue since 50% of respondents indicated it is rarely or never an issue.

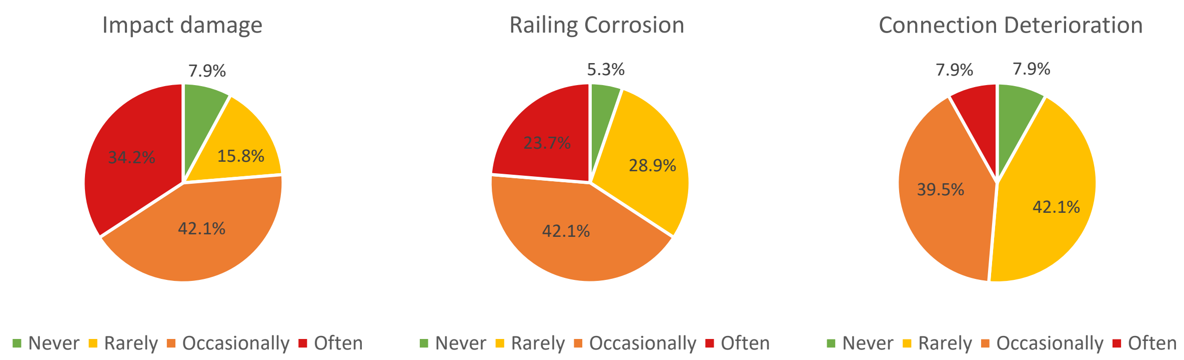

Figure 34. Most common steel railing durability issues

4.2.3 Superstructures, Substructures, and Bearings

Question 13: *Identify the most frequent superstructure maintenance actions. Please indicate how frequently each action is needed.*

This question was aimed to understand which superstructure maintenance activities are most frequent throughout the state of Iowa. For this purpose, six maintenance actions were provided for local DOT jurisdictions to indicate how often are each of the maintenance actions needed; respondents were able to add up to three maintenance actions beyond those provided. However, none were added. Figure 35 summarizes how often are each superstructure maintenance action is needed.

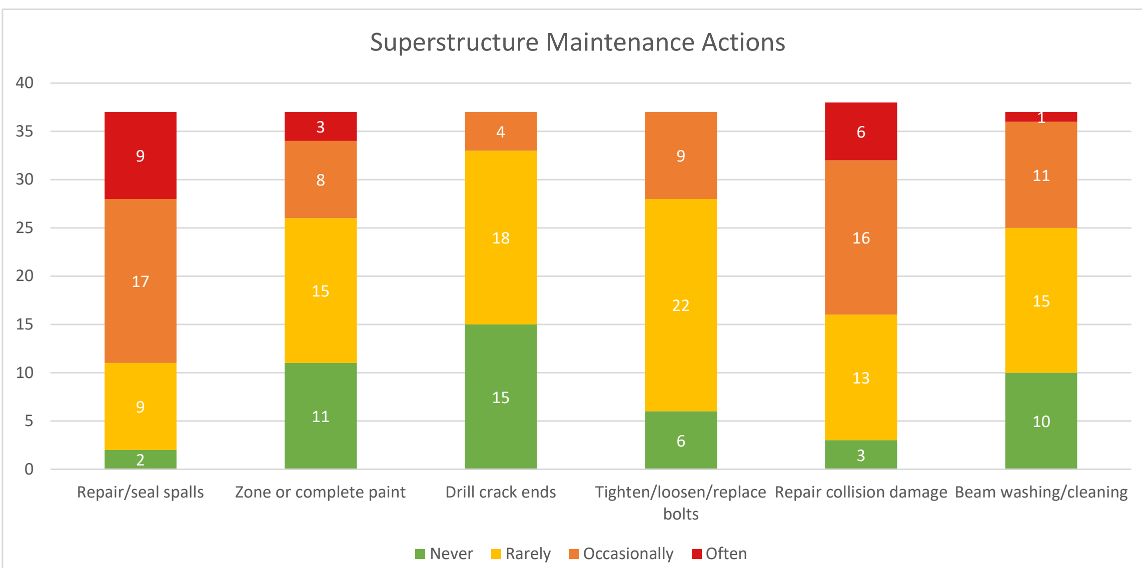

Figure 35. Frequency of superstructure maintenance actions

Repair of spalls was reported as the most common superstructure maintenance action, since 68% of respondents indicated that this maintenance action is needed often or occasionally. The second most common maintenance activity is collision damage repairs which was reported by 58% of respondents as needed often or occasionally. The third most common superstructure maintenance activity is zone or complete painting which 29% of respondents indicated is occasionally (21%) or often needed (8%). Figure 36 shows the distribution of responses for the three most needed superstructure maintenance actions.

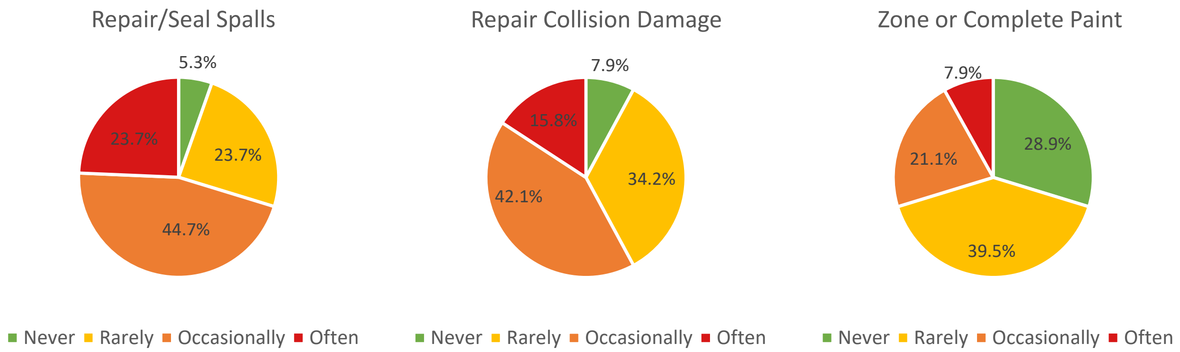

Figure 36. Most common superstructure maintenance actions.

Question 14: *What are the durability issues affecting the bridge superstructure that are most commonly observed by your agency? Please indicate how common each issue is.*

The purpose of this question was to understand which superstructure distress or deterioration mechanisms most often affects the durability of Iowa DOT bridges, as opposed to which maintenance is most common (Question 13). For this purpose, six deterioration mechanisms were provided for local DOT jurisdictions to indicate how often they affect their bridges; respondents were able to add up to three maintenance actions beyond those provided. However, none were added. Figure 37 summarizes how prevalent is each deterioration mechanisms.

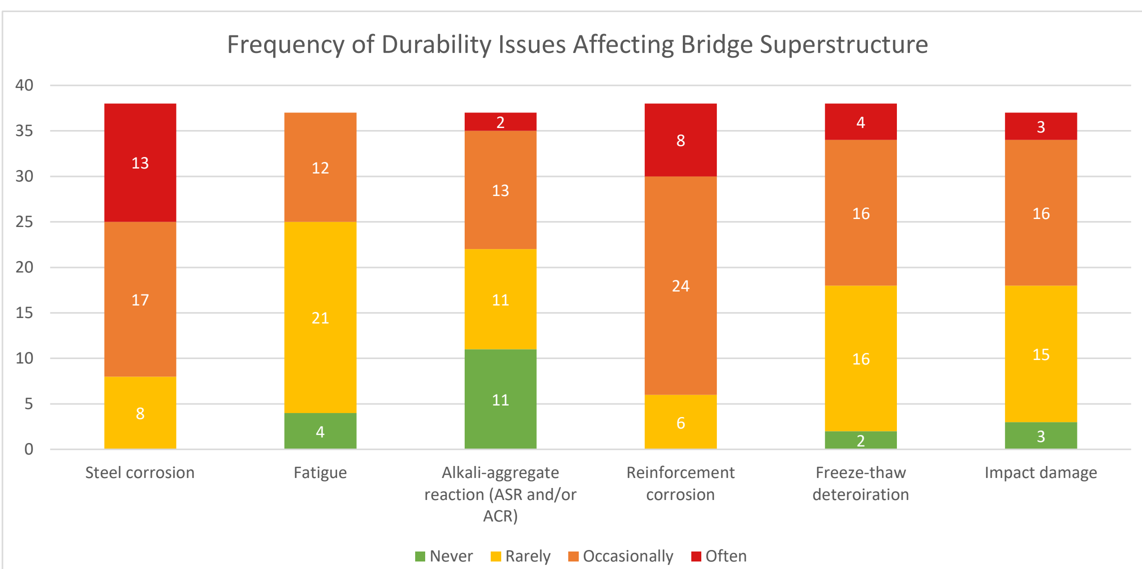

Figure 37. Frequency of durability issues affecting bridge superstructures

Approximately 79% of respondents indicated that corrosion of steel members occasionally or often affects their bridge superstructures. Furthermore, corrosion of steel members was reported as the most prevalent distress affecting bridge superstructures with 34% of respondents indicating it is often an issue. The two other most prevalent distresses affecting superstructures were reinforcement corrosion (21% of respondents indicated it is often an issue) and freeze thaw deterioration (10.5% of respondents indicated it is often an issue). Figure 38 shows the distribution of responses for the top three most prevalent distresses affecting superstructures.

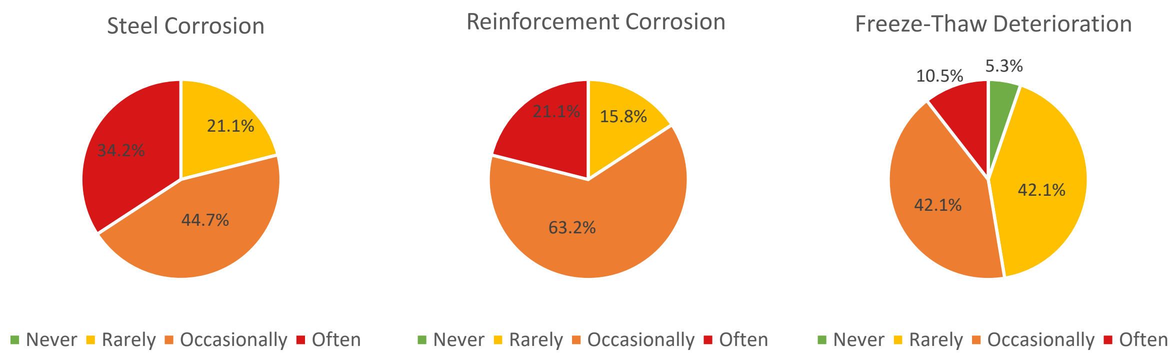

Figure 38. Most common durability issues affecting bridge superstructures

Question 15: *Identify the most frequent substructure maintenance actions. Please indicate how frequently each is needed.*

This question was aimed to understand which substructure maintenance activities are most frequent throughout the state of Iowa. For this purpose, five maintenance actions were provided for local DOT jurisdictions to indicate how often are each of the maintenance actions needed; respondents were able to add up to three maintenance actions beyond those provided. A respondent indicated that erosion around backwalls occasionally requires maintenance. Figure 39 summarizes how frequent is each maintenance action needed.

The most frequent maintenance action was reported to be repairs to abutment backwalls which 63% of respondents indicated it is often or occasionally needed. In addition to the repair of backwalls, repair of abutment face and seats was reported to be occasionally or often needed by 50% of respondents. Approximately 45% of respondents indicated that column repairs are occasionally or often needed. However, column repairs had a larger percentage of respondents that indicated it is often needed. Therefore, column repairs are likely as frequently needed as abutment face and seat repairs. Figure 40 shows the distribution of responses for the top three most common substructure maintenance actions.

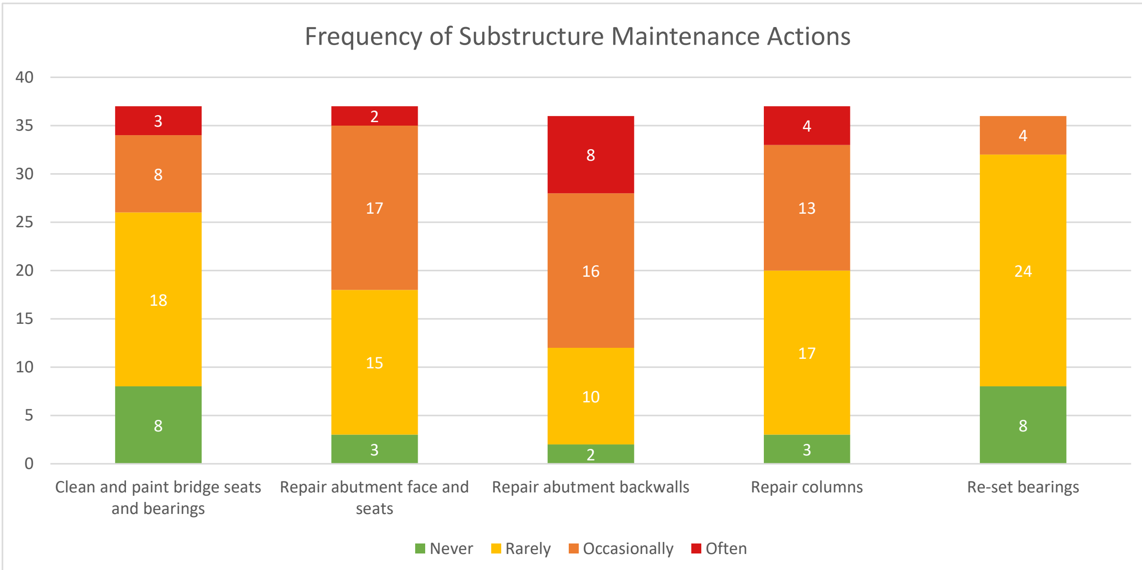

Figure 39. Frequency of substructure maintenance actions

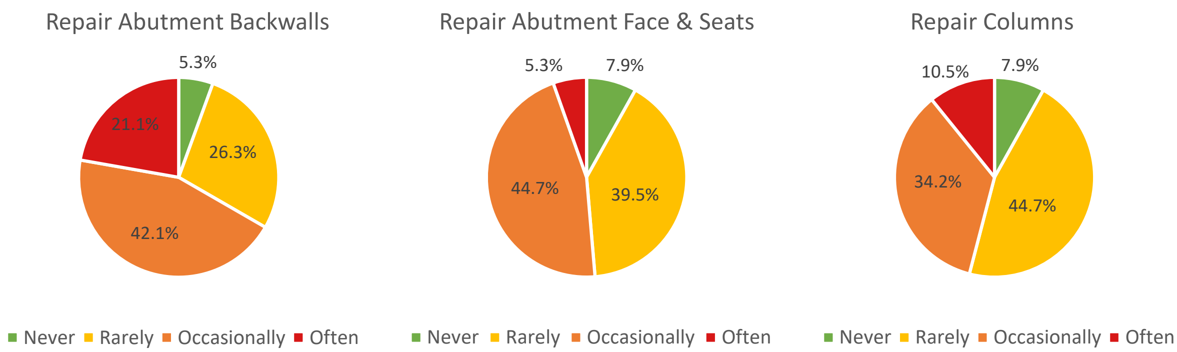

Figure 40. Most common substructure maintenance actions

***Question 16:** Indicate how common the following types of bearings are among the bridges managed by your jurisdiction.*

The purpose of this question was to understand what the most prevalent types of bearings among Iowa DOT bridges are. Six types of bearings were provided for local DOT jurisdictions to indicate how prevalent each type of bearing is among their bridges; respondents were able to add up to two bearing options beyond those provided. However, none were added. Figure 41 summarizes how prevalent is type of bearing is among Iowa DOT bridges.

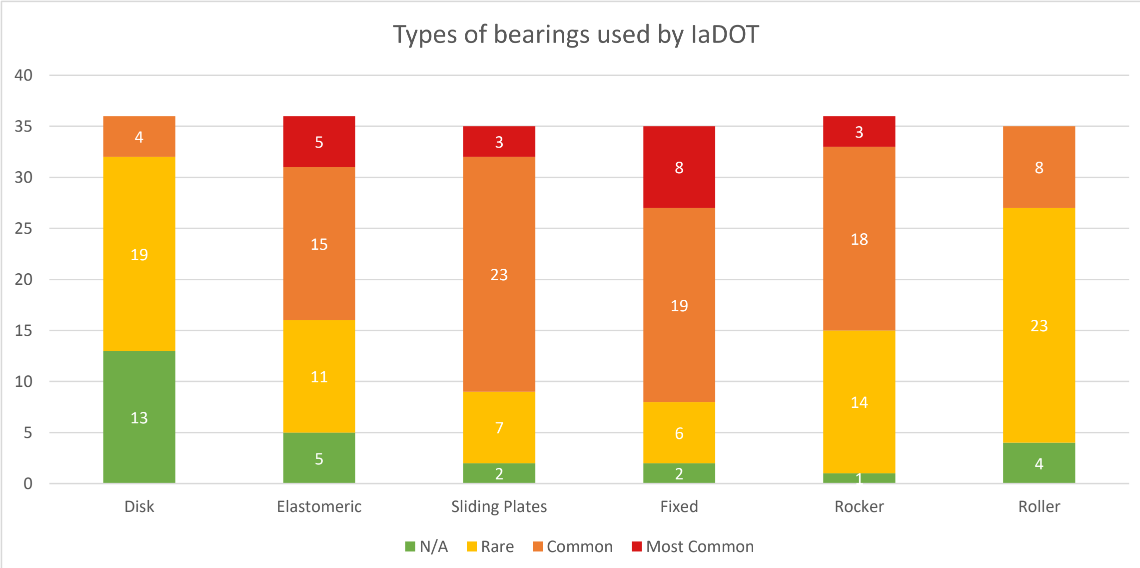

Figure 41. Types of bridge bearings used by Iowa DOT

Fixed bearings were reported as the most common type of bearing by 21% of respondents and common or most common by 71% of respondents. Sliding plate bearings were the second most common type of bearings and were reported as common or most common type of bearing by 68% of respondents. Elastomeric and rocker bearing were reported as common or most common type of bearing by 53% and 55% of respondents, respectively. Note however, that elastomeric bearings had a higher percentage of respondents indicating it is the most common type of bearing than rocker bearings; i.e, 13% compared to 8% of respondents. Figure 42 shows the distribution of responses for the three most common types of bearings among Iowa DOT bridges.

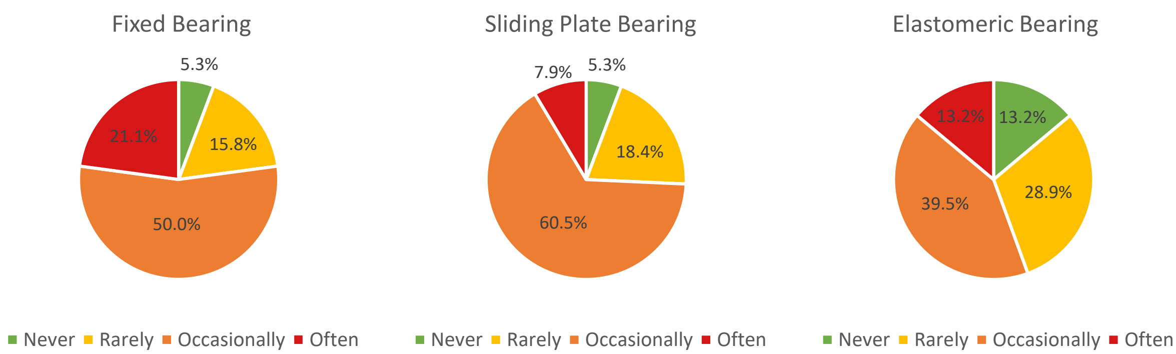

Figure 42. Most prevalent types of bearings among Iowa DOT bridges.

Question 17: *List the types of bearings most commonly specified by your agency for repairs or new design. Please list them from most common to least common.*

Of the 38 respondents only 19 provided answers to this question. Based on the survey responses, fixed and elastomeric bearings are the two most specified bearings by Iowa DOT with approximately 32% and

26% of respondents indicating these are the most commonly specified bearings, respectively. Table 11 presents a summary of the types of bearings specified by Iowa DOT and the count of respondents that included each bearing type in their responses. Most common, common, and least common designations were assigned based on the order each respondent included the bearing type in their response.

Table 11. Summary of bearing types most commonly specified by Iowa DOT for new designs and repairs

Bearing Type	Most Common	Common	Least Common	Total
Fixed	6	3	3	**12**
Elastomeric	5	4		**9**
Sliding Plate	4	2	1	**7**
Integral	2			**2**
Rocker	2	5	1	**8**
Pin			1	**1**
Roller		1	1	**2**
Grand Total	**19**	**15**	**7**	**41**

Question 18: *What are the most commonly observed durability issues in your jurisdiction affecting bridge bearings? Please indicate how common each issue is.*

Respondents were provided a list of four issues affecting bridge bearing durability. Additionally, respondents were given the option to add up to three other issues that may be relevant to bearings deterioration. However, none were added. Figure 43 summarizes bearing issues affecting bridge durability and how often local Iowa DOT jurisdictions encounter these in their bridges.

As shown in Figure 43, steel corrosion appears to be the most common issue affecting bridge bearings (84% of respondents indicated it is occasionally or often an issue) closely followed by deterioration due to leaky joints (76% of respondents indicated they are occasionally or often an issue). The third most common issue with bearings is freezing or locking of bearings, which 66% of respondents indicated is occasionally or often an issue. Clearly, leaky joints allow water and chlorides to reach the bearings more easily, exacerbating corrosion, and increasing the potential for freezing of bearings. Figure 44 shows the distribution of responses for the three most common issues with bridge bearings.

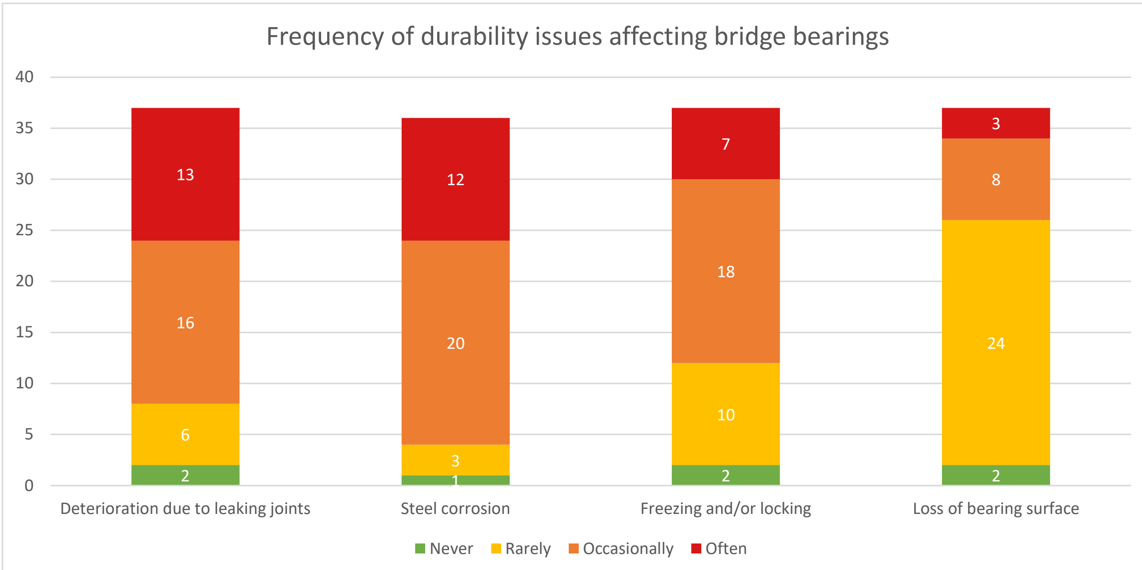

Figure 43. Frequency of common durability issues affecting Iowa DOT bridge bearings.

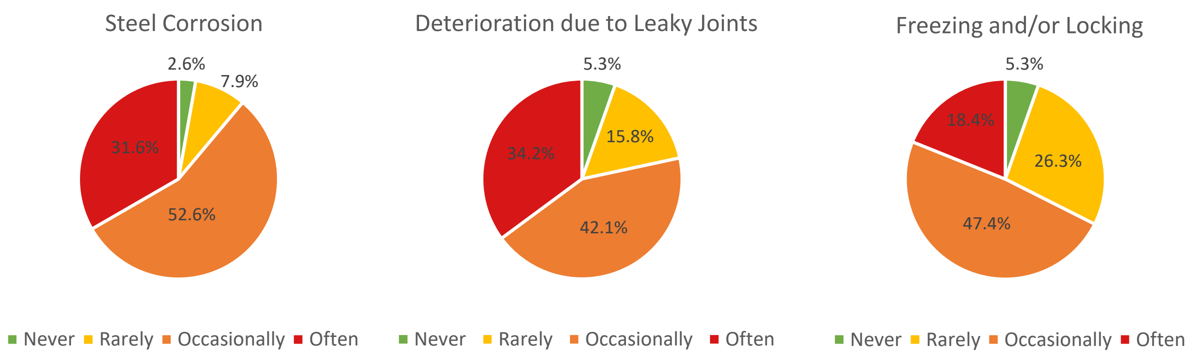

Figure 44. Most common deterioration mechanism affecting the bearings of Iowa DOT bridges

Question 19: *What are the most commonly observed durability issues affecting the bridge piers, walls, and abutments? Please indicate how common each issue is.*

Respondents were provided a list of four issues affecting the durability of bridge piers, walls, and abutments. Additionally, respondents were given the option to add up to three other issues that may be relevant to bearings deterioration. A respondent indicated that timber decay is often an issue. Figure 45 summarizes common issues affecting bridge durability and how often local Iowa DOT jurisdictions encounter these in their bridges.

Reinforcement corrosion and freeze-thaw deterioration were reported to be the most prevalent distresses affecting bridge piers, walls, and abutments with approximately 68% and 61% of respondents, respectively, indicating these are often or occasionally an issue. Figure 46 shows the distribution of responses for the top three distresses affecting the piers, walls, and abutments of Iowa DOT bridges.

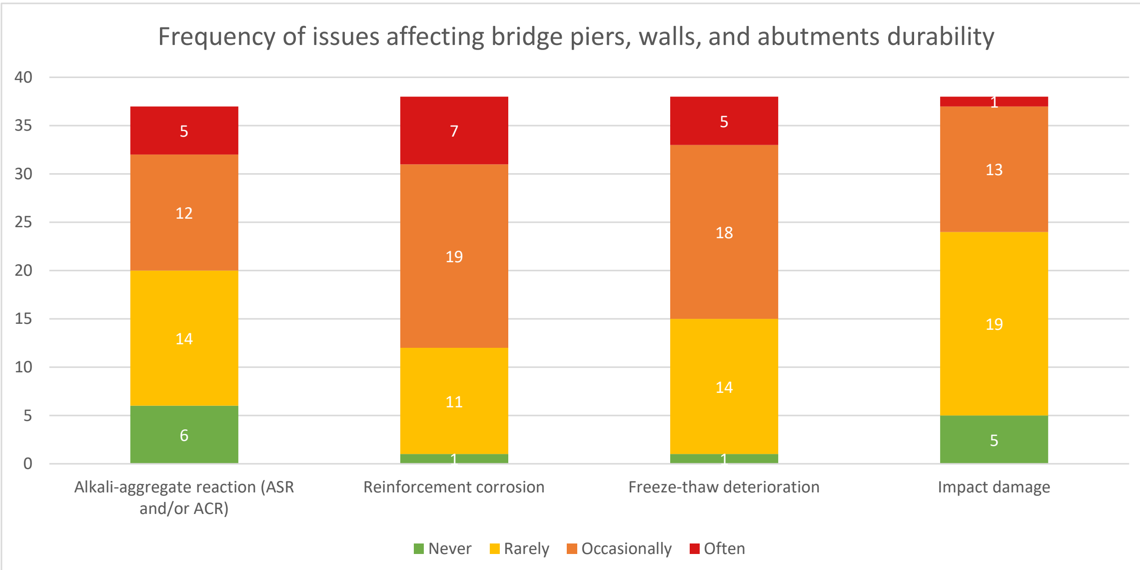

Figure 45. Frequency of common durability issues affecting Iowa DOT bridge piers, walls, and abutments

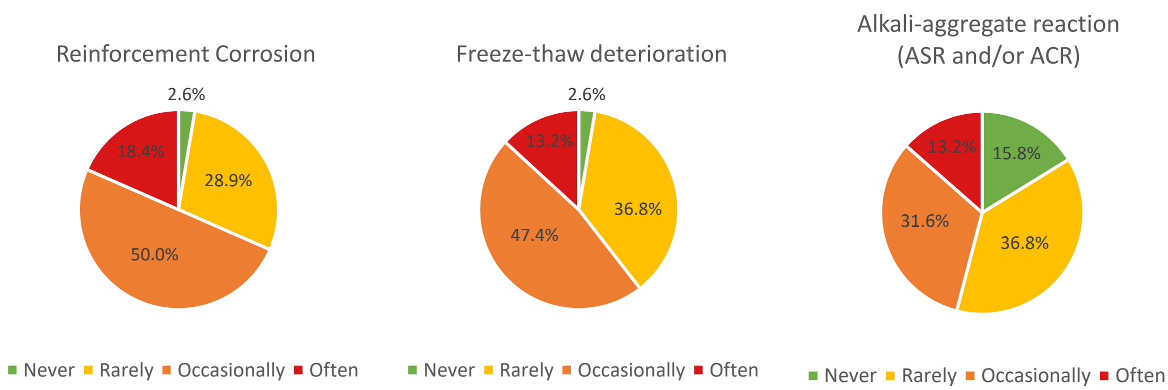

Figure 46. Most common distress affecting Iowa DOT bridge piers, walls, and abutments

4.2.4 Bridge Foundations, Pavements, and Approaches

Question 20: *What are the most commonly observed durability issues affecting the bridge foundations? Please indicate how common each issue is.*

Respondents were provided a list of seven issues affecting the durability of bridge foundations. Additionally, respondents were given the option to add up to three other issues that may be relevant to foundation deterioration. However, none were added. Figure 47 summarizes common foundation issues affecting bridge durability and how often local Iowa DOT jurisdictions encounter these in their bridges.

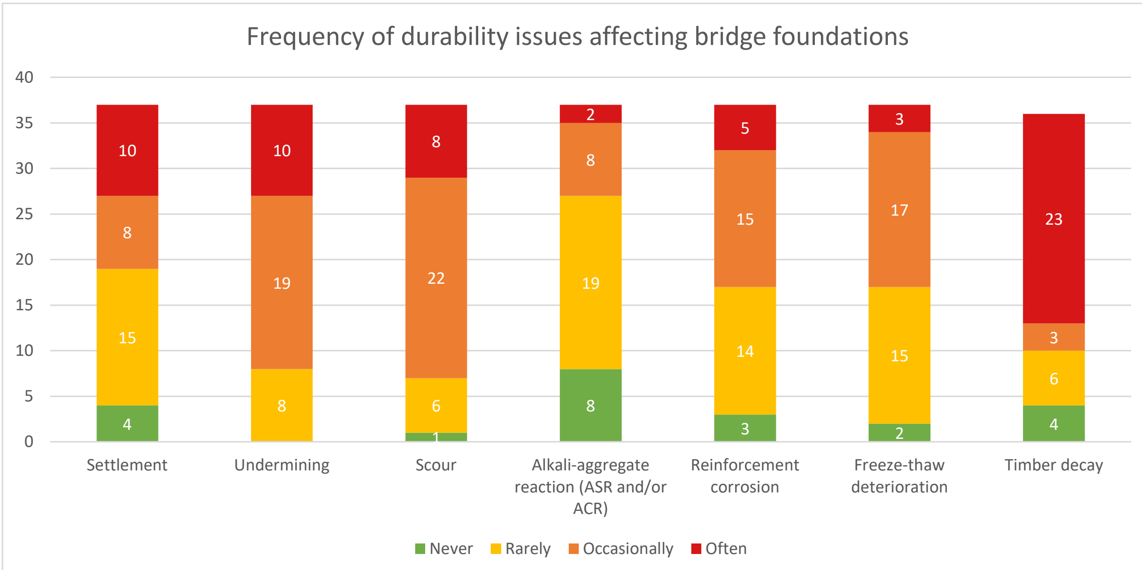

Figure 47. Frequency of common durability issues affecting Iowa DOT bridge foundations

Timber decay appears to be, by far, the most common problem affecting bridge foundations; approximately 61% of respondents indicated timber decay is often an issue. Based on the survey results, Iowa DOT appears to have many older bridges with wood piles which are often in need of repair or replacement due to pile deterioration. Other common issues were foundation undermining and scour which approximately 26% and 21% of respondents, respectively, indicated are often an issue. Figure 48 shows the distribution of responses for the top three most common foundation issues among Iowa DOT bridges.

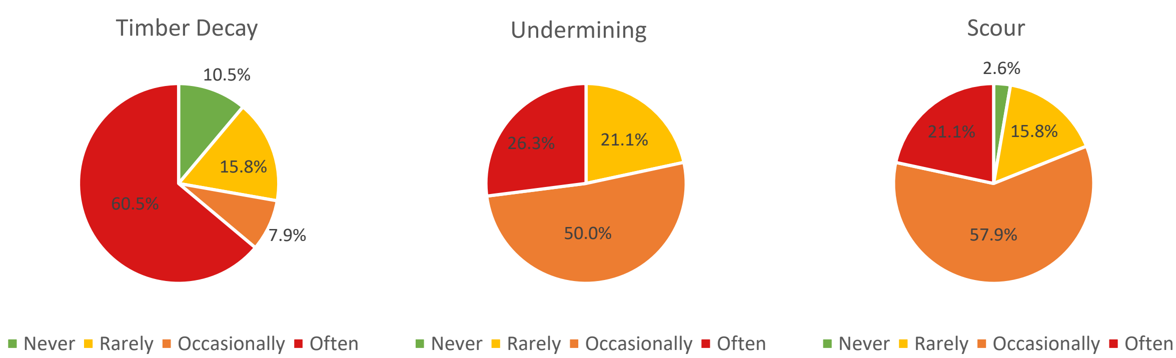

Figure 48. Most common foundations issues affecting the durability of Iowa DOT bridges

Question 21: *Identify the most frequent pavement maintenance actions. Please indicate how frequently each is needed.*

This question included a list of seven issues affecting the durability of bridge pavements for Iowa DOT personnel to consider. Respondents were also given the option to add up to three other issues that may be relevant to pavement deterioration. Approach slab settlement was identified as often being an issue by a respondent. Figure 49 summarizes pavement issues affecting bridge durability and how often local Iowa DOT jurisdictions encounter these in their bridges.

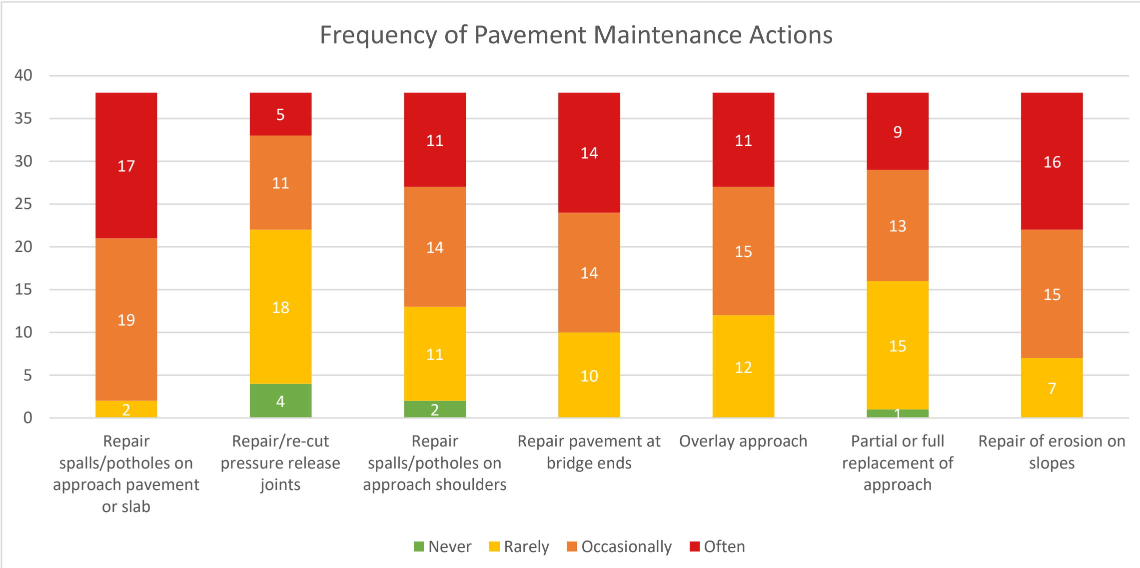

Figure 49. Frequency of Pavement Maintenance Actions

The most frequent pavement maintenance action is the repair of spalls and potholes on the approach pavement or slab, which was identified by 95% of respondents as occasionally or often needed. The second most frequent maintenance action is slope erosion repairs which 82% of respondents indicated is occasionally or often needed. Pavement repairs at the bridge ends was the third most frequently needed repair according to 74% of respondents. Figure 50 shows the distribution of answers for the three most frequently needed pavement maintenance actions.

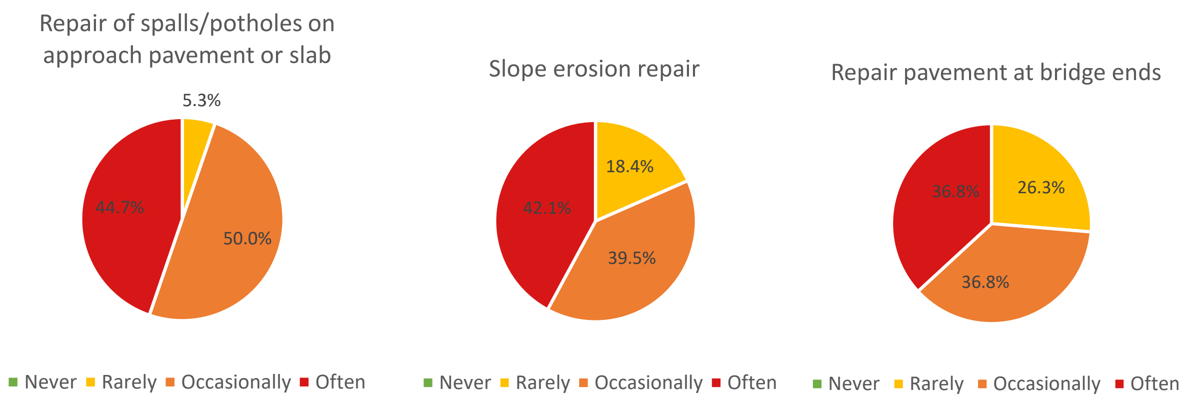

Figure 50. Most frequent pavement maintenance actions

Question 22: *What are the three primary durability issues observed by your agency regarding approaches that cause them to require repair? Please list them from most common to least common.*

Of the 38 respondents only 27 provided answers to this question. According to 81% of respondents, settlement is the most common issue affecting approaches. Table 12 presents a summary of common approach issues affecting Iowa DOT bridges. Most common, common, and least common designations were assigned based on the order of each respondent answer.

Table 12. Summary of durability issues affecting Iowa DOT bridge approaches

Bearing Type	Most Common	Common	Least Common	Total
Settlement	22	1	2	25
Cracking	3	2		5
Erosion	1	1	2	4
Pavement failure - potholes/spalls	1	2	3	6
Approach curb failure			1	1
Approach failure		1		1
Dirt buildup at guard rail		1		1
Failed joints		1		1
Freeze-thaw			1	1
Joint deterioration/corner cracking		1		1
Joint failure		1		1
Pavement deterioration		1		1
Paving notch failure		1		1
Rocking		1		1
Undermining		3	2	5
Washing			1	1
Total	**27**	**17**	**12**	**56**

CHAPTER 5. APPROACH FOR DEVELOPMENT OF DURABLE BRIDGE DESIGNS

Based on the current maintenance practices and needs of Iowa DOT, the following bridge components were identified as requiring the most maintenance:

1. Reinforced concrete members
    a. Concrete decks
    b. Concrete barriers
    c. Concrete girders
    d. Concrete pier caps
    e. Concrete pier columns
2. Steel superstructures
3. Joints
4. Bearings
5. Foundations
6. Approach systems
7. Berm erosion

The following chapter presents analysis and recommendations to design the elements listed above for different target service lives, with the underlying assumption that a longer service life will inherently reduce the maintenance needs for a longer period during the life of a bridge component. The AASHTO *Guide Specification for Service Life Design of Highway Bridges* (AASHTO, 2020) identifies certain components that are in this list as renewable including: joints, bearings, railings and barriers, and approach slabs. Renewable elements are those that are designed to be replaced within the service life of the bridge. While some of these items are addressed in the next section, the expectation that some of these elements will need to maintained or replaced during the life of a bridge.

In this report, recommendations are provided for improving the durable design for the various bridge components identified above. Recommendations are provided based on a review of the literature and understanding of the best current practices. For reinforced concrete members and steel superstructures, a targeted service life approach is also used to model and predict the service life of the elements.

Note that all the recommendations herein assume a strict adherence to the quality of materials and construction.

5.1 Target Service Life Design Approach for Alternative Element Designs

The original goal of this work has been to identify the durable designs that will minimize the need for maintenance of bridges in Iowa in the first 50 years of service. Inherently, this requirement will lead to longer service lives than are currently exhibited by the standard Iowa DOT design specifications. During the course of the project, the project team and Iowa DOT decided that while designs for minimum maintenance requirements are needed, an approach that uses target service lives for different cases maybe more desirable. This approach assumes that using a longer service life target will reduce the required maintenance for a longer period of time. This approach also follows more closely the recent trend for durability design, which was introduced by the *Guide Specification for Service Life Design of Highway Bridges, HBSLD-1* (AASHTO, 2020) and is currently being implemented by a number of states, at least for signature and critical bridges.

The alternative element design cases were classified according to a "good-better-best" model, similar to those used in the HBSLD-1 (AASHTO, 2020) and the MnDOT Service Life Design Guide for Bridges (MnDOT, 2022) publications. These two publications define the three tiers of service life as follows:

- "Normal", corresponding to 75 years of service life, consistent with the current AASHTO *LRFD Bridge Design Specifications*;
- "Enhanced", corresponding to 100 years of service life; and
- "Maximum", corresponding to 150 years of service life.

While the AASHTO publication includes some generalized service life modeling to support these classifications, the MnDOT publication is more empirical and is largely based on industry consensus. In addition, both publications assume that designs conforming to current AASHTO LRFD standards will yield a 75-year service life; however, this service life expectation has not been verified for typical Iowa exposures.

For the Iowa DOT bridge designs cases considered in this study, WJE considered modified service life categories as follows, with service lives confirmed via service-life modeling with WJE CASLE™:

- "**Baseline**", corresponding to the approximate service life obtained with current element designs;
- "**Normal**", corresponding to the **75-year** service life design target (note that for example for reinforced concrete, some elements may require more corrosion resistant reinforcing steel, alternative minimum cover depths, or high-performance concrete mixture designs than the minimums covered by the AASHTO LRFD specification in order to achieve 75 years of service under certain Iowa exposure conditions);
- "**Enhanced**", corresponding to **100 years** of service life; and
- "**Maximum**", corresponding to **125 years** of service life.

The intent of using these design categories in the analysis is not necessarily to state that elements need to be designed for such service lives, but rather to compare the maintenance and repair needs for bridge elements designed for specific service life targets. While the overall objective of this study is to minimize maintenance activities in the first 50 years of service, an element designed for "maximum" service life may require less maintenance in the first 50 years of service than an element designed for "normal" service life.

This study does not include life cycle cost analysis and as such, it is possible that there exist scenarios where designing for the 'normal' service life option of 75 years and regular maintenance of the bridge components thereafter could lead to a more cost effective solution than designing for an 'enhanced' or 'maximum' service life.

CHAPTER 6. ELEMENT-SPECIFIC SERVICE LIFE DESIGN CONSIDERATIONS

6.1 Reinforced Concrete Members

Maintenance and repair activities for reinforced concrete elements frequently result from deterioration of the element under environmental exposures. Deterioration mechanisms that may lead to maintenance or repair activities include chloride- and carbonation-induced corrosion of reinforcing steel, freeze-thaw deterioration of concrete, and materials-related deterioration such as alkali-silica reaction (ASR) or sulfate attack. Of these, corrosion of reinforcing steel is by far the most common durability-related issue prompting maintenance activities on Iowa DOT bridge assets.

This section discusses various design concepts that may be considered to reduce or eliminate the maintenance on reinforced concrete elements that comprise Iowa DOT bridges over their first 50 years of service. Design concepts are categorized based on expected service life targets of 75 ("Normal"), 100 ("Enhanced"), or 125 years ("Maximum"). While the primary focus is on chloride-induced corrosion of the reinforcing steel, other relevant deterioration mechanisms are also addressed.

The guidance presented in this section references the following Iowa DOT bridge design standards and specifications:

- Barrier Rail Standard Drawings, dated May 8, 2024
- Beam Standard Drawings, dated October 2, 2024
- BT Beams with Integral Abutment Drawings, dated July 30, 2024
- *Iowa DOT Design Manual*
    - Section 1C-1, *Selecting Design Criteria*, revised November 17, 2021
- *Iowa DOT LRFD Bridge Design Manual*, dated July 11, 2024
- *Iowa DOT Standard Specifications for Highway and Bridge Construction*, Series 2023
- Materials IM 491.12, *Concrete Sealers*, rev. 3, dated October 17, 2023
- Materials IM 529, *Portland Cement (PC) Concrete Proportions*, rev. 6, dated October 15, 2024
- Materials IM 570, *Precast & Prestressed Concrete Bridge Units*, rev. 4, dated October 14, 2024

6.1.1 Elements Considered

The reinforced concrete bridge elements considered in this section include the cast-in-place and precast bridge deck elements; cast-in-place and precast concrete barriers; precast prestressed concrete girders; and cast-in-place pier caps and pier columns. Minimum cover depths and reinforcing requirements for these elements per the current Iowa DOT design standards are summarized in Table 13. For bridge decks, Iowa DOT County bridge standards were also considered. Additional relevant design details and materials are described in the following sections for each element type.

Table 13. Standard minimum cover and reinforcing steel requirements for reinforced concrete bridge elements

Element Type	Typical Minimum Cover Depth	Reinforcing Requirement
Bridge Deck	Top transverse bars – 2-3/4 inches (2-1/2 inches for county bridges) Bottom transverse bars – 1-1/2 inches (1 inch for county bridges) All other surfaces – 2 inches	Epoxy-coated steel, unless otherwise noted
Barrier	Roadway-facing surface – 2 inches All other surfaces – 2-1/2 inches	At connection to deck for interstate and primary bridges – Stainless steel All other locations – Epoxy-coated steel, unless otherwise noted
Precast Prestressed Girder	Stirrups – 1 inch Prestressing strand – 1-1/2 inches	Stirrups – Uncoated carbon steel or epoxy-coated steel Prestressing strand – Uncoated carbon steel
Pier Cap	2 inches (to ties)	At expansion joint – Epoxy-coated steel All other locations – Uncoated carbon steel
Pier Column	At expansion joint – 2 inches (to ties) Within 25 feet of the edge of the traveled way – 2 inches (to ties) All other locations – 1-1/2 inch or 1-7/8 inches (to ties)	At expansion joint – Epoxy-coated steel Within 25 feet of the edge of the traveled way – Epoxy-coated steel All other locations – Uncoated carbon steel

6.1.1.1.1 Bridge Decks

Reinforced concrete bridge decks conforming to current Iowa DOT standards (Figure 51) are 8-1/2 inches thick and have 2-3/4 inches of clear cover to the top mat of reinforcing bars and 1-1/2 inches of cover to the bottom mat of reinforcing bars. Clear cover to all other exposed surfaces is a minimum of 2 inches. Bridge decks on county roads are currently designed with a clear cover of 2-1/2 inches to the top mat of reinforcing steel and 1 inch to the bottom mat of reinforcing steel, with 2 inches of minimum clear cover to all other exposed surfaces. Epoxy-coated reinforcing steel conforming to ASTM A775 is specified for all bridge deck reinforcement except at the barrier rails, where stainless steel is specified for interstate and primary bridges.

Section 2412.02 of the *Standard Specifications for Highway and Bridge Construction* specifies the use of air-entrained concrete conforming to C-4WR and C-V47B designations per Materials IM 529 for all reinforced concrete bridge decks. IM 529 specifies a maximum water-cementitious materials ratio (w/cm) of 0.45 and 0.488 for mixtures C-4WR and C-V47B, respectively. Supplementary cementitious materials (SCMs) may be used at rates up to 35 percent slag cement and/or 20 percent fly ash, by weight of total cementitious material. The total SCM content may not exceed 50 percent of the weight of total cementitious material.

The *Iowa DOT LRFD Bridge Design Manual* permits the use of prestressed deck panels (stay-in-place forms) for bridges meeting certain criteria. If precast decks are used, Section 2407 of the *Standard Specifications for Highway and Bridge Construction* specifies the use of air-entrained concrete with a maximum w/cm of 0.45. SCMs may be used at rates up to 35 percent slag cement and/or 25 percent fly ash, by weight of total cementitious material, and the total SCM content may not exceed 50 percent of the weight of total cementitious material.

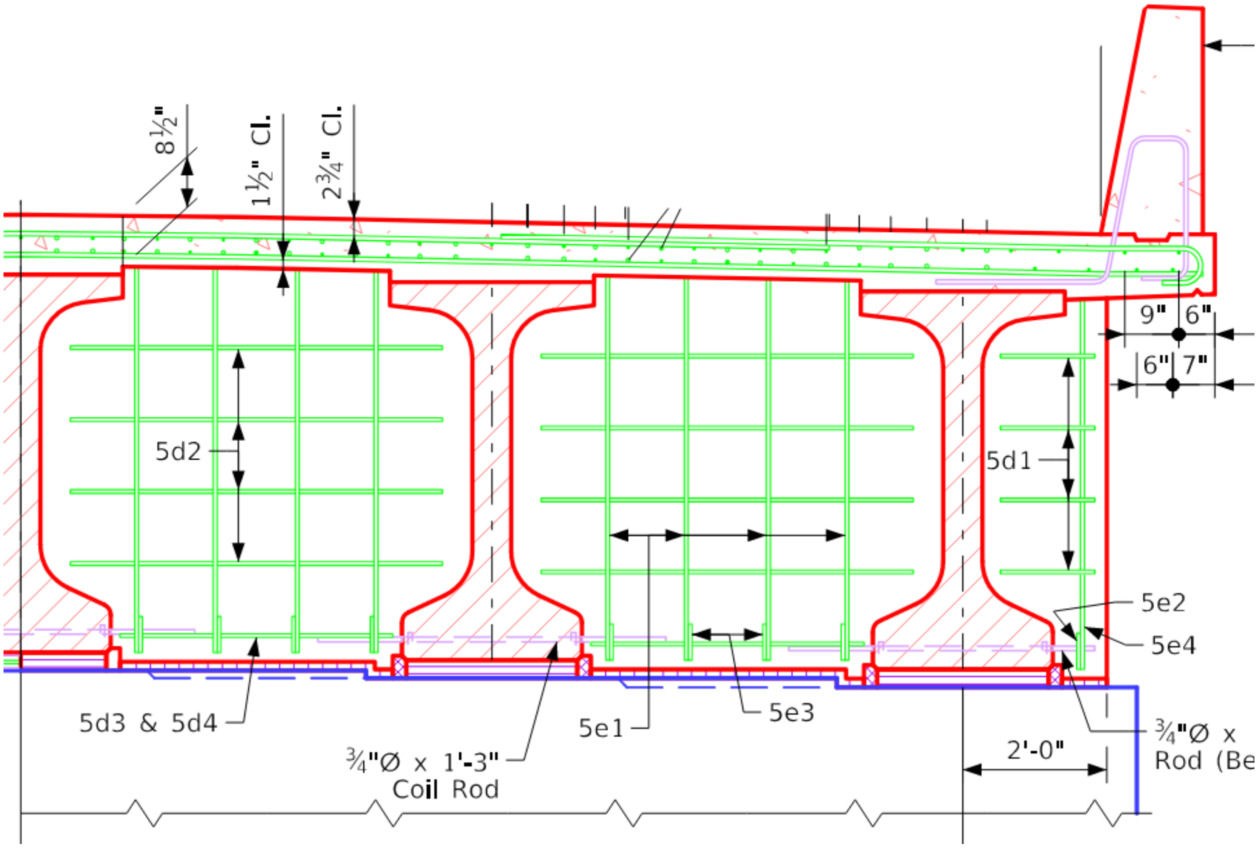

Figure 51. Partial cross-section of bridge deck for a 30-foot roadway. Excerpted from standard sheet 4380-BTE-5, dated July 30, 2024. Epoxy-coated reinforcement is shown in green; stainless steel reinforcement is shown in magenta.

6.1.1.1.2 Barriers

Standard barrier rails for concrete bridge decks are typically 1-foot-7-inches wide at their base and are either 3 feet 2 inches (TL-4) or 3 feet 8 inches (TL-5) in height, as shown in Figure 52. Typical shoulder widths on Iowa DOT bridges vary between 0 and 12 feet, and the Design Criteria Worksheets included in *Iowa DOT Design Manual* Section 1C-1 list preferred shoulder widths of 6 and 10 feet for most roadway types..

Barriers are typically reinforced with epoxy-coated reinforcing steel conforming to ASTM A775; however, Iowa DOT standards specify the use of stainless steel reinforcement at the base of barriers used on interstate and primary bridges. Section 4151.03.E of the *Standard Specifications for Highway and Bridge Construction* permits the following types of stainless steel reinforcement to be used: UNS Designations S31653 (316LN), S31803, or S32304 (2304). Clear cover to the reinforcing steel is specified as 2 inches minimum on the roadway-facing side of the barrier and 2-1/2 inches minimum on the opposite side; a minimum of 2 inches of clear cover is also used for barriers on county bridges.

Section 2403.02 of the *Standard Specifications for Highway and Bridge Construction* specifies that concrete used in barrier rails conform to either Class BR concrete for slip-formed barrier rails or Class C concrete for cast-in-place barrier rails, per Materials IM 529. Alternative requirements for precast mixtures are specified in Section 2407.02. Similar to the bridge deck concrete, concrete mixtures used in the barrier rails have a maximum allowable w/cm of 0.45 and may include SCMs at rates up to 35 percent slag cement and/or 20 percent fly ash, by weight of total cementitious material, for cast-in-place and slip-form mixtures, and up to 35 percent slag cement and/or 25 percent fly ash, by weight of total cementitious material, for precast barriers. The total SCM content may not exceed 50 percent of the total cementitious material for any mixture.

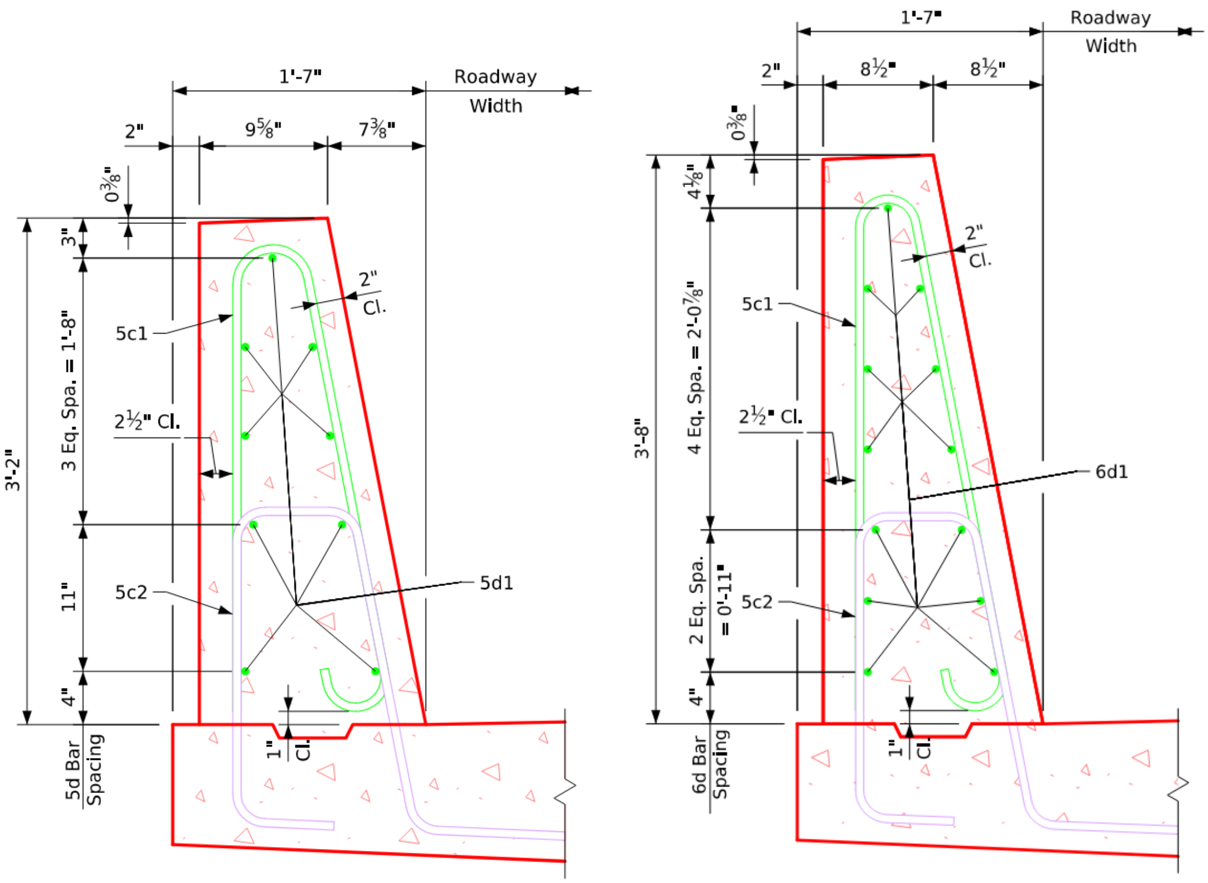

Figure 52. TL-4 (left) and TL-5 (right) barrier rails with stainless steel reinforcement. Excerpted from standard sheets 1020SA-1 and 1020SD-1, dated May 8, 2024. Epoxy-coated reinforcement is shown in green; stainless steel reinforcement is shown in magenta. Alternative barrier rail designs permit the use of epoxy-coated reinforcement where stainless steel is shown on the drawings (e.g., 1020A-1 and 1020D-1).

6.1.1.1.3 Precast Prestressed Girders

A variety of standard precast, prestressed concrete girder designs may be used for Iowa DOT bridges. One such example is presented in Figure 53. For all girders, the *Iowa DOT LRFD Bridge Design Manual* specifies that "except for unusually severe exposures", the minimum clear cover to the stirrups shall be 1 inch and the minimum clear cover to the strands shall be 1-1/2 inches. Although the designs account for the use of epoxy-coated stirrups in combination with other mild reinforcing steel, the use of epoxy-coated reinforcement is currently optional for these elements, except where the stirrups extend into the deck. Strand is typically uncoated carbon steel, low relaxation strand.

Per the standard drawings, the ends of beams located at expansion joints are to be coated and sealed in accordance with Materials IM 570 or IM 491.12. At the time of this report, all materials currently approved for use per these two Materials IMs are gray or clear epoxies. Based on correspondence with Iowa DOT personnel in November 2024, we understand that all Iowa DOT bridges are currently designed such that girder ends are very rarely located beneath expansion joints.

Section 2407.02 of the *Standard Specifications for Highway and Bridge Construction* specifies that precast concrete have a w/cm not exceeding 0.45. Precast concrete may include supplementary cementitious materials at rates up to 35 percent slag cement and/or 25 percent fly ash, by weight of total cementitious material, and the total SCM content may not exceed 50 percent of the total cementitious material.

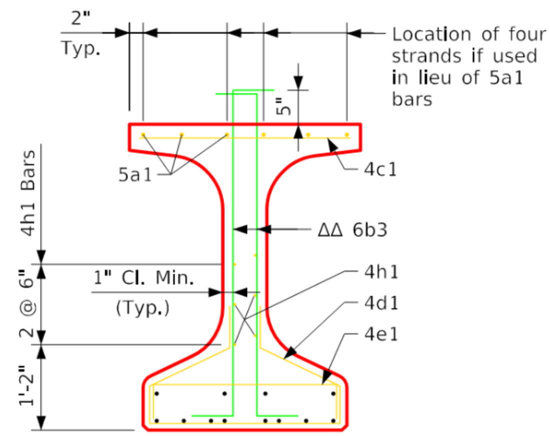

Figure 53. Section view of standard Bulb Tee "C" beam for a 50-foot span length (BTC50). Excerpted from standard sheet 4705, dated October 2, 2024. Epoxy-coated reinforcement is shown in green; uncoated carbon steel is shown in yellow; prestressing strand is shown in black. Similar reinforcing steel types and minimum cover requirements are shown for other standard beam designs.

6.1.1.1.4 Pier Caps

The *Iowa DOT LRFD Bridge Design Manual* specifies that pier caps be designed according to the 8th Edition of the *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD-8) (AASHTO, 2017). This results in a variety of permissible pier cap designs. For pier caps located beneath expansion joints, the *Iowa DOT LRFD Bridge Design Manual* specifies that the pier cap reinforcing be epoxy-coated steel, and that the pier cap concrete be sealed "to reduce deicer damage." Sealers approved for use on pier caps are listed in Materials IM 491.12 Appendix A. At the time of this report, only a single low-viscosity epoxy is included on this list; however, we understand that silane sealers are more commonly used for this application. No specific requirements are listed for pier caps located away from expansion joints, and therefore, it is understood that uncoated carbon steel is permissible in these elements.

AASHTO LRFD-8 specifies a minimum clear cover of 2-1/2 inches to the primary reinforcement for elements with exposure to deicing salts in service and allows cover to the ties to be 1/2 inch less than that of the primary reinforcement; therefore, the minimum allowable clear cover to the ties may be as low as 2 inches.

Section 2403.02 of the *Standard Specifications for Highway and Bridge Construction* specifies the use of air-entrained, Class C concrete for all structural cast-in-place concrete, which includes pier caps and columns (see below). Class C concrete may be substituted with Class D or Class M concrete; however, all structural concrete is subject to the same limits on SCM contents, with no more than 35 percent slag cement and/or 20 percent fly ash, by weight of total cementitious material, permissible in a concrete mixture, and no more than 50 percent total SCM substitution by total weight of cementitious material. Class C and Class D concrete mixtures both have a maximum allowable w/cm of 0.45, while Class M has a maximum allowable w/cm of 0.40.

6.1.1.1.5 Pier Columns

The *Iowa DOT LRFD Bridge Design Manual* specifies that pier columns be designed using the AASHTO LRFD method, which results in a variety of permissible element cross-sections. Iowa DOT requires epoxy-coated reinforcement be used for all pier columns located under an expansion joint or within 25 feet of the edge of any traveled roadway crossed by the bridge; however, uncoated carbon steel is permissible in all other locations. Minimum permissible cover to the spirals or ties depends on the size of the reinforcement and is specified in the *Iowa DOT LRFD Bridge Design Manual* as either 1-1/2 inches for No. 4 ties/spirals or 1-7/8 inches for No. 5 ties/spirals. A minimum 2 inches of cover is required by the AASHTO LRFD-8 for ties/spirals in elements exposed to deicing salts in service; therefore, at locations where epoxy-coated reinforcement is required per the *Iowa DOT LRFD Bridge Design Manual*, it is also assumed that a minimum 2 inches of cover is required to the ties/spirals.

Like the pier caps, Section 2403.02 of the *Standard Specifications for Highway and Bridge Construction* specifies that concrete for pier columns be air-entrained, Class C concrete, with no more than 35 percent slag cement and/or 20 percent fly ash, by weight of total cementitious material, and no more than 50 percent total SCM substitution by total weight of cementitious material. Class C concrete has a maximum allowable w/cm of 0.45, but may be substituted with Class D or M concrete mixtures having maximum allowable w/cm's of 0.45 or 0.40, respectively. The use of corrosion inhibitors is prohibited in pier columns per the *Iowa DOT LRFD Bridge Design Manual*.

6.1.2 Relevant Deterioration Mechanisms

A variety of deterioration mechanisms can affect the durability and service life of reinforced concrete structures. The applicability and severity of a particular deterioration mechanism will depend on several factors, including the properties of the constituent materials, the exposure conditions present, and the protection strategies employed. The following list of deterioration mechanisms may affect service life of reinforced concrete bridge elements in Iowa:

- Alkali-aggregate reactions (AAR) within concrete
- Delayed ettringite formation (DEF) in precast and mass concrete elements
- Sulfate attack of concrete exposed to sulfate-containing soil or groundwater
- Freeze-thaw deterioration and/or deicer scaling of concrete
- Corrosion of steel reinforcement due to chloride ingress (deicer) or carbonation
- Dissimilar metals corrosion of steel reinforcement or embedded elements

- Abrasion of bridge deck surfaces[1]

This list is not an exhaustive list of potential deterioration mechanisms for all reinforced concrete structures, but rather serves as a starting point for the most common materials-related deterioration mechanisms that may affect the service life of reinforced concrete bridge elements in Iowa. Details about these deterioration mechanisms can be found in ACI PRC-201.2-23, *Durable Concrete – Guide* (ACI Committee 201, 2023).

6.1.3 Approach to Verifying Service Life

The service life of a reinforced concrete element is defined as the time at which deterioration exceeds a defined limit. As an example, in the modeling approach outlined in *fib Bulletin 34, Model Code for Service Life Design* (hereafter referred to as "*fib 34*"), the service life of a reinforced concrete member is commonly based on a reliability index of 1.3, which is equivalent to a 10 percent probability that a defined limit state is exceeded (fib, 2006). In practical terms, the end-of-service life defined by *fib 34* is the time at which 10 percent of the surface of the element is expected to exceed that limit state due to a particular deterioration mechanism. [2]

While *fib 34* based analysis is typically based on 10 percent probability of corrosion initiation, this is not the only rational threshold for this type of analysis. In some cases, the service life may be defined as the time to the first major repair, in which case, the end-of-service may be defined as the time at which deterioration is predicted to affect 20 or 30 percent of the element's surface area. In other cases, it may be desirable to assign more stringent criteria with respect to predicted service life due to the critical nature of a structure or to the limited frequency with which inspections may be conducted; for example, below-grade elements may be designed such that the predicted end-of-service is based on the time to 5 percent deterioration.

For the service life analysis presented in this report, two service life limits were considered: the time-to-first maintenance and the time-to-major repair. The time-to-first maintenance was taken as the time at which a particular deterioration mechanism is predicted to affect 5 percent of the element's surface area. The time-to-major repair was taken as the time at which a particular deterioration mechanism is predicted to affect 20 percent of the element's surface. This latter limit is taken as the effective "service life" of the element.

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[1] Based on historic performance of concrete bridge decks in Iowa and the lack of use of chains or studded tires in the winters, abrasion is not considered to be a governing deterioration mechanism with respect to the service life of the bridge decks and will not be discussed further in this report.

[2] It is common to misinterpret the limit state defined by *fib 34* as applying to an entire element (e.g., a 10% probability that a bridge deck will not exhibit corrosion-related deterioration at any location). However, because probabilistic analysis considers variation in key parameters over the surface area of an element, the limit state associated with the reliability index must also be quantified over the surface area of the element. For example, a bridge deck designed with a reliability index of 1.3 and a limit state defined as "corrosion initiation" would be expected to have initiated corrosion over 10% of its surface area at the end of service. A reliability index of 1.3 does not mean that there is a 90% probability that the entire element will be free of corrosion at the end of service, but rather that there is a 90% probability that an individual location on the deck will be free of corrosion.

The objective of the service life analyses performed in this study was to determine the minimum combinations of concrete materials and reinforcing steel that may be used in standard Iowa DOT reinforced concrete bridge elements to achieve target service lives of either 75, 100, or 125 years, corresponding with qualitative service-life designations of "Normal", "Enhanced", and "Maximum", respectively, for a range of exposure conditions. As an additional constraint, the combinations further sought to limit the time-to-first maintenance to no less than 50 years in each case.

In accordance with *fib 34*, the following approaches were considered as means to achieve these service life objectives (fib, 2006):

- The **full-probabilistic approach**, in which critical parameters that govern service life are represented by statistical distributions, and a reliability analysis is performed to determine the percentage of the structure surface anticipated to exceed the defined limit state at the target service life;
- The **deemed-to-satisfy approach**, in which minimum design details, materials, and construction practices are provided to ensure that the service life is reliably achieved; or
- The **avoidance-of-deterioration approach**, in which materials or design details are selected such that a particular deterioration mechanism will not occur.

For the applicable deterioration mechanisms listed above, delayed ettringite formation and dissimilar metals corrosion are managed in this report through avoidance-of-deterioration methods, while sulfate attack, freeze-thaw deterioration and deicer scaling are managed through deemed-to-satisfy approaches. Alkali-aggregate reactions are managed through either avoidance-of-deterioration methods (i.e., using non-reactive aggregates) or through deemed-to-satisfy approaches (i.e., using SCMs to mitigate expansions). Corrosion of reinforcing steel is managed through full-probabilistic modeling to demonstrate that the extent of corrosion-related deterioration will not exceed acceptable limits over the target service life.

6.1.4 Protection Strategies

6.1.4.1 Alkali-Aggregate Reactions

Alkali-aggregate reactions (AAR) refer to a general class of deleterious reactions that occur between certain aggregates that are reactive with alkalis (i.e., sodium and potassium ions) that are present in hardened concrete. Two types of AAR have been recognized: alkali-carbonate reaction (ACR) and alkali-silica reaction (ASR). Both mechanisms can result in internal expansions and extensive cracking of concrete over time, upon exposure to moisture in service.

Three conditions must be present for deleterious AAR to occur: (1) high-alkali pore solution, usually due to high alkali contents present in the cement; (2) reactive aggregate; and (3) available moisture. Avoidance of AAR is best achieved by using aggregates that are non-reactive. In the case of ASR, marginally reactive aggregates can also be mitigated using supplemental cementitious materials (SCMs) following the approaches outlined in ASTM C1778, *Standard Guide for Reducing the Risk of Deleterious Alkali-Aggregate Reaction in Concrete* (2022).

Current Iowa DOT standards do not require screening of aggregates for susceptibility to ACR or ASR; however, Section 4101 of the *Standard Specifications for Highway and Bridge Construction* requires all

ASTM C150 portland cements to contain no more than 0.60 percent total alkali (sodium oxide equivalent) and all ASTM C595 blended cements to contain no more than 0.75 percent total alkali. While limits on alkali content in cements are often specified to reduce the risk of AAR, ASTM C1778 notes that it is important to limit the total alkali loading of the concrete, and not just the alkali content of the cement, to mitigate the risk of deleterious AAR.

For structures designed with service lives of 75 years or more, it is recommended to screen coarse and fine aggregates for their potential for AAR. Aggregates should be either non-reactive with respect to both ASR and ACR, as demonstrated through petrographic examination, physical testing, or documented history of satisfactory performance, or the concrete mixture should incorporate SCMs at minimum dosages necessary to mitigate expansions due to ASR or ACR, as outlined in ASTM C1778.

6.1.4.2 Delayed Ettringite Formation

Ettringite is a compound that generally forms in concrete as it initially hardens. Under certain conditions, primarily exposure to elevated temperatures (> 160 °F) during initial curing, the formation of ettringite is suppressed so that it may then form after the concrete has hardened and is exposed to moisture in service. This late formation of ettringite (i.e., DEF) can lead to internal expansion and subsequent cracking of the concrete.

For DEF to occur, the concrete must: (1) have cementitious materials that are susceptible to DEF, (2) be exposed to temperatures greater than 160 °F during initial curing (either due to heat of hydration, in the case of mass concrete elements, or to external curing in the case of precast elements), and (3) be exposed to moisture in service. Therefore, to minimize the risk for DEF for Iowa bridge elements, measures should be taken to prevent internal temperatures from exceeding 160 °F in mass concrete placements and in precast elements. Guidance for controlling temperatures in mass concrete placements is presented in ACI PRC-207.1-21, *Mass Concrete – Guide* (ACI Committee 207, 2021). If temperatures cannot be maintained below 160 °F, the risk of DEF may also be mitigated using SCMs following the approaches outlined in ACI PRC-201.2-23, *Durable Concrete – Guide* (ACI Committee 201, 2023), or in the optional requirements checklist of ACI SPEC-301-20, *Specifications for Concrete Construction* (ACI Committee 301, 2020).

The current Iowa DOT standard specifications do not include direct provisions for elements considered mass concrete; rather, mass concrete is typically addressed via Developmental Specification DS-23025 (dated October 17, 2023), which defines mass concrete as any placement with a least dimension greater than 4.5 feet and specifies requirements to ensure that the temperature of the mass concrete does not exceed 160 °F. The Iowa DOT *LRFD Bridge Design Manual* identifies elements for which these developmental specifications apply. In addition, Section 2407.03 of the *Standard Specifications for Highway and Bridge Construction* also limits the maximum temperature for precast elements to no more than 160 °F.

6.1.4.3 Sulfate Attack

Water-soluble sulfates in soil or groundwater may cause a chemical attack of the cement paste within the concrete. If reinforced concrete substructure elements are in contact with soil or groundwater containing elevated levels of water-soluble sulfate ions, concrete mixtures should be proportioned to resist deterioration due to this mechanism. The current Iowa DOT *Standard Specifications for Highway and Bridge Construction* and *LRFD Bridge Design Manual* do not directly address identifying and mitigating the risk of sulfate attack; however, ACI SPEC-301-20 (ACI Committee 301, 2020) prescribes minimum cementitious materials requirements for various categories of sulfate exposure, as shown in Table 14. Alternatively, cementitious materials may also be deemed to satisfy the service life requirements if they meet the physical requirements shown in Table 15 when tested per ASTM C1012.

Table 14. Sulfate exposure categories and materials requirements per ACI SPEC-301-20 (ACI Committee 301, 2020)

Exposure Category	Water-Soluble Sulfate (SO_4^{2-}) in soil, percent by mass	Dissolved sulfate (SO_4^{2-}) in water, ppm	Cementitious Materials Requirements
S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$	No restrictions
S1	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1500$ or seawater	ASTM C150 Type II, ASTM C595 with (MS) designation, or ASTM C1157 MS
S2	$0.20 \leq SO_4^{2-} \leq 2.00$	$1500 \leq SO_4^{2-} \leq 10{,}000$	ASTM C150 Type V, ASTM C595 with (HS) designation, or ASTM C1157 HS
S3[1]	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10{,}000$	ASTM C150 Type V, ASTM C595 with (HS) designation, ASTM C1157 HS and pozzolan or slag cement meeting the requirements of Table 15

[1] *Requirements listed for Class S3 Option 1, which has a maximum w/cm of 0.45 and a minimum design compressive strength (f'_c) of 4500 psi.*

Table 15. Sulfate resistance test requirements per ACI SPEC-301-20 (ACI Committee 301, 2020)

Exposure Category	Maximum Length Change per ASTM C1012, percent		
	At 6 months	At 12 months	At 18 months
S1	0.10	No requirement	No requirement
S2	0.05	0.10	No requirement
S3[2]	No requirement	No requirement	0.10

[1] *The 12-month expansion limit applies only if the measured expansion exceeds the 6-month maximum expansion limit.*

[2] *Requirements listed for Class S3 Option 1, which has a maximum w/cm of 0.45 and a minimum design compressive strength (f'_c) of 4500 psi.*

6.1.4.4 Freeze-Thaw Deterioration and Deicer Scaling

Deterioration of concrete due to freezing and thawing occurs when concrete is subjected to multiple cycles of freezing and thawing while "critically saturated" (i.e., with at least 78 to 91 percent of the internal capillary porosity filled with water (Wilson & Tennis, 2021)). The result is a progressive deterioration of the concrete, including delaminations and surface scaling.

The risk of freeze-thaw damage can be mitigated by using properly air entrained concrete mixtures. ACI SPEC 301-20 defines three exposure classes for concrete that will be exposed to cycles of freezing and thawing in service, as summarized in Table 16. Iowa bridge elements are expected to fall primarily within the F2 or F3 categories. For these categories, ACI SPEC 301-20 (ACI Committee 301, 2020) prescribes the target total air contents listed in Table 17, with allowable variations of ± 1.5 percent. The Iowa DOT *Standard Specifications for Highway and Bridge Construction* specifies a target air content of 6.5 percent, with a maximum variation of -1.0 and +2.0 percent, for all cast-in-place structural concrete (Section

2403.03) and 6.5 ± 1.0 percent for all precast concrete (Section 2407.03). The *Specifications* note that these targets are intended to result in a 6.0 percent target air content after placement, which is generally consistent with ACI SPEC 301-20 requirements for concrete having 3/4- or 1-inch maximum size aggregates subject to an F2 or F3 exposure.

While providing at least the minimum total air is often correlated with good freeze-thaw resistance, for structures requiring service lives of 75, 100, or 125 years, it may further be desirable to verify freeze-thaw resistance through air void analysis of the hardened concrete per ASTM C457, or through freeze-thaw resistance testing per ASTM C666, Procedure A. For freeze-thaw resistant concrete, ACI PRC-201.2-23 recommends that the concrete have a spacing factor less than or equal to 0.008 inches, a specific surface greater than or equal to 600 $in.^2/in.^3$, and a total air content that complies with Table 17. For verification of freeze-thaw resistance per ASTM C666, Procedure A, performance-based specifications often require that concrete mixtures demonstrate a minimum durability factor of 90 percent after 300 cycles of freezing and thawing.

To provide resistance to scaling from deicing salts, ACI SPEC-301-20 (ACI Committee 301, 2020) further specifies that the replacement of cement with SCMs not exceed the limits listed in Table 18 for concrete subject to deicer salts. These limits align with current Iowa DOT limits on SCM contents for cast-in-place and precast structural concrete as referenced above; however, if SCM contents exceeding these limits are needed to mitigate the risks of ASR, DEF, sulfate attack, or corrosion, over the design-service life, scaling resistance testing per CSA A23.2-22C or ASTM C672 modified to include measurement of mass loss may be performed to verify the scaling resistance of the concrete mixture. Elements having a target service life of 75 years should exhibit less than 0.5 kg/m^2 (0.9 lb/yd^2) of mass loss after 50 cycles of freezing and thawing, while elements having a target service life of 100 or 125 years should exhibit less than 0.3 kg/m^2 (0.6 lb/yd^2) of mass loss after 50 cycles.

Adequate curing and avoidance of frost-susceptible aggregates are also key to obtaining concrete that is resistant to freeze-thaw and deicer scaling deterioration (ACI Committee 201, 2023).

Table 16. Freeze/thaw exposure categories per ACI SPEC-301-20 (ACI Committee 301, 2020)

Exposure Category	Description
F0	Concrete not exposed to cycles of freezing and thawing
F1	Concrete exposed to cycles of freezing and thawing with limited exposure to water
F2	Concrete exposed to freezing and thawing with frequent exposure to water
F3	Concrete exposed to freezing and thawing with frequent exposure to water and deicing chemicals

Table 17. Total air content requirements for concrete exposed to cyclic freezing and thawing per ACI SPEC-301-20 (ACI Committee 301, 2020)

Nominal Maximum Aggregate Size, in.	Target air content[1], percent	
	F1	F2 and F3
3/8	6.0	7.5
½	5.5	7.0
¾	5.0	6.0
1	4.5	6.0
1-1/2	4.5	5.5

[1] *Total air content delivered shall be within ± 1.5 percent of target listed. Target may be reduced by 1.0 percent for concrete having design compressive strength (f'_c) greater than 5000 psi.*

Table 18. Limits on SCM contents for concrete materials subject to exposure category F3 per ACI SPEC-301-20 (ACI Committee 301, 2020)

SCM	Maximum percent of total cementitious materials
Fly ash or natural pozzolan	25
Slag cement	50
Silica fume	10
Total fly ash + silica fume	35
Total fly ash + slag cement + silica fume	50

6.1.4.5 Dissimilar Metals Corrosion

Galvanic or dissimilar metals corrosion can occur when two or more dissimilar metals are in electrical contact with one another in a moist environment. The more noble metal in the pair becomes the cathode, while the less noble metal becomes the anode and corrodes at a faster rate than it would otherwise corrode on its own. The rate of corrosion of the anode is related to the potential difference between the two metals, the environmental conditions (e.g., availability of moisture and corrosive agents, such as salts), and the relative sizes of the cathode and the anode.

The most effective means of ensuring a 75, 100, or 125-year service life with respect to dissimilar metals corrosion is to avoid contact between dissimilar metals within the element. If two types of reinforcing steel are used, dielectric isolation between the two metals should be provided. Epoxy coatings are considered a type of dielectric isolation. Contact between epoxy-coated reinforcing bars and other types of reinforcement generally do not need to be avoided; however, if epoxy-coated carbon reinforcing steel is used in the same element as an uncoated, more noble alloy (such as stainless steel), the rate of corrosion may be accelerated at holidays and other breaches in the epoxy coating due to the large cathode / small anode effect.

Some current Iowa DOT standards include the use of stainless steel in only portions of the elements that may be subject to higher concentrations of chloride ions, while the rest of the element may be reinforced with either uncoated carbon steel or epoxy-coated steel. In these cases, dissimilar metals corrosion can occur if the two types of reinforcing steel are in contact with one another in a moist, corrosive environment. Generally speaking, when both uncoated carbon steel and stainless steel are used to reinforce a single element, there may be an increase in the rate of corrosion of the uncoated carbon steel relative to the rate that it would otherwise corrode if the element were reinforced entirely with uncoated carbon steel; however, the magnitude of the increase in corrosion rate will depend on the relative areas of the two steel types that are participating in the corrosion cell and may be relatively small for smaller areas of stainless steel relative to uncoated carbon steel, due to the large cathode / small anode effect.

Dissimilar metals corrosion does not only occur when two types of steel are used for reinforcing. Dissimilar metals corrosion can also occur when dissimilar metals are used as reinforcing ties, chairs/supports, inserts, or embedments, or when a metallic component such as a joint or drain is installed in cotact with the concrete reinforcement. The risk of dissimilar metals corrosion is generally reduced when the more noble (cathodic) metal has a smaller area than the more anodic metal (e.g., using stainless steel inserts with uncoated carbon steel reinforcement). As such, while it is better practice to avoid using dissimilar metals or to electrically isolate dissimilar metals whenever possible, if such a condition cannot be avoided, it is preferred that the smaller component be the more noble metal.

6.1.4.6 Corrosion of Reinforcing Steel

Corrosion of embedded reinforcing steel in Iowa bridge elements may occur either when the concentration of chloride ions within the concrete (typically introduced by deicing chemicals) exceeds a critical “threshold” level at the depth of the reinforcing steel, or when the carbonation of the concrete reduces the pH of the pore solution of the concrete surrounding the reinforcement to a pH below about 11. Protective strategies to provide a 75, 100, or 125-year service life with respect to chloride- and carbonation-induced corrosion both rely upon a combination of using low-permeability concrete, sufficient cover to the reinforcing steel, and, where necessary, corrosion-resistant reinforcement. For the minimum cover depths summarized in Table 13, strategies to resist chloride-induced corrosion are considered effective at also resisting carbonation-induced corrosion. Therefore, the analysis using full-probabilistic modeling presented in this section focuses on protection of the element against chloride-induced corrosion.

Full-probabilistic modeling of chloride-induced corrosion was performed for the reinforced concrete members using WJE’s in-house service life model, WJE CASLE™. WJE CASLE is a diffusion-based model for chloride-induced corrosion that is based on an analytical (finite difference) solution to Fick’s Second Law of diffusion. The general approach for full-probabilistic modeling using WJE CASLE aligns with the methodology outlined in Appendix B2 of *fib 34* (fib, 2006).

The full-probabilistic approach to design of new structures recognizes that service life cannot be predicted exactly due to uncertainties in the factors that affect service life, including the exposure conditions, material properties, and as-built conditions. As such, the modeling approach calculates the probability of corrosion initiation at each point in time based on statistical distributions of key factors that govern corrosion of reinforced concrete elements, including the exposure conditions, the transport properties of the concrete, the cover to the reinforcing steel, and the chloride threshold to the reinforcing steel. The predicted time-to-corrosion initiation is then added to a propagation time, which may also be described by a statistical distribution, to determine the probability of corrosion-related damage across the surface of each element as a function of time.

As previously noted, the objective of the service life analyses performed in this study was to determine the minimum combinations of concrete materials and reinforcing steel that may be used in standard Iowa DOT reinforced concrete bridge elements to achieve target service lives of either 75, 100, or 125 years with respect to chloride-induced corrosion. This corresponds to a probability of corrosion-related damage at 75, 100, or 125 years of no more than 20 percent. As an additional constraint, the combinations further sought to limit the time-to-first maintenance (i.e., 5 percent surface damage) to no less than 50 years in each case. All models were performed assuming typical construction practices and exposure conditions for bridges maintained by the Iowa DOT, as described below.

6.1.4.6.1 Chloride Exposure

The primary source of chloride exposures for the reinforced concrete elements is chloride-containing deicers applied to the roadway surfaces. Deicing chemicals applied directly to the top surfaces of the bridge decks may splash onto the surfaces of nearby barrier elements as vehicles pass, resulting in both direct and indirect exposures to chlorides for the reinforced concrete deck and barrier elements. In a similar way, when bridges cross over roadways or other bridges, deicing chemicals applied to those element surfaces may also splash onto the surfaces of overpassing bridge substructure elements, girders, and deck undersides.

Currently, the Iowa DOT uses a salt/brine solution to deice bridge decks in periods of cold weather. Table 19 shows typical deicing salt application rates for various bridge types, as reported by Iowa DOT personnel in August 2023. Per DOT personnel, urban interstates typically receive twice the total amount of deicing salt as state highways, and rural interstates typically receive an intermediate concentration of deicing salts. It is noted that bridges located along critical traffic routes, or near schools, hospitals, and other emergency services may be salted at a more frequent rate during winter weather, so these categories are intended to be descriptive and not prescriptive representations of salting practices in the state. County and city bridges may be salted at different rates, depending on the practices of the specific county or city where the bridge is located.

Table 19. Typical Iowa DOT deicer application rates

Bridge Type	Deicers Used	Typical Application Rate	Total Annual Application
Urban Interstate	Salt/Brine	150 lbs/lane-mile every 1.5 hours or 100 lbs/lane-mile every 1 hour	20,400 lbs/lane-mile per year
Rural Interstate	Salt/Brine	150 lbs/lane-mile every 2 hours or 75 lbs/lane-mile every 1 hour	13,600 lbs/lane-mile per year
State Highway	Salt/Brine	150 lbs/lane-mile every 3 hours or 50 lbs/lane-mile every 1 hour	10,200 lbs/lane-mile per year

To better characterize the impact of these salting practices on the chloride exposures for reinforced concrete bridge elements across the state, WJE performed a field study that included sampling of 81 cores from 10 different bridge decks representing a range of highway types and geographic areas throughout the state of Iowa. Two bridges were examined for each of the following five categories: urban/suburban interstate, rural interstate, US/state highway in the National Highway System, US/state highway not in the NHS system, and county highways not in the NHS system. Chloride concentration profiles were obtained in the laboratory for 55 of the 81 cores and models were fit to the resulting profiles to estimate the surface chloride concentration and the apparent diffusion coefficient (see below) of the concrete represented by the cores. Appendix A provides additional details regarding the locations of the cores and the individual results obtained for each core and bridge.

Based on the chloride concentrations measured from these core samples, it was determined that the deicers applied to the top surfaces of bridge decks result in a surface concentration of chloride ions that ranges between approximately 3200 and 9500 parts per million (ppm) by weight of concrete. While higher chloride concentrations were measured on cores sampled from urban and suburban interstate highway bridges, some of the highest concentrations were measured on cores sampled from rural state highways, which although they are reportedly salted less frequently than interstate bridges, also see less daily traffic to redistribute the salt/brine and may also see fewer snow removal operations. Therefore, rather than consider exposures based on bridge type or reported salting frequency, for the service life analysis presented in this report, the chloride exposure concentrations were instead classified according to three exposure categories, as summarized in Table 20 and below:

- The **very high** chloride exposure category corresponds to an average surface chloride concentration of 9000 ppm. This category generally applies to urban/suburban interstate highway bridges but may also apply to rural state highway bridges that see a high level of salting, for which there is more limited traffic to redistribute the salt/brine or less frequent snow removal operations.
- The **high** chloride exposure category corresponds to an average surface chloride concentration of 6000 ppm. This category generally applies to rural interstate bridges and urban/suburban state highways but may also apply to county highway bridges located near schools, hospitals, or other critical infrastructure.
- The **moderate** chloride exposure category corresponds to an average surface chloride concentration of 4000 ppm. This category generally applies to county highways and rural state highways. Some rural interstate bridges may also be considered to have a "moderate" chloride exposure if salting occurs infrequently.

Table 20. Chloride exposure categories for element subject to direct application of deicing salts

Exposure Category	**Typical Bridge Types**	**Average Surface Chloride Concentration (ppm by weight of concrete)**
Very High	Urban and suburban interstate highway bridges; heavily salted state highways with limited traffic or less frequent snow removal	9000
High	Rural interstate bridges; urban and suburban state and county highways; state and county roads located near critical facilities and infrastructure	6000
Moderate	Rural state and county roads	4000

For chloride exposures resulting from roadway splash, *fib 34* presents a model for estimating surface exposures based on the horizontal and vertical distance of the element to the roadway. Although the *fib 34* model was developed for a specific roadway subject to a specific deicing salt concentration, the model can be scaled to present a reasonable estimate of surface chloride concentrations for other elements and exposure conditions. Therefore, the following equation, based on Equation B2.2-5 of *fib 34* (fib, 2006), was used estimate the maximum surface chloride concentration on the barriers, girders, pier caps, columns, and deck undersides due to splash from deicing salts:

$$C_{s,splash}(h,v) = \frac{C_s}{0.465}\left[0.465 - 0.051\ln(30.5h+1) - \left(0.020 * (30.5h+1)^{-0.187}\right)v\right] \qquad \text{Eq. 1}$$

where C_s is the surface concentration of deicing salts applied to the road (in units of ppm by weight of concrete), and h and v are the horizontal and vertical distances, respectively, from the roadway to the surface of the element, in feet. Figure 54 shows the distribution of chloride concentrations with horizontal and vertical distance from the roadway for a “High” exposure category (i.e., C_s = 6000 ppm).

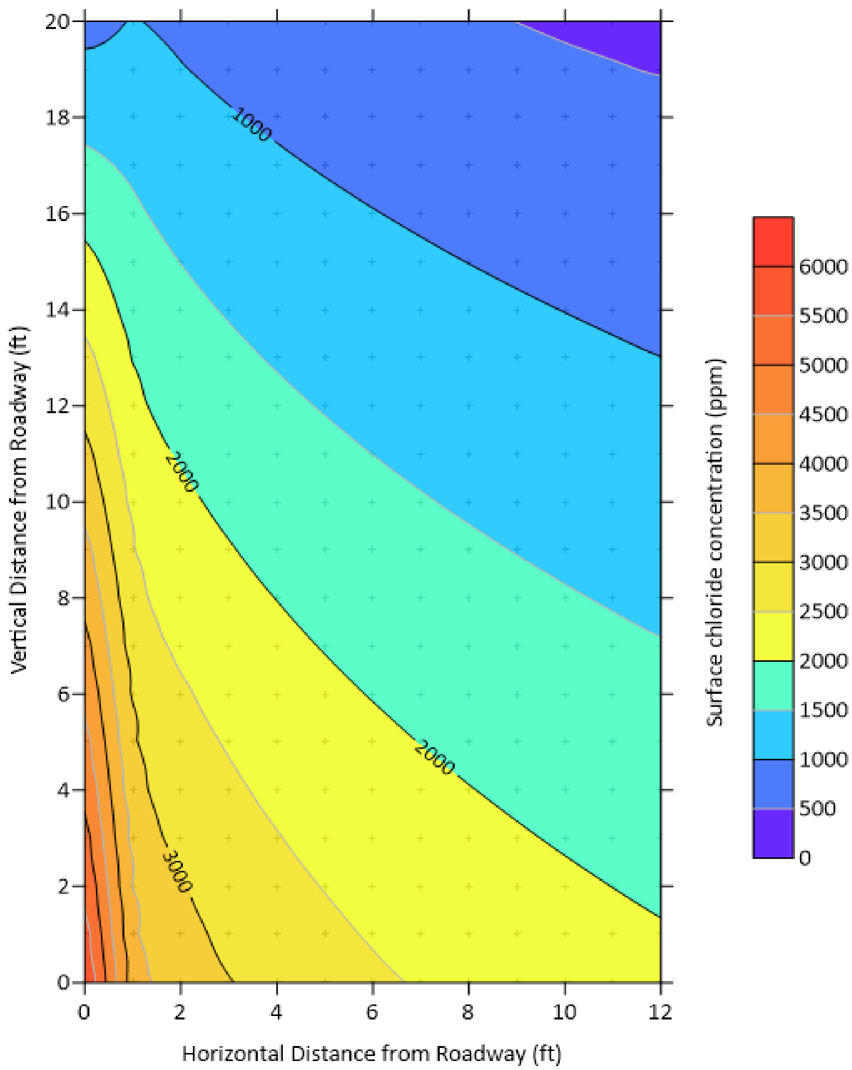

Figure 54. Contour plot showing surface chloride concentrations, in ppm by weight of concrete, due to deicer splash as a function of the element's horizontal and vertical distance, in feet, from a roadway with a "High" exposure to deicing salts (6000 ppm).

It was noted that chloride concentrations measured in cores sampled from the roadway shoulders indicated that the surface chloride concentration in the shoulders was generally independent of the horizontal distance from the edge of the roadway (likely due to deicing practices resulting in direct salting of the shoulders or ponding of deicer runoff on the shoulder surfaces). Therefore, for elements located in shoulders along the salted roadway (i.e., the barriers), it was conservatively assumed that the chloride exposure occurring at the base of the element was equivalent to the surface concentration of chlorides applied to the deck; that is, the horizontal distance from the roadway was assumed to be equal to 0 in Equation 1, irrespective of shoulder width.

Table 21 summarizes the indirect chloride exposures due to splash from deicing salts assumed for the analysis, based on Equation 1. The basis for these exposures is further summarized as follows:

- **Barriers** were assumed to be subject to deicing salt splash from the bridge deck and therefore had the same exposure categories as the bridge decks (i.e., very high, high, and moderate). While shoulder widths may be up to 12 feet on Iowa bridges, the model assumed that all barriers were located at a horizontal distance of 0 feet from the roadway, for the reasons previously noted. All models conservatively considered only the deicer splash at the base of the barrier (v = 0 feet), as this location is expected to govern the service life on the roadway side of the barrier. Therefore, the modeled exposures for the barriers were the same as the modeled exposures for the bridge decks.
- The **deck underside**, as well as the **prestressed girders**, and **pier caps** were assumed to be subject to deicing salt splash from the roadway/feature that is crossed by the bridge. It was assumed that these elements were located a minimum of 16 feet vertical distance above the roadway below[3], and Equation 1 was used to estimate the average surface concentrations of chlorides resulting from these exposures. Since bridge decks are more heavily salted than other roadways, exposure categories for the deck underside, prestressed girders, and pier caps assume that the intersected roadway/feature is either a "heavily salted" roadway, resulting in a Cs of 6000 ppm for use in Equation 1; a "moderately salted" roadway, resulting in a Cs of 4000 ppm for use in 1; or an unsalted feature (e.g., a river or railroad), such that Equation 1 does not apply. For the case in which these elements are located over a feature that is not salted, it was assumed that deicing salts churned up from splash on the deck top surfaces could result airborne chlorides that deposit on the surfaces of the elements beneath at a maximum surface concentration of 500 ppm.

Table 21. Chloride exposure categories for element subject to indirect application of deicing salts

Element	Modeled Distance from Roadway (ft)	Exposure Category	Average Surface Chloride Concentration (ppm by weight of concrete)
Barriers	h = 0 ft, v = 0 ft	Very High	9000
		High	6000
		Moderate	4000
Deck underside, prestressed girder, and pier caps (not subject to chlorides from the deck)	h = 0 ft, v = 16 ft	Over heavily salted roadway	2000
		Over moderately salted roadway	1250
		Not over salted roadway	500

For each element, the maximum surface chloride concentration within the concrete was assumed to be represented by a normal distribution with a mean equal to the value listed in Table 20 or Table 21 and a coefficient of variation equal to 20 percent, based on the variation observed among in the core samples. The surface concentration of chloride ions in the concrete was generally considered to build up linearly over a certain period to this maximum concentration, as illustrated conceptually in Figure 55. Chloride ion concentrations on the deck top surfaces and at the bases of the barriers were assumed to build up to their maximum concentration over a period of 5 years due to the direct application of deicing salts/deicing salt

[3] While some Iowa bridges have vertical clearances as low as 14 feet, most have vertical clearances of at least 16 feet.

run-off on these surfaces. For example, for a "High" chloride exposure concentration of 6000 ppm, the model considered a 1200 ppm increase in the surface chloride concentration every year for 5 years until the concentration reached a maximum value of 6000 ppm; this is illustrated conceptually with the solid red line in Figure 55. Chloride ion concentrations on other elements and surfaces, including the deck underside, were generally assumed to build up to their maximum concentration over a period of 10 years, due to the indirect application of chloride ions on these element surfaces by roadway splash. Thus, for the same "High" 6000 ppm chloride exposure, the models for these element surfaces considered a 600 ppm increase in the surface chloride concentration every year for 10 years, as illustrated by the dashed blue line in Figure 55.

For the pier caps, it was assumed that joint failures in the deck would result in an additional chloride exposure from the top surface, starting 10 years after initial construction. Therefore, for the pier caps located at joints, the modeled chloride concentration was assumed to build up to the initial concentration listed in Table 21 over a period of 10 years, and then build up further to the final concentration listed in Table 20 over an additional period of 10 years, as illustrated by the solid black line in Figure 55.

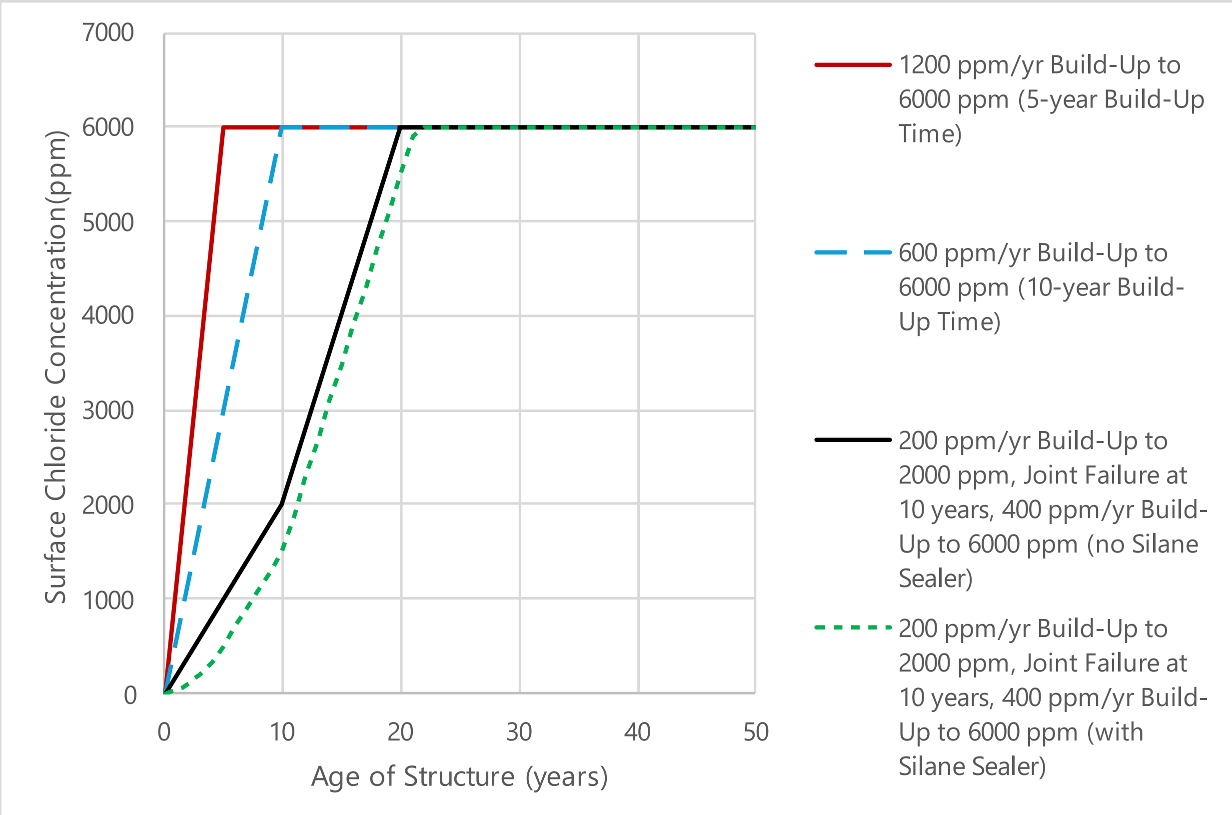

Figure 55. Conceptual illustration of build-up times for various chloride exposures, shown for first 50 years of service.

In addition, current Iowa DOT LRFD standards require sealing of pier caps located at expansion joints to provide supplemental protection against direct deicing salt exposures. We understand that silane sealers are currently used for this purpose. Therefore, the models assumed that pier caps located at joints were treated with a penetrating silane sealer at the time of construction, and that the sealer had a 90 percent effectiveness (i.e., blocked 90 percent of exposed chloride ions and reduced the chloride build-up rate by 90 percent) at its initial application, and decreased to 0 percent effectiveness (i.e., allowed 100 percent of exposed chloride ions to reach concrete surface) over a period of 5 years. The effect of this sealer is illustrated conceptually with the dotted green line in Figure 55. Although epoxy coatings are also approved for sealing of pier caps, epoxy coatings have not been modeled since they are currently not being used in these applications.

6.1.4.6.2 Reinforcing Steel Cover

Cover to the reinforcing steel is a key parameter in the ability of a reinforced concrete element to resist chloride-induced corrosion. Corrosion of the reinforcing steel was modeled for each element based on the minimum design covers summarized in Table 13. Cover to the reinforcing steel was assumed to be described by a lognormal distribution centered around the minimum design cover, with 95 percent of cover depths falling within ACI SPEC-117-10 standard cover tolerances (ACI Committee 117, 2015). A tolerance of 3/8 inch was assumed to apply to the deck and barrier elements (corresponding to a modeled standard deviation of 0.19 inches), while a tolerance of 1/2 inch was assumed to apply to the other elements (corresponding to a modeled standard deviation of 0.24 inches).

6.1.4.6.3 Reinforcing Steel Type

Five classes of reinforcing steel were considered for the service life analysis. Aligning with the approach adopted by the *AASHTO Guide Specification for Service Life Design of Highway Bridges* (AASHTO, 2020), these categories were described as Classes A, B, C, D, and E. Class A reinforcing steel corresponds to uncoated carbon steel, while Classes B through E correspond to reinforcing steel with increasing resistance to chloride-induced corrosion. Epoxy-coated reinforcing steel generally corresponds to Class B reinforcement, while stainless steel may correspond to Class D or E reinforcement, depending on the specific alloy. Other types of corrosion-resistant reinforcement may more closely align to either Class B or C reinforcement, depending on the specific composition of the steel or its coating. Galvanized reinforcing steel (ASTM A767 or ASTM A1094) may be more closely aligned with Class B, while low-carbon chromium reinforcing steel (ASTM A1035) may be more closely aligned with Class B or C.

Each reinforcing class is associated with a different chloride threshold concentration at which corrosion is assumed to initiate on the surface of the reinforcing steel. The chloride threshold is a function of several factors that can vary over the surface of the reinforcement and is therefore described by a statistical distribution for each class of steel. The assumed statistical distributions for each reinforcing bar are summarized in Table 22, in units of percent by mass of cement. To convert the thresholds to modeled units of ppm by mass of concrete, the thresholds are multiplied by the "equivalent cement content" of the concrete mixture design per cubic yard of concrete (see below) and then multiplied by a factor of 10,000 to convert from percent to ppm.

It is noted that the distributions presented in Table 22 differ from those adopted by the *AASHTO Guide Specification for Service Life Design of Highway Bridges* (AASHTO, 2020). For Class A and B reinforcing, this study assumes chloride threshold distributions based on work previously published by Breit (1997) and by WJE (Lawler, Kurth, Garrett, & Krauss, 2021) for uncoated carbon steel and epoxy-coated steel, respectively. For Class C, D, and E reinforcement, while the mean chloride thresholds are assumed to be the same as those used in the *AASHTO Guide*, it has been assumed in this study that the thresholds are described by normal distributions with wider tails than the beta distributions used in the *AASHTO Guide*, to better account for the wide range of thresholds that have been published for these corrosion-resistant reinforcing alloys.

Use of corrosion inhibitors may increase chloride thresholds of uncoated carbon steel or other reinforcing steel types, depending on the dosage and type of inhibitor and steel type. This benefit could be recognized by consideration of a more corrosion-resistant class.

It is noted that although fiber-reinforced polymer (FRP) bars may also be used in certain elements, this type of reinforcement is not subject to deterioration by chloride-induced corrosion and has not been considered in the analysis presented herein.

Corrosion propagation times are also assumed to vary among the different reinforcing steel classes. As outlined in Table 23, propagation times are assumed to follow a normal distribution with a mean value equal to 5 years for Class A (uncoated carbon steel) reinforcement, 10 years for Classes B and C (epoxy-coated and other corrosion-resistant steel) reinforcement, and 20 years for Classes D and E (stainless steel) reinforcement. In all cases, a coefficient of variation of 20 percent has been assumed, resulting in modeled standard deviations of 1, 2, and 4 years, respectively.

Table 22. Assumed statistical distributions for chloride thresholds of various reinforcing classes

Reinforcing Class	**Statistical Distribution**	**Distribution Parameters[1] (percent by mass of cement)**
Class A	Beta Distribution	m: 0.48, s: 0.15, LL: 0.2, UL: 2.0
Class B	Normal Distribution	m: 1.06, s: 0.28
Class C	Normal Distribution	m: 1.5, s: 0.40
Class D	Normal Distribution	m: 3.0, s: 0.75
Class E	Normal Distribution	m: 6.0, s: 0.75

[1] m = mean, s = standard deviation, LL = lower limit, UL = upper limit

Table 23. Assumed statistical distributions for corrosion propagation of various reinforcing classes

Reinforcing Class	**Statistical Distribution**	**Distribution Parameters[1] (years)**
Class A	Normal Distribution	m: 5, s: 1
Class B	Normal Distribution	m: 10, s: 2
Class C	Normal Distribution	m: 10, s: 2
Class D	Normal Distribution	m: 20, s: 4
Class E	Normal Distribution	m: 20, s: 4

[1] m = mean, s = standard deviation

6.1.4.6.4 Concrete Mixture Designs and Transport Properties

Five categories of concrete mixture design were considered for this study, each having increasing levels of SCMs. The baseline concrete mixture design (“No SCM” category) consisted of a straight cement mix at a 0.45 w/c, with no SCMs. The four additional mixture design categories considered increasing quantities of fly ash or slag cement, and the possible use of silica fume (SF), also at a 0.45 w/cm. The "No SCM" and "Low SCM" mixture categories had SCM contents that are similar to those currently used in Iowa DOT reinforced concrete bridge elements. The "Moderate SCM" category had an SCM content that is consistent with the maximum SCM contents currently allowed by Iowa DOT standards. The "Moderate SCM + SF" and "High SCM" mixture design categories had SCM contents that exceeded the limits set in the current Iowa DOT standards; however, such mixture designs may be necessary to achieve extended service life targets in certain applications. All mixture designs were assumed to have a w/cm of 0.45 and a total cementitious materials content of 571 pounds per cubic yard of concrete, which is the minimum cementitious materials content permitted by Iowa DOT standards for cast-in-place concrete mixtures.

As previously described, the chloride threshold for reinforcing steel in units of ppm by weight of concrete is a function of the "equivalent cement content" of the concrete mixture. The equivalent cement content is equal to the total cementitious materials content of the concrete mixture, reduced by a factor that accounts for the consumption of hydroxide ions by the reaction of the SCMs, as follows (Bamforth, 2004):

$$Cement_{eq} = CM_{total}[1 - 0.01 * \max(\mathrm{FA} - 10{,}0) - 0.005 * \max(\mathrm{SG} - 20{,}0) - 0.025(SF)] \quad \text{Eq. 2}$$

where CM_{total} is the total cementitious materials content in the mix design and *FA*, *SG*, and *SF* are the percentages of the total cementitious material that are fly ash, slag cement, and silica fume, respectively. Equation 2 applies only to concrete mixture designs with fly ash contents between 10 and 50 percent, slag cement contents between 20 and 80 percent, and silica fume contents up to 20 percent. The equivalent cement contents assumed for each concrete mixture design category are summarized in Table 24, and are conservatively based on the maximum fly ash and silica fume contents within each category.

The resistance of a concrete mixture to chloride ingress can be defined using an apparent diffusion coefficient, D_a. The apparent diffusion coefficient is a characteristic of the concrete mixture that describes the rate of chloride movement under a concentration gradient (i.e., diffusion), and will decrease over time as the concrete mixture ages and matures. The rate at which the apparent diffusion coefficient decreases with time may be described by an aging factor, *m*, as follows:

$$D_a(t, m) = D_{28}\left(\frac{t}{28}\right)^{-m} \quad \text{Eq. 3}$$

where *t* is the age of the concrete, in days, and D_{28} is the apparent diffusion coefficient of the concrete at a reference age of 28 days. D_{28} is conventionally determined through long-term salt ponding tests, such as ASTM C1556; however, a related parameter, the chloride migration coefficient, may be determined more rapidly using the NT Build 492 test method. While the more rapid NT Build 492 test method is recommended by both *fib 34* and the *AASHTO Guide Specification for Service Life Design of Highway Bridges*, in our experience, the migration coefficient determined through electrical migration tests tends to overestimate the actual rate of chloride transport. As such, the D_{28} based on long-term ponding tests, has been considered for this analysis. This parameter can be estimated for a concrete mixture design as a function of its w/cm and silica fume content using the following equation (Thomas & Bentz, 2000):

$$D_{28} = 10^{-12.06+2.4(w/cm)}\exp(-0.165SF) \quad \text{Eq. 4}$$

where D_{28} is the 28-day apparent diffusion coefficient in units of m^2/s and *SF* is the percentage of the total cementitious material that is silica fume. For a baseline w/cm of 0.45, the predicted 28-day apparent diffusion coefficient according to this equation is 1.05 x 10^{-11} m^2/s (0.512 in^2/yr) for concrete mixtures without silica fume and 4.60 x 10^{-12} m^2/s (0.222 in^2/yr) for concrete mixtures with 5 percent silica fume.

The aging factor is a function of the type and amount of SCMs present in the concrete mixture. For this analysis, the aging factor defined by Thomas and Bentz (2000) was used:

$$m = 0.2 + 0.4\left(\frac{FA}{50} + \frac{SG}{70}\right) \leq 0.6 \qquad \text{Eq. 5}$$

where FA and SG are the percentages of the total cementitious material that are fly ash and slag cement, respectively. The aging factor was assumed to apply for the first 25 years of service, then the apparent diffusion coefficient was held constant thereafter. The assumed aging factors for each concrete mixture design category are summarized in Table 24.

Table 24. Modeled concrete mixture design parameters, at w/cm = 0.45

Concrete Mixture Design Category	Typical SCM Content	Modeled Equivalent Cement Content (lb/yd^3)	Modeled 28-day Apparent Diffusion Coefficient (in^2/yr)	Modeled Aging Factor (--)
No SCM	None	571	0.512	0.2
Low SCM	10-15% fly ash or 15-20% slag cement	542	0.512	0.3
Moderate SCM	20-25% fly ash or 30-35% slag cement	485	0.512	0.4
Moderate SCM + SF	20-25% fly ash or 30-35% slag cement AND 3-5% silica fume	414	0.222	0.4
High SCM	35% fly ash, 50% slag cement, or up to 50% total SCM	428	0.512	0.5

In accordance with *fib 34*, the apparent chloride diffusion coefficient at each age was multiplied by an environmental coefficient, k_e, to account for the influence of temperature on chloride transport. The environmental coefficient is defined by the Arrhenius equation, as follows (fib, 2006):

$$k_e = \exp\left(b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right) \qquad \text{Eq. 6}$$

where b_e is a regression variable described by a normal distribution with a mean of 4800 K and a standard deviation of 700 K, and T_{ref} and T_{real} are the reference temperature and the modeled actual temperature, in degrees Kelvin, respectively. For the model presented in this analysis, T_{ref} is assumed to be equal to 293 K per *fib 34*, while T_{real} is assumed to be modeled by a normal distribution with a mean of 49.1 °F (282.7 K) and a standard deviation of 19.1 °F (10.6 K), based on monthly average temperatures reported by the National Oceanic and Atmospheric Administration (NOAA) for eight cities/metro areas across the state.[4]

[4] Referenced climate data includes Monthly US Climate Normals, 1991-2020, from the following weather stations, accessed July 2024:

6.1.4.6.5 Near-Surface Effects

The WJE CASLE service life model considers chloride transport from the concrete surface to the reinforcing steel primarily via diffusion; however, in the near-surface area of reinforced concrete elements subject to deicer splash, chloride transport may occur more rapidly due to capillary absorption (e.g., resulting from cyclic wetting and drying). The near-surface zone over which capillary absorption governs chloride transport is described in *fib 34* using a parameter called the "transfer function." Because chloride transport is considered more rapid over this zone, the model assumes that the maximum "surface" chloride concentration (as previously described in Table 20 and Table 21) occur at the depth represented by the transfer function, rather than at the true surface of the element.

For elements subject to direct deicing splash conditions, *fib 34* defines the transfer depth according to a beta distribution with a mean value of 0.35 inches, a standard deviation of 0.22 inches, and lower and upper bounds of 0 and 2 inches, respectively (fib, 2006). For elements subject to direct deicer applications and elements subject to airborne chlorides and splash at elevations greater than 5 feet, no transfer function is considered to apply. Therefore, for the service life analysis presented in this study, a transfer depth consistent with *fib 34* was considered to apply to the barrier rails and to the bases of the columns but was not considered to apply to the other elements.

6.1.4.6.6 Influence of Cracking

Cracking that develops in reinforced concrete elements can permit moisture and chloride ions to more easily access the reinforcing steel and supporting elements and can trigger premature corrosion. Both *fib 34* and the *AASHTO Guide Specification for Service Life Design of Highway Bridges* models for chloride-induced corrosion assume that all cracks in concrete are either repaired or are too small to influence chloride ingress. For the purposes of this analysis, this assumption was considered to apply to the precast girders and pier elements, but for the bridge deck and barrier rails, it was assumed that one un-repaired crack would be present every 30 linear feet, which is consistent with a "mild" cracking density as described

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Burlington, IA: Burlington Municipal Airport, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=IA&station=USW00014931

Cedar Rapids, IA: Cedar Rapids Municipal Airport, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=IA&station=USW00014990

Des Moines, IA: Des Moines International Airport, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=IA&station=USW00014933

Fort Dodge, IA: Ft Dodge 5NNW Station, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=IA&station=USC00132999

Mason City, IA: Maon City Municipal Airport, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=IA&station=USW00014940

Sioux City, IA: Sioux City Army National Guard, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=IA&station=USC00137702

Waterloo, IA: Waterloo Municipal Airport, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=IA&station=USW00094910

Omaha, NE: Omaha Eppley Airfield, https://www.ncei.noaa.gov/access/us-climate-normals/#dataset=normals-monthly&timeframe=30&location=NE&station=USW00014942

in the 2022 Iowa DOT Report No. TR-782, *Guide to Remediate Bridge Deck Cracking* (ElBatanouny M. K., et al., 2022).

The impact of cracking on chloride ion ingress was modeled by assuming a higher rate of chloride ingress in the vicinity of the cracks. The apparent diffusion coefficient of cracked concrete was assumed to be described by a normal distribution with a mean value equal to 0.8 in^2/yr (1.6×10^{-11} m^2/s) and a coefficient of variation of 20 percent. Additionally, it was assumed that the apparent diffusion coefficient of the concrete does not change as a function of time (i.e., the aging factor was assumed to be 0). Thus, while the modeled diffusion coefficient of the cracked concrete was approximately 50 percent greater than that of the uncracked concrete at an age of 28 days, at an age of 1 year, it was at least 2.5 times greater and at an age of 10 years, it was at least 4 times greater.

The area where the higher diffusion rate was assumed to apply is referred to as the "crack-affected area" and is estimated based on the cover to the reinforcing steel. If it is assumed that the crack forms along a reinforcing bar and locally increases the diffusion rates over a distance equal to the depth of the reinforcing bar on each side of the crack face, then the crack-affected area can be estimated as twice the cover depth divided by the crack spacing (i.e., 30 feet). This results in crack-affected areas for the bridge deck and barrier surfaces as outlined in Table 25. For the elements in which cracks were considered to apply, the predicted percent corrosion-related damage at each point in time is taken as a weighted average of the predicted percent damage for uncracked concrete and the predicted percent damage for cracked concrete, with the weighting factors equal to the unaffected and crack-affected areas, respectively.

Table 25. Modeled crack-affected areas for bridge deck and barrier elements, based on 1 unrepaired crack every 30 feet

Element Type	Typical Minimum Cover Depth	Crack-Affected Area
Bridge Deck	Top transverse bars – 2-3/4 inches Bottom transverse bars – 1-1/2 inches	Top surface of deck – 1.5.% Bottom surface of deck – 0.8%
Barrier	Roadway-facing surface – 2 inches	Roadway-facing surfaces – 1.1%

6.1.5 Recommendations for Achieving Target Service Life

The following recommendations are presented for achieving target service lives of 75 years ("Normal"), 100 years ("Enhanced"), or 125 years ("Maximum") for reinforced concrete bridge elements constructed and maintained by the Iowa DOT. The strategies are based on current Iowa DOT design details and minimum cover depths and are intended to minimize maintenance activities due to deterioration of the concrete or reinforcing steel over the first 50 years of service. It is noted that these strategies do not consider material availability or cost, nor do they cover all possible types of deterioration that may occur. In addition, it is recommended that a durability engineer be engaged to perform project-specific service life analyses during design prior to construction of structures requiring "Enhanced" or "Maximum" service life.

6.1.5.1 General Recommendations

To achieve target service lives of 75 years or more, all reinforced concrete bridge elements should be produced with durable material and following good construction practices. The following recommendations apply to all reinforced concrete elements:

- **Aggregates**: As described in Section 6.1.4.1, aggregates should be either non-reactive with respect to both ASR and ACR, as demonstrated through petrographic examination, physical testing, or documented history of satisfactory performance, or the concrete mixture should incorporate SCMs at minimum dosages necessary to mitigate expansions due to ASR or ACR, as outlined in ASTM C1778. Further, aggregates should also be resistant to freeze/thaw deterioration, as demonstrated through sulfate soundness testing per ASTM C33 or documented history of satisfactory performance.
- **Cementitious materials**: For concrete used in substructure elements in contact with sulfate-containing soil or groundwater in service, cementitious materials should be proportioned to provide the minimum required resistance to sulfate attack as listed in Table 14 and Table 15.

  For concrete used in bridge decks, barriers, and columns that may be exposed to deicing salts in service, SCM contents should not exceed those listed in Table 18 unless additional testing is performed per CSA A23.2-22C or ASTM C672, as described in Section 6.1.4.4, to verify the scaling resistance of the concrete mixture.
- **Chemical admixtures**: Chemical admixtures should be compatible with one another and with the concrete mixture constituents. Chloride-containing chemical admixtures, including calcium chloride accelerators, should not be used.
- **Air content**: Concrete should be air-entrained to the minimum plastic air contents listed in Table 17. In addition, for structures with “Enhanced” or “Maximum” service life targets, it is recommended that freeze-thaw resistance be verified during mixture prequalification, either through air void analysis of hardened concrete per ASTM C457, or through freeze-thaw resistance testing per ASTM C666, Procedure A, as described in Section 6.1.4.4.
- **Reinforcement**: Reinforcing steel should be handled in a manner that minimizes damage to coatings, and any damage to epoxy or galvanized coatings should be repaired prior to placement of concrete. Contact between dissimilar metals in the final element should be avoided. The final placement of the reinforcement should be verified to comply with applicable construction tolerances.

  While not explicitly considered in this study, non-metallic reinforcing may be considered a viable alternative to reinforcing steel for certain applications. Non-metallic reinforcement is not susceptible to chloride-induced corrosion; however, it may exhibit a gradual reduction of mechanical properties over time due to the alkaline environment of the concrete. If non-metallic reinforcement is used, the potential for reduced mechanical performance over the design service life should be evaluated and considered in the design, if applicable.
- **Mass concrete**: Substructure elements that have the potential to reach or exceed internal temperatures of 160 °F or more should be considered mass concrete, and a thermal control plan should be developed prior to placement of these elements to ensure that internal temperatures do not exceed 160 °F and that temperature differentials between the center and surface of the element do not result in thermal cracking.

- **Precast concrete**: Precast concrete elements may be cured with an external source of heat; however, the external heat should not be applied until after the concrete has reached initial set and heating should not result in internal concrete temperatures that exceed 160 °F.
- **Curing**: Concrete should be cured according to good industry practice. A minimum of 7 days of moist curing is preferred.
- **Early-Age Cracking**: Cracks greater than 0.007 inches in width should be repaired to minimize the ingress of chloride ions and other contaminants into the concrete.

6.1.5.2 Corrosion Mitigation

To minimize maintenance activities resulting from premature corrosion of the reinforced concrete elements, the following recommendations have been developed for each element type, based on the results of the service life analyses performed. The service life analyses were based on limiting the percent surface area exhibiting corrosion-related deterioration to no more than 5 percent at 50 years and no more than 20 percent at the end of each target service life.

As previously noted, additional project-specific analyses should be performed for structures having "Enhanced" or "Maximum" service life targets to better account for the project-specific mixture designs, element details, and exposure conditions. If it is desirable to limit corrosion-related deterioration to a smaller percentage of the surface area, additional analyses would also be needed.

6.1.5.2.1 Bridge Decks

Table 26 shows the predicted service life for reinforced concrete bridge deck elements based on current Iowa DOT standards and current county bridge standards. Predictions are shown for each surface separately, since the top and bottom surfaces of the deck may be subject to different exposure conditions. The predictions presented in Table 26 assume that all reinforcing steel is epoxy-coated and installed with the minimum cover depths outlined in Table 13. Further, it has also been assumed that the concrete mixture designs have SCM contents that comply with current Iowa DOT limits (i.e., up to 35 percent slag cement and/or 20 or 25 percent fly ash, for cast-in-place and precast concrete, respectively, and no more than 50 percent total SCM replacement); elements constructed with concrete mixtures containing no SCMs are predicted to have service lives consistent with the minimum value listed in each range, while elements constructed with concrete mixtures containing the maximum permissible SCM contents are predicted to have service lives at the upper end of each range.

Based on the results presented in Table 26, current Iowa DOT standards for bridge decks are unlikely to achieve service lives of 75 years or more without maintenance for the severe exposure conditions that were observed in Iowa bridges as part of this project. Even shorter service lives were predicted for the county bridge decks, which are currently designed with less cover to the top and bottom mats of reinforcing steel than the standard Iowa DOT designs.

Table 27 summarizes the minimum reinforcing steel class (A, B, C, D, or E) that is necessary to achieve the target service lives of 75, 100, or 125 years for each concrete mixture design category (no SCM, low SCM, moderate SCM, moderate SCM with silica fume, and high SCM) and deck exposure condition considered. All models have assumed concrete with a maximum w/cm of 0.45 and reinforcement installed with the minimum cover depths per Table 13. For each combination of concrete mixture design and chloride exposure, reinforcement providing corrosion resistance for the listed class or greater is expected to provide the target service life. For example, if Class C reinforcement is listed, then any of Class C, D or E reinforcement may be considered to provide the minimum service life; if Class E reinforcement is listed, then only Class E reinforcement may be considered to provide the minimum service life.

As shown in Table 27, use of more corrosion-resistant reinforcement and/or concrete mixtures with higher SCM contents are needed to achieve extended service life targets of 75 years or more for current Iowa DOT bridge deck designs. Given the heavy salting practices used on Iowa DOT bridge decks, it is likely that periodic maintenance and repair activities will be necessary for bridge decks unless stainless steel reinforcement is used in the top mat of the deck. For bridge decks subject to the highest chloride exposures on their top surfaces, a stainless steel (Class D and E) reinforcing top mat is generally necessary to achieve a service life of 75 years or more with only minimum maintenance activities, while top mat reinforcing comprised of epoxy-coated, galvanized, or low-carbon chromium (Class B and C) reinforcement may be able to provide enhanced or maximum service lives with minimum maintenance in less severe exposures. Note, although less corrosion resistant reinforcing steel is generally required for the bottom mat than for the top mat of the bridge deck, it is recommended that a single reinforcement type be used in both mats to minimize the risk of dissimilar metals corrosion. Use of a single reinforcement type is particularly important for epoxy-coated reinforcing steel to prevent a large uncoated bottom mat cathode supporting corrosion at small anodes at any localized damage sites on the top mat.

Other strategies for enhancing the service life of reinforced concrete bridge decks may include use of corrosion inhibiting admixtures, increasing the cover depth, or application of sealers and/or overlays. Assessing the benefits of these strategies would require project-specific analyses. Maintaining proper function of joints, joints seals, and drainage structures will also be critical to ensuring the service life of the bridge deck.

Table 26. Predicted service life for bridge deck elements with respect to chloride-induced corrosion, based on current Iowa DOT practices

Deck surface	Chloride Exposure	Predicted Service Life
Top (standard deck designs; 2-3/4 inches cover)	Very High (9000 ppm)	20-40 years
	High (6000 ppm)	25-50 years
	Moderate (4000 ppm)	35-75 years
Bottom (standard deck designs; 1-1/2 inches cover)	Over heavily salted road (2000 ppm)	50-80 years
	Over moderately salted road (1250 ppm)	> 125 years
	Not over salted road (500 ppm)	> 125 years
Top (county bridge designs; 2-1/2 inches cover)	Very High (9000 ppm)	20-35 years
	High (6000 ppm)	25-45 years
	Moderate (4000 ppm)	30-65 years
Bottom (county bridge designs; 1 inch cover)	Over heavily salted road (2000 ppm)	25-40 years
	Over moderately salted road (1250 ppm)	> 125 years
	Not over salted road (500 ppm)	> 125 years

Table 27. Minimum reinforcement class and concrete mixture design required to achieve service life targets for bridge decks, based on current Iowa DOT design details

Element / Surface	Chloride Exposure	Concrete Mix	Minimum Reinforcement Class Required for Service Life Target		
			Normal	Enhanced	Maximum
Bridge Deck - Top Surface; 2.75 in. cover	Very High (9000 ppm)	No SCM	E	E	E
		Low SCM	D	E	E
		Moderate SCM	D	D	E
		Moderate SCM + SF	C	D	D
		High SCM	D	D	D
	High (6000 ppm)	No SCM	D	D	E
		Low SCM	D	D	D
		Moderate SCM	D	D	D
		Moderate SCM + SF	B	C	C
		High SCM	C	C	D
	Moderate (4000 ppm)	No SCM	D	D	D
		Low SCM	C	D	D
		Moderate SCM	C	C	D
		Moderate SCM + SF	B	B	B
		High SCM	B	B	C
Bridge Deck - Bottom Surface; 1.5 in. cover	Over heavily salted road (2000 ppm)	No SCM	C	C	C
		Low SCM	C	C	C
		Moderate SCM	B	C	C
		Moderate SCM + SF	B	B	C
		High SCM	B	B	C
	Over moderately salted road (1250 ppm)	No SCM	B	B	B
		Low SCM	B	B	B
		Moderate SCM	B	B	B
		Moderate SCM + SF	B	B	B
		High SCM	B	B	B
	Not over salted road (500 ppm)	No SCM	A	A	A
		Low SCM	A	A	A
		Moderate SCM	A	A	A
		Moderate SCM + SF	A	A	A
		High SCM	A	A	A

Element / Surface	Chloride Exposure	Concrete Mix	Minimum Reinforcement Class Required for Service Life Target		
			Normal	Enhanced	Maximum
Bridge Deck - Top Surface; 2.5 in. cover (County Bridges)	Very High (9000 ppm)	No SCM	E	E	E
		Low SCM	E	E	E
		Moderate SCM	D	E	E
		Moderate SCM + SF	C	D	D
		High SCM	D	D	D
	High (6000 ppm)	No SCM	D	D	E
		Low SCM	D	D	D
		Moderate SCM	D	D	D
		Moderate SCM + SF	B	C	C
		High SCM	C	D	D
	Moderate (4000 ppm)	No SCM	D	D	D
		Low SCM	D	D	D
		Moderate SCM	C	C	D
		Moderate SCM + SF	B	B	C
		High SCM	B	C	C
Bridge Deck - Bottom Surface; 1 in. cover (County Bridges)	Over heavily salted road (2000 ppm)	No SCM	C	C	D
		Low SCM	C	C	C
		Moderate SCM	C	C	D
		Moderate SCM + SF	C	C	C
		High SCM	C	C	D
	Over moderately salted road (1250 ppm)	No SCM	B	B	B
		Low SCM	B	B	B
		Moderate SCM	B	B	B
		Moderate SCM + SF	B	B	B
		High SCM	B	B	B
	Not over salted road (500 ppm)	No SCM	A	A	A
		Low SCM	A	A	A
		Moderate SCM	A	A	A
		Moderate SCM + SF	A	A	A
		High SCM	A	A	A

6.1.5.2.2 Barriers

Table 28 shows the predicted service life for reinforced concrete barriers based on current Iowa DOT standards. Because the models conservatively assumed that the same chloride exposure applied over the entire roadway-facing surface of the barriers, the service life models were based on the exposure at the base of the barriers, which was assumed to be equivalent to the exposure of the adjacent bridge deck surface. Although standard barriers are designed with stainless steel reinforcing at their base to resist these higher chloride concentrations, because the models consider only a single exposure, the service life predictions presented in Table 28 are limited by the epoxy-coated reinforcing bar installed in the barrier with a minimum cover of 2 inches on the roadway-facing side, as outlined in Table 13. Further, it has also been assumed that the concrete mixture designs have SCM contents that comply with current Iowa DOT limits (i.e., up to 35 percent slag cement and/or 20 or 25 percent fly ash, for cast-in-place and precast concrete, respectively, and no more than 50 percent total SCM replacement); elements constructed with concrete mixtures containing no SCMs are predicted to have service lives consistent with the minimum value listed in each range, while elements constructed with concrete mixtures containing the maximum permissible SCM contents are predicted to have service lives at the upper end of each range. Based on the results presented in Table 28, current Iowa DOT bridge barriers have service lives that are predicted to range between 15 and 35 years if no maintenance is performed.

Table 29 summarizes the minimum reinforcing steel classes (A, B, C, D, or E) necessary to achieve target service lives of 75, 100, or 125 years for each concrete mixture design category and chloride exposure category considered. To achieve target service lives of 75 years or more for the barriers with only minimum maintenance, Table 29 indicates that stainless steel reinforcement (Class D or E) is necessary due to the high chloride exposures at the base of the barrier. Increasing cover to the reinforcement on the roadway-facing sides of the barriers may allow for less corrosion resistant classes of reinforcing steel to provide extended service lives with only minimum maintenance, as could the use of non-metallic reinforcing steel (e.g., fiber-reinforced polymer [FRP] bars), sealers, and/or barrier coatings; however, these alternatives have not been considered directly in this analysis.

Table 28. Predicted service life for barrier elements with respect to chloride-induced corrosion, based on current Iowa DOT practices

Chloride Exposure	Predicted Service Life
Very High (9000 ppm)	15-20 years
High (6000 ppm)	15-25 years
Moderate (4000 ppm)	20-30 years

Table 29. Minimum reinforcement class and concrete mixture design required to achieve service life targets for barriers, based on current Iowa DOT design details

Element	Chloride Exposure	Concrete Mix	Minimum Reinforcement Class Required for Service Life Target		
			Normal	Enhanced	Maximum
Barrier - 2 in. cover	Very High (9000 ppm)	No SCM	E	E	E
		Low SCM	E	E	E
		Moderate SCM	E	E	E
		Moderate SCM + SF	E	E	E
		High SCM	E	E	E
	High (6000 ppm)	No SCM	E	E	E
		Low SCM	D	E	E
		Moderate SCM	D	E	E
		Moderate SCM + SF	D	D	D
		High SCM	D	D	E
	Moderate (4000ppm)	No SCM	D	D	D
		Low SCM	D	D	D
		Moderate SCM	D	D	D
		Moderate SCM + SF	D	D	D
		High SCM	D	D	D

6.1.5.2.3 *Precast Prestressed Girders*

Current Iowa DOT standards permit the use of uncoated carbon steel prestressing strand and stirrups in the precast prestressed girders; therefore, the predicted service life of these elements will be limited by the relatively shallow cover to this uncoated steel reinforcement. Where bridges pass over salted roadways with a vertical clearance of 16 feet, the service life analyses summarized in Table 30 predict that the girders will exhibit corrosion-related damage over up to 20 percent of their surfaces within 15 to 40 years of service; while not explicitly modeled, girders having greater vertical clearance over salted roadways are expected to have longer service lives than those with only 16 feet of clearance. Where the bridges pass over features that are not salted, uncoated carbon steel strand and stirrups are considered adequate with respect to chloride-induced corrosion, but may begin to exhibit damage due to carbonation-induced corrosion (not modeled) over service lives of 100 or 125 years. Localized corrosion-related deterioration at the girder ends has not been modeled but may further reduce service life if the ends are not adequately protected.

Table 31 summarizes the minimum reinforcing steel classes for both the stirrups and the strands that are necessary to achieve target service lives of 75, 100, or 125 years for each concrete mixture design category and chloride exposure condition. Again, the analyses are based on a vertical clearance of 16 feet, and it should be noted that less corrosion-resistant reinforcing steel may provide adequate service life for bridges with higher clearances.

Based on the full set of analyses performed for the precast prestressed girders, Table 31 indicates that epoxy-coated reinforcing, galvanized, or low-carbon chromium reinforcing steel (Classes B and C) strands and stirrups, at minimum, are needed to achieve service lives of at least 75 years for bridge girders that pass 16 feet over moderately salted roadways, assuming the stirrups and strands have the minimum specified clear cover of 1 and 1-1/2 inches, respectively. While stirrups are available for a variety of reinforcing types, strand is most commonly available in uncoated carbon steel, epoxy-coated steel, stainless steel, and fiber reinforced polymer (FRP) varieties. Therefore, achieving Enhanced or Maximum service lives for prestressed girders located over salted roadways may require the use of epoxy-coated steel, stainless steel or non-metallic strands, corrosion inhibitors in the concrete mixtures, increased cover to the strand, and/or application of sealers or barrier coatings to the girders to provide supplemental protection from corrosion. Assessing the benefits of these strategies would require project-specific analyses that consider the local salting practices and the vertical clearance of the girders.

Table 30. Predicted service life for precast prestressed girders with respect to chloride-induced corrosion, based on current Iowa DOT practices and a vertical clearance of 16 feet

Element	Chloride Exposure	Predicted Service Life
Precast Girder	Over heavily salted road (2000 ppm)	15-25 years
	Over moderately salted road (1250 ppm)	25-40 years
	Not over salted road (500 ppm)	> 125 years

Table 31. Minimum reinforcement class and concrete mixture design required to achieve service life targets for precast prestressed girders, based on current Iowa DOT design details and a vertical clearance of 16 feet

Element / Reinforcement	Chloride Exposure	Concrete Mix	Minimum Reinforcement Class Required for Service Life Target		
			Normal	Enhanced	Maximum
Precast Girder - Stirrup; 1 in. cover	Over heavily salted road (2000 ppm)	No SCM	C	C	D
		Low SCM	C	C	D
		Moderate SCM	C	C	D
		Moderate SCM + SF	C	C	C
		High SCM	C	C	D
	Over moderately salted road (1250 ppm)	No SCM	B	B	B
		Low SCM	B	B	B
		Moderate SCM	B	B	B
		Moderate SCM + SF	B	B	B
		High SCM	B	B	B
	Not over salted road (500 ppm)	No SCM	A	A	A
		Low SCM	A	A	A
		Moderate SCM	A	A	A
		Moderate SCM + SF	A	A	A
		High SCM	A	A	A
Precast Girder - Strand; 1.5 in. cover	Over heavily salted road (2000 ppm)	No SCM	C	C	C
		Low SCM	C	C	C
		Moderate SCM	B	C	C
		Moderate SCM + SF	B	B	C
		High SCM	B	C	C
	Over moderately salted road (1250 ppm)	No SCM	B	B	B
		Low SCM	B	B	B
		Moderate SCM	B	B	B
		Moderate SCM + SF	B	B	B
		High SCM	B	B	B
	Not over salted road (500 ppm)	No SCM	A	A	A
		Low SCM	A	A	A
		Moderate SCM	A	A	A
		Moderate SCM + SF	A	A	A
		High SCM	A	A	A

6.1.5.2.4 Pier Caps

Current Iowa DOT design standards specify different levels of corrosion protection for reinforced concrete pier caps depending on the proximity of the pier to expansion joints. Pier caps located beneath expansion joints are reinforced with epoxy-coated reinforcement and sealed with a penetrating sealer. Based on the assumption that deicing salts from the top surface of the deck will begin to leak onto the pier cap surfaces starting after 10 years of service, Table 32 shows that pier caps located at expansion joints are predicted to have service lives between 30 and 60 years, depending on the combination of exposures from the deck top surface and from the roadway underneath.

As shown in Table 33, to achieve a predicted service life of 75 years or more, stainless steel reinforcement (Class D or E) is generally needed for pier caps located at expansion joints if minimal maintenance is to be performed over the first 50 years of service. Low-carbon chromium reinforcement (Class C), and in some cases galvanized and epoxy-coated bars (Class B) may also be used to achieve 75 years or 100 years of predicted service life for certain combinations of SCMs that are at or greater than the maximum SCM levels currently permitted by Iowa DOT standards. For these elements, service life may further be enhanced and maintenance minimized through the use of corrosion inhibitors or increased cover to the reinforcing steel, as well as through the use of more durable expansion joints (see Section 6.3) that have extended service lives beyond the 10 to 15 years assumed in this study. Assessing the benefits of these strategies would require project-specific analyses.

For pier caps located away from expansion joints, current Iowa design standards do not restrict the type of reinforcing steel used or require the use of penetrating sealers; therefore, uncoated carbon steel reinforcing steel has been conservatively assumed for modeling of these elements. While uncoated carbon steel has been found to provide at least 125 years of predicted service life for pier caps not subject to deicer exposures from above or below, when pier caps are located along salted roadways but away from expansion joints, the predicted service life ranges between only 20 and 65 years for uncoated carbon steel (Table 32). As shown in Table 33, epoxy-coated, galvanized, and low-carbon chromium reinforcing steels (reinforcing Classes B and C) are typically needed to achieve service lives of 75 or more years for these elements.

Table 32. Predicted service life for reinforced concrete pier caps with respect to chloride-induced corrosion, based on current Iowa DOT practices and assumed leaking from the joints starting after 10 years of service

Element / Underside Chloride Exposure	Topside* Chloride Exposure	Predicted Service Life
Pier Cap at Joint / Over Heavily Salted Road	Very High (9000 ppm)	30-40 years
	High (6000 ppm)	30-45 years
	Moderate (4000 ppm)	35-55 years
Pier Cap at Joint / Over Moderately Salted Road	Very High (9000 ppm)	30-40 years
	High (6000 ppm)	30-50 years
	Moderate (4000 ppm)	35-60 years
Pier Cap at Joint / Not Over Salted Road	Very High (9000 ppm)	30-40 years
	High (6000 ppm)	35-50 years
	Moderate (4000 ppm)	35-60 years
Pier Cap Away from Joint	Over heavily salted road (2000 ppm)	20-35 years
	Over moderately salted road (1250 ppm)	35-65 years
	Not over salted road (500 ppm)	> 125 years

** Chloride exposures listed in the second column for pier caps located away from joints are exposures from the underside (i.e. exposure originating from the feature crossed).*

Table 33. Minimum reinforcement class and concrete mixture design required to achieve service life targets for reinforced concrete pier caps, based on current Iowa DOT design details and assumed leaking from the joints starting after 10 years of service

Element / Underside Chloride Exposure	**Topside* Chloride Exposure**	**Concrete Mix**	**Minimum Reinforcement Class Required for Service Life Target**		
			Normal	**Enhanced**	**Maximum**
Pier Cap, At Joint - Over Heavily Salted Road (2000 ppm); 2 in. cover	Very High (9000 ppm)	No SCM	E	E	E
		Low SCM	E	E	E
		Moderate SCM	D	E	E
		Moderate SCM + SF	D	D	D
		High SCM	D	D	E
	High (6000 ppm)	No SCM	D	E	E
		Low SCM	D	D	E
		Moderate SCM	D	D	D
		Moderate SCM + SF	C	D	D
		High SCM	C	D	D
	Moderate (4000 ppm)	No SCM	D	D	D
		Low SCM	D	D	D
		Moderate SCM	C	D	D
		Moderate SCM + SF	B	C	C
		High SCM	B	C	D
Pier Cap, At Joint - Over Moderately Salted Road (1250 ppm); 2 in. cover	Very High (9000 ppm)	No SCM	E	E	E
		Low SCM	E	E	E
		Moderate SCM	D	E	E
		Moderate SCM + SF	D	D	D
		High SCM	D	D	E
	High (6000 ppm)	No SCM	D	E	E
		Low SCM	D	D	E
		Moderate SCM	D	D	D
		Moderate SCM + SF	C	D	D
		High SCM	C	D	D
	Moderate (4000 ppm)	No SCM	D	D	D
		Low SCM	D	D	D
		Moderate SCM	C	D	D
		Moderate SCM + SF	B	C	C
		High SCM	B	C	D

Element / Underside Chloride Exposure	Topside* Chloride Exposure	Concrete Mix	Minimum Reinforcement Class Required for Service Life Target		
			Normal	Enhanced	Maximum
Pier Cap, At Joint - Not Over Salted Road (500 ppm); 2 in. cover	Very High (9000 ppm)	No SCM	E	E	E
		Low SCM	E	E	E
		Moderate SCM	D	E	E
		Moderate SCM + SF	D	D	E
		High SCM	D	D	E
	High (6000 ppm)	No SCM	D	E	E
		Low SCM	D	D	E
		Moderate SCM	D	D	D
		Moderate SCM + SF	C	D	D
		High SCM	C	D	D
	Moderate (4000 ppm)	No SCM	D	D	D
		Low SCM	D	D	D
		Moderate SCM	C	D	D
		Moderate SCM + SF	B	C	C
		High SCM	B	C	D
Pier Cap, Away from Joint; 2 in. cover	Over Heavily Salted Road* (2000 ppm)	No SCM	B	C	C
		Low SCM	B	B	C
		Moderate SCM	B	B	B
		Moderate SCM + SF	B	B	B
		High SCM	B	B	B
	Over Moderately Salted Road* (1250 ppm)	No SCM	B	B	B
		Low SCM	B	B	B
		Moderate SCM	B	B	B
		Moderate SCM + SF	A	A	B
		High SCM	A	B	B
	Not Over Salted Road* (500 ppm)	No SCM	A	A	A
		Low SCM	A	A	A
		Moderate SCM	A	A	A
		Moderate SCM + SF	A	A	A
		High SCM	A	A	A

** Chloride exposures listed in the second column for pier caps located away from joints are exposures from the underside.*

6.1.5.2.5 Pier Columns

Pier columns were not explicitly modeled as they have similar cover and exposure conditions as the barriers and pier caps. Chloride exposures to the pier columns vary over the height of the column, ranging between the exposures modeled for the barriers at their bases and the exposures modeled for the pier caps at their tops. As such, strategies for achieving service lives of 75, 100, and 125 years for the pier columns should be based on the more conservative recommendations of Table 29 (barriers) and Table 33 (pier caps) for the applicable column exposures. Note, when considering exposures at the bases of the columns, exposures are relative to the salting practices of the feature crossed by the bridge and not the top surface of the deck; as such, only the “High” and “Moderate” exposure conditions are applicable. For pier columns for bridges that cross unsalted features (e.g., rivers and railroads), the relevant pier cap exposures are considered to govern over the full height of the column.

6.2 Corrosion Protection of Steel Superstructures

This section focuses on the strategies that can be implemented to protect steel superstructures from corrosion. The corrosion protection strategies that are recommended for steel superstructures in Iowa based on (1) their exposure environment and (2) the desired service life, i.e., “Normal,” “Enhanced,” or “Maximum” as defined in Section 5.1, Target Service Life Design Approach for Alternative Element Designs, are presented at the end in Section 6.2.5, Recommendations. The earlier parts of this section introduce the background information and data that were compiled and the analyses that were performed to support the development of the recommendations. Specifically:

- Section 6.2.1, Principles of Steel Corrosion, provides background on the corrosion mechanisms that can be experienced by steel superstructures and the influence of the type of steel selected for the superstructure on corrosion performance.
- Section 6.2.2, Corrosion Protection Strategies from Literature, presents an overview of the corrosion protection strategies reported in literature for steel superstructures, including how they work, how long they have been used in the United States, and published data or evidence of their performance.
- Section 6.2.3, Current Standard Iowa DOT Practice, describes the corrosion protection strategies currently implemented by the Iowa DOT according to policy.
- Section 6.2.4, Expected Performance of Steel Corrosion Protection Strategies in Iowa, interprets and applies the information collected in the previous sections to predict how corrosion protection strategies applied to steel superstructures in Iowa environments may be expected to perform.

Additionally, supporting discussion within Section 6.2.5, Recommendations, explains how the information contained within these sections was synthesized and leveraged to develop the recommended Corrosion Protection Strategy Selection Table (Table 51). Topics and questions recommended for further investigation or research are identified at the end of this section.

6.2.1 Principles of Steel Corrosion

“Corrosion” is defined as the deterioration of a material caused by reaction with its environment (Roberge, 2006). This definition encompasses a broad array of material degradation mechanisms, many of which are not relevant to steel. For the discussion in Section 6.2, Corrosion Protection of Steel Superstructures, “corrosion” refers to the reduction-oxidation (redox) reactions that cause loss of metal and the formation of corrosion products, such as rust. “Rust,” which is typically iron oxide, is the product that forms specifically as a result of the reaction of iron with its surrounding environment.

6.2.1.1 Corrosion Basics

Steel corrosion can occur in a variety of environments, but always requires the presence of a complete “corrosion cell”, which consists of four components:

1. **Anode.** The anode is the location on the metal where metal loss occurs and is typically identified visually by the presence of rust (or other corrosion products, in the case of metals other than steel or iron). The solid metal is consumed by the anodic reaction, also referred to as the oxidation reaction, in which the metal loses electrons and positively charged metal ions (e.g., Fe^{2+}) are released. The metal ions may subsequently react with available oxygen to form metal oxides, i.e., corrosion products.

2. **Cathode.** The cathode is the location on the metal where the cathodic reaction, also referred to as the reduction reaction, occurs. The cathodic reaction consumes the electrons produced by the anodic reaction, and is referred to as the reduction reaction because the specific molecule that gains the electron(s) consequently experiences a reduction in its charge. The reduction of oxygen is a common cathodic reaction. The location of the cathode is challenging to identify visually because the cathodic reaction does not produce or otherwise lead to visually observable products.
3. **Electrical Current Path.** The anode and the cathode must be electrically connected such that the electrons produced by the anodic reaction can travel to the cathode and be consumed by the cathodic reaction. The anode and cathode may be located on the same metal element, in which case an electrical connection is inherently present, or may be located on different metal elements, in which case the metal elements must be in contact directly or connected electrically in some other manner, e.g., by contact with other metal hardware.
4. **Ionic Current Path.** The cathode and the anode must also be connected ionically by an electrolyte. The cathodic reaction generates ions, which are inherently charged. The electrolyte must be able to carry the charge through to the anode, thereby completing the circuit. The electrolyte may be water, water-saturated concrete, or another medium containing conductive moisture, such as moist debris.

A schematic of a carbon steel corrosion cell is shown in Figure 56. The anode and cathode are on the same metal element and a generic electrolyte is present. In the schematic, the oxidation reaction shown consumes iron metal (Fe) to produce Fe^{2+} ions and electrons. The reduction reaction shown consumes O_2 molecules, water, and electrons to produce hydroxide (OH^-) ions. The specific oxidation and reduction reactions shown are a common combination for carbon steel, but other oxidation and/or reduction reactions may occur instead.

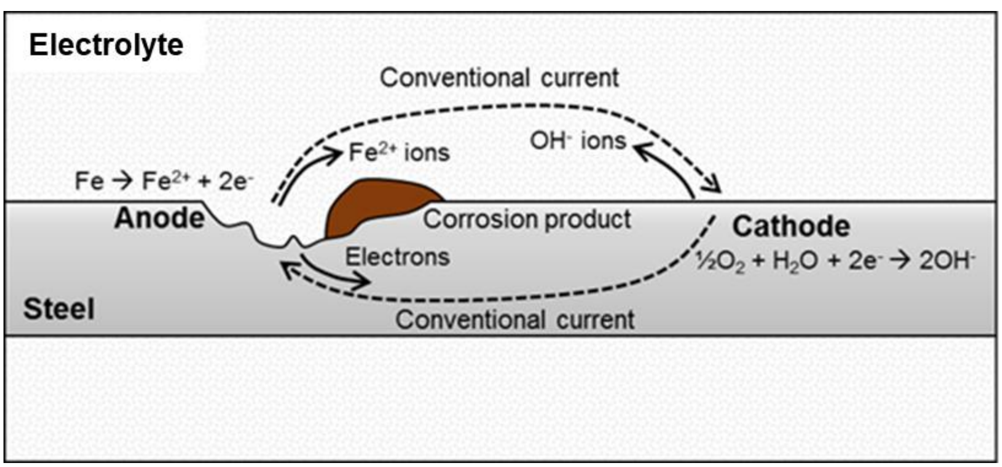

Figure 56. Schematic of a carbon steel corrosion cell.

The circuit is driven by the difference in electric potential between the anode and the cathode. The electric potential of a metal surface is inherently variable across its area such that anodes and cathodes with potential differences are generally present. In such cases, the individual anodes and cathodes may be microscopically small such that each anode-cathode system is referred to as a “microcell.” A difference in electric potential may also exist due to the use of different types of metals or metal alloys, or because exposure conditions, such as the concentrations of specific ions or molecules, vary along the metal surface. In these cases, the anodic and cathodic areas are relatively large, with the extent of the anodic area clearly distinguishable to the naked eye as an area with relatively more section loss. These systems may be referred to as “macrocells”.

6.2.1.2 Metal Passivation

“Passivation” refers to the development of a layer of dense corrosion products on the surface of a corroding metal that inhibits corrosion. When the layer of initial corrosion products is dense enough such that further corrosion occurs at a much slower or even negligible rate, the layer is referred to as a “passive layer” or “patina.” The corrosion rate of the metal when it is in a passivated state is referred to as the “passive corrosion rate.” Only specific metals or metal alloys under specific environmental conditions develop a passive layer.

For example, carbon steel does not passivate when exposed to atmospheric conditions. However, carbon steel does passivate when exposed to highly alkaline environments, such as when it is embedded in concrete. Carbon steel will typically remain passivated in concrete either until a sufficient amount of chloride ions penetrate the concrete to the location of the steel and cause breakdown of the passive layer, or until the pH of the concrete decreases sufficiently such that the passive layer is no longer stable. In both cases, the carbon steel “depassivates” such that the corrosion rate increases to a non-negligible amount.

Stainless steel is known for its high corrosion resistance, which is due to its ability to passivate. Stainless steel consists of up to 1.2 percent carbon by weight and at least 10.5 percent chromium by weight. The chromium content affects the types of corrosion products formed and is what allows stainless steel to form and maintain its passive layer in a variety of environments (Cramer & Covino, Jr., 2003).

6.2.1.3 Types of Corrosion

The type of corrosion can often be identified in the field based on visual inspection and knowledge of the structure and its exposure conditions. Corrosion is commonly classified as either “general” or “localized” depending on the extent of the corroding area. “General corrosion” refers to the uniform corrosion of a relatively large area and is characterized by relatively consistent metal loss; “general corrosion” and “uniform corrosion” are often used interchangeably. “Localized corrosion” refers to relatively quick corrosion of a relatively small area compared to the larger surface area of the metal. While a useful descriptor, the term “localized corrosion” is unspecific and may refer to a variety of corrosion situations. For example, steel beam ends or lengths under leaking expansion joints experience greater rates of corrosion than the general beam length, and may consequently be described as having localized corrosion relative to the rest of the steel beam. As another example, pitting corrosion, in which pits on the order of microns in size form on a metal surface (Cramer & Covino, Jr., 2003), is considered a type of severe localized corrosion. Many other types of localized corrosion exist, but are not discussed here.

While the type of corrosion, e.g., general, localized, pitting, etc., can often be identified in the field, further details and testing may be needed to identify or verify the specific corrosion mechanism(s) at work. The corrosion mechanism refers to the underlying cause of the corrosion. There are many mechanisms by which corrosion can be initiated and driven, and the specific corrosion mechanisms that occur generally depend on the type of metal(s) present, their composition, the environmental exposure, and the service conditions.

For steel superstructures in Iowa, the following four types of corrosion are assumed to be most pertinent and discussed in greater detail below: (1) general atmospheric corrosion; (2) pitting corrosion; (3) crevice corrosion; and (4) galvanic corrosion.

6.2.1.3.1 General Atmospheric Corrosion

General atmospheric corrosion refers to the uniform corrosion that occurs when metal is exposed to the atmosphere. In atmospheric exposures, key environmental parameters that affect metal corrosion rates are time of wetness (TOW); the sulfur dioxide (SO_2) concentration in the atmosphere, particularly in urban and industrial locations; atmospheric salinity, i.e., the presence of chlorides that can be deposited on the structure; and temperature (Revie, 2006). Additionally, the rate of corrosion typically decreases over time due to the buildup of corrosion products such that a bilinear, power, or linear bilogarithmic law is often used to describe atmospheric corrosion rate over time (Revie, 2006). General atmospheric corrosion can be a cause for concern because it results in reduced section area and consequently reduced structural capacity.

6.2.1.3.2 Pitting Corrosion

Pitting corrosion is characterized by the presence of small pits where metal loss has occurred preferentially to varying depths on a metal surface. In severe cases, pitting can cause through-depth perforation(s) of a metal member.

Pitting may be caused by several different mechanisms, but in the context of bridge structures, is likely caused by exposure of a passivated steel element to aggressive ions, such as chlorides. When no passivation layer is present, chloride ions typically cause increased uniform corrosion rates, as acknowledged in Section 6.2.1.3.1, General Atmospheric Corrosion. However, when a passivation layer is present such that uniform corrosion rates are negligible, chloride ions or other anions instead cause local breakdown of the passivation layer such that small areas of unpassivated metal are exposed. Each small area is anodic and corrodes preferentially to the surrounding, passivated metal, resulting in a pit on an otherwise pristine-looking metal surface. Once a pit has been initiated, it may or may not continue to grow; such pits are referred to as “active” and “inactive,” respectively.

The significance of pitting corrosion to structural capacity is case-specific. In certain cases, pitting may simply be an aesthetic concern. In other cases, pitting may compromise structural capacity. The impact of pitting corrosion on capacity depends on the location, number, and depth of the pits.

6.2.1.3.3 Crevice Corrosion

Crevice corrosion is another form of localized corrosion that may be severe. As its name suggests, crevice corrosion occurs in narrow crevices, such as gaps between adjacent steel elements in connections. Crevice corrosion is similar to pitting corrosion, with the crevice functioning as a pre-formed pit, and is of particular concern when passivated metals are used. The crevice geometry can cause a locally severe environment and very high rates of metal loss within the crevice. Metal connections in superstructures are likely to have crevices, and crevice corrosion can be highly damaging if it occurs.

6.2.1.3.4 Galvanic Corrosion

Galvanic corrosion is also called “dissimilar metals corrosion” because it occurs when two different metals are in contact with each other (and an electrolyte is present). In galvanic corrosion, the corrosion cell is driven by the inherent difference in potential between the two metals. The anode is always located on the metal with the lower potential, referred to as the "less noble" metal, and the cathode is always located on the metal with the higher potential, referred to as the "nobler" metal. As a result, metal loss occurs preferentially on the less noble metal while the nobler metal appears unaffected.

Galvanic corrosion is a risk wherever two different metals are in contact with each other or otherwise electrically connected. For example, fasteners are commonly a different metal than the girders or secondary steel elements being fastened. Galvanic corrosion can cause a great amount of damage because corrosion occurs preferentially in the anodic metal until the metal is consumed. Metal loss will proceed particularly quickly if the surface area of the anode is relatively small compared to the cathode area, which increases metal consumption at the anode. Galvanic corrosion should be considered during the design stage and is often avoided by the selection of metals or metal alloys with similar electrochemical potentials, electrical isolation of metal components, and/or the application of protective coatings. Galvanic corrosion can also be leveraged as a protective strategy using cathodic protection or sacrificial coatings.

6.2.1.4 Classification of Atmospheric Environments

The rate of general atmospheric corrosion depends on the severity of the environment. Atmospheric exposure environments have traditionally been described as mild (rural), moderate (industrial), or severe (marine) (Stephens, Gleeson, Mash, & Li, 2019). However, ISO 9223:2012(E), *Corrosion of metals and alloys - Corrosivity of atmospheres - Classification, determination and estimation*, provides a more refined system for categorizing exposure severity with respect to atmospheric corrosion. ISO 9223 is referenced by the *AASHTO Guide Specification for Service Life Design of Highway Bridges, 1st Edition* (2020) and is the basis for categorizing exposure that is employed herein. The categories vary from C1 (Very Low) to CX (Extreme). The qualitative descriptions provided by Annex C of ISO 9223 are presented in Table 34 for reference.

In the context of coatings, practitioners usually reference ISO 12944-2, *Paints and varnishes – Corrosion protection of steel structures by protective paint systems – Part 2: Classification of environments*, instead of ISO 9223 for definitions of atmospheric exposure categories. However, ISO 12944-2 defines the same corrosivity categories as ISO 9223.

Table 34. Description of Typical Outdoor Atmospheric Environments Related to the Estimation of Corrosivity Categories from ISO 9223

Corrosivity Category	Corrosivity	Typical Outdoor Environments - Examples
C1	Very Low	Dry or cold zone, atmospheric environment with very low pollution and time of wetness, e.g. certain deserts, Central Arctic/Antarctica
C2	Low	Temperate zone, atmospheric environment with low pollution (SO_2 < 5 μg/m^3), e.g. rural areas, small towns Dry or cold zone, atmospheric environment with short time of wetness, e.g. deserts, subarctic areas
C3	Medium	Temperate zone, atmospheric environment with medium pollution (SO_2: 5 μg/m^3 to 30 μg/m^3) or some effect of chlorides, e.g. urban areas, coastal areas with low deposition of chlorides

Corrosivity Category	Corrosivity	Typical Outdoor Environments - Examples
		Subtropical and tropical zone, atmosphere with low pollution
C4	High	Temperate zone, atmospheric environment with high pollution (SO_2: 30 µg/m^3 to 90 µg/m^3) or substantial effect of chlorides, e.g. polluted urban areas, industrial areas, coastal areas without spray of salt water or, exposure to strong effect of de-icing salts Subtropical and tropical zone, atmosphere with medium pollution
C5	Very High	Temperate and subtropical zone, atmospheric environment with very high pollution (SO_2: 90 µg/m^3 to 250 µg/m^3) and/or significant effect of chlorides, e.g. industrial areas, coastal areas, sheltered positions on coastline
CX	Extreme	Subtropical and tropical zone (very high time of wetness), atmospheric environment with very high SO_2 pollution (higher than 250 µg/m^3) including accompanying and production factors and/or strong effect of chlorides, e.g. extreme industrial areas, coastal and offshore areas, occasional contact with salt spray

6.2.2 Corrosion Protection Strategies from Literature

Design strategies that avoid or slow the deterioration of steel superstructures due to corrosion include the following (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014; Azizinamini, Power, Myers, & Ozyildirim, Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems, and Components, 2014; AASHTO, 2020; Stephens, Gleeson, Mash, & Li, 2019):

- Use of corrosion-resistant steels,
- Application of protective coatings,
- Elimination of joints, and
- Incorporation of proper design details.

An overview of each strategy and why it is effective in protecting steel superstructures from corrosion is provided below. The history of use and current understanding of the effectiveness of the various strategies is also included where available.

6.2.2.1 Corrosion-Resistant Steels

In the context of steel bridges, corrosion-resistant steels include weathering steel and potentially several types of stainless steel. Corrosion-resistant steels rely on the premise that they will passivate under atmospheric exposure conditions and therefore corrode at such a low rate that coatings are not necessary. In the case of weathering steel, despite the presence of the passive layer, corrosion still occurs at a non-negligible rate and some amount of “sacrificial steel” that can be lost without compromising the ability of the structural elements to provide sufficient capacity may be included in the original design. Stainless steels are usually assumed to corrode at a negligible rate when passivated such that a sacrificial steel thickness is considered unnecessary.

6.2.2.1.1 Weathering Steel

Weathering steel has enhanced corrosion resistance in atmospheric exposures compared to carbon steels. This enhanced resistance to atmospheric corrosion is achieved by alloying carbon steel with a variety of other elements, up to a few percent maximum. These alloying elements, particularly copper (Cu), chromium (Cr), nickel (Ni), and silicon (Si), cause the weathering steel to develop a patina (i.e. to passivate) under atmospheric conditions (Revie, 2006). Phosphorus (P) also greatly improves atmospheric corrosion resistance, but is limited in weathering steels used for bridges because it decreases the weldability of the steel.

Weathering steel performance depends on the patina that develops under atmospheric conditions, but experience has shown that a high time of wetness or humidity, exposure to ponded water, or the presence of aggressive chlorides will weaken or prevent the patina from forming, resulting in corrosion rates comparable to or worse than those of carbon steel. Chlorides can also cause pitting of weathering steel. As a result, the characteristics of the bridge site are taken into account when considering the use of weathering steel, and joint, drainage, and steel details that minimize water leakage onto the member and ponding are essential for durable weathering steel structures. Protective coatings are also often implemented in tandem with weathering steel, particularly under expansion joints and when weathering steel is embedded directly in concrete.

Weathering steel has a long history of use in the United States. It was first developed by the U.S. Steel Corporation in 1933, under the product name "CORTEN" (Revie, 2006), after which other steel manufacturers began producing their own weathering steel products. Initially considered a "maintenance-free" alternative to bridges with painted carbon steel superstructures, the first weathering steel bridges in the United States were constructed in 1964 in New Jersey over the New Jersey Turnpike (AISC Marketing, Inc., 1993), and by the City of Des Moines in Iowa, carrying Iowa 28 over the Raccoon River (Crampton, Holloway, & Fraczek, 2013). The Michigan DOT constructed its first uncoated weathering steel bridges shortly after in 1965 in the Metropolitan Detroit area (McCrum & Arnold, 1985). Several years later in 1968, ASTM A 588, *Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance*, was published such that designers could specify the use of ASTM A 588 Grade 50W steel, in which the "W" refers to weathering steel. ASTM A 709, *Standard Specification for Structural Steel for Bridges*, was published shortly after in 1974 and compiled the various standards used for bridge steel, including ASTM A 588, such that ASTM A 709 Grade 50W or 70W could be specified (Ocel, 2021).

However, the Michigan DOT identified undesirable corrosion of its weathering steel bridges approximately 7 years after construction with advanced deterioration visible after 10 to 15 years of exposure due to the lack of formation of the patina (McCrum & Arnold, 1985). As a result, the Michigan DOT implemented a moratorium on the construction of uncoated weathering steel bridges in urban and industrial environments as well as sites with tunnel-like environments in 1979, followed by a full moratorium on new uncoated weathering steel bridges in 1980. The Iowa DOT and several city and county agencies had constructed additional uncoated weathering steel bridges, but the Iowa DOT discontinued the use of weathering steel bridges in approximately 1980 based on Michigan's experience. At this time, the first Iowan uncoated weathering steel bridge carrying Iowa 28 over the Raccoon River was found to have multiple areas where the protective patina had not formed, which was attributed to exposure to deicing chemicals, and the bridge was remedially painted in the mid-1980s. The other 4 uncoated weathering steel bridges that had been constructed by the Iowa DOT between 1964 and 1972 had satisfactory patinas in the 1980s (Crampton, Holloway, & Fraczek, 2013).

Following the moratorium placed by the Michigan DOT, many other state DOTs minimized or eliminated the use of uncoated weathering steel bridges because of the experience reported by the Michigan DOT and/or the observation of similar conditions within their own jurisdictions (Crampton, Holloway, & Fraczek, 2013; McDad, Laffrey, Dammann, & Medlock, 2000). As of 1987, 4 state DOTs did not have any weathering steel bridges, largely because weathering steel was not advantageous in their arid climates or not economical, and 14 state DOTs were former users of weathering steel bridges (Albrecht, Coburn, Wattar, Tinklenberg, & Gallagher, 1989). In response to this scepticism, the American Iron and Steel Institute (AISI), the National Cooperative Highway Research Program (NCHRP), and the Federal Highway Administration (FHWA) conducted various work throughout the 1980s to characterize the reported performance issues and provide guidelines for the successful use of weathering steel. Based on this work, the FHWA attributed the undesirable performance to the use of uncoated weathering steel in improper locations or conditions and unrealistic expectations of its performance, and issued Technical Advisory 5140.22, Uncoated Weathering Steel in Structures, in 1989 to provide guidance identifying the types of corrosive environments for which uncoated weathering steel is and is not a suitable option. Locations where the use of uncoated weathering steel was recommended to be considered with caution are (FHWA, 1989):

- Locations in coastal (marine) environments;
- Locations in environments with frequent high rainfall, high humidity, or persistent fog;
- Locations in industrial environments where the structure may be exposed to concentrated chemical fumes;
- Locations with grade separations that cause "tunnel-like" conditions; and
- Locations with low-level water crossings.

Japan experienced a similar issue in the 1960s in which weathering steel did not perform as desired, sparking improvements in the alloy design and study of the corrosiveness of the environments across Japan. One study determined that in order for weathering steel to experience a corrosion rate less than 0.3 mm over 50 years, which corresponds to 12 mils of loss over 50 years or an average corrosion rate of 0.24 mils per year, the deposition rate for air-borne salt should not exceed 0.05 mg per dm^2 per day (Revie, 2006). In contrast, the United Kingdom Department of Transport has suggested a maximum chloride deposition rate of 0.1 mg per dm^2 per day, and the FHWA estimated that uncoated weathering steel could be used at locations with chloride deposition rates up to 0.5 mg per dm^2 per day, on average across the United States. The different recommended limits are a product of the different moisture conditions between the countries (FHWA, 1989).

The findings of the research performed for NCHRP in the 1980s indicated that uncoated weathering steel bridges were performing satisfactorily when located in suitable environments and with proper detailing. The authors also emphasized that weathering steel is not a maintenance-free material and that routine cleaning of the steel as well as repair of leaking joints, drains, and other bridge elements should be expected (Albrecht, Coburn, Wattar, Tinklenberg, & Gallagher, 1989). As a result of the research, bridge owners took renewed interest in weathering steel bridges. The Iowa DOT began constructing weathering steel bridges again in approximately 1993 (Crampton, Holloway, & Fraczek, 2013).

Following the resurgence in uncoated weathering steel bridge construction, the FHWA commissioned a study to review the performance of uncoated weathering steel bridges constructed after the dissemination and presumed implementation of the guidelines published in the late 1980s, and potentially to update the guidelines based on performance data. Another survey of state practice and perception of uncoated weathering steel bridges was conducted in 2012 as part of this effort. Of all the states, only the Hawaii DOT had no uncoated weathering steel bridges, which aligned with the 1987 survey in which the Hawaii DOT responded that steel bridges were not economical compared to concrete. Four other state DOTs were former users, including the Michigan and Alaska DOTs, which were the only two DOTs to report a negative perception of uncoated weathering steel bridges. The other DOT respondents to the 2012 survey reported having no overall performance problems with uncoated weathering steel bridges, or a mostly positive experience with some performance drawbacks, typically associated with specific environments or scenarios (McConnell, Shenton III, Bai, & Rupp, 2024). As a result of the quantitative performance analysis, the research team identified three deicing chemical environments in which uncoated weathering steel would be expected to demonstrate accelerated corrosion. These three environments, referred to as "inferior environments" and defined in Table 35, are all heavily aggressive due to chloride-contaminated spray from traffic on an underlying roadway onto the bottom flanges of the girders. The researchers noted that these environments should be considered aggressive for I-girder bridges but not necessarily for box girder bridges, whose geometry inherently avoids the issue of chloride-contaminated water catching on a flange. The presence of such environments across Iowa and implications for the performance and appropriate use of uncoated weathering steel in Iowa are discussed in Section 6.2.4.2, Performance of Weathering Steel.

Table 35. Deicing chemical environments heavily aggressive towards uncoated weathering steel bridges (McConnell, Shenton III, Bai, & Rupp, 2024).

Variable	Inferior Environment 1	Inferior Environment 2	Inferior Environment 3
Crossing type	Highway	Highway	Highway
Vertical underclearance	No limit	≤ 18 ft	≤ 18 ft
ADT of roadway crossed	≥ 100,000 vehicles	≥ 10,000 vehicles	≥ 4,000 vehicles
Average annual snowfall	≥ 18 inches	≥ 22 inches	≥ 22 inches
Atmospheric Cl^-	Not applicable	Not applicable	≥ 0.1 ppm

6.2.2.1.2 ASTM A1010 (ASTM A709 Grade 50CR) Stainless Steel

Stainless steel has traditionally been considered too expensive for bridge superstructures. However, due to the poor suitability of uncoated weathering steel for certain corrosive environments, the high maintenance costs associated with coated steel, and a growing need to minimize traffic disruption associated with bridge maintenance and replacement, DOTs have taken increasing interest in stainless steel and the potential maintenance-free longevity it may offer to superstructures, particularly for environments where uncoated weathering steel does not perform well. Using life cycle cost analysis, Fletcher (2011) showed that in a severe environment, stainless steel complying with ASTM A1010, *Standard Specification for Higher-Strength Martensitic Stainless Steel Plate, Sheet, and Strip*, is a cost-effective alternative to conventional painted steel requiring cyclic repainting when considering a 125-year analysis period, assuming the stainless steel has no maintenance needs.

Stainless steel by definition contains a minimum chromium content of 10.5 percent (Fletcher F. B., 2011). The chromium gives stainless steel its superior corrosion resistance by supporting the formation of chromium oxide corrosion products, which make a thin, continuous, tightly adherent, and highly effective passive layer. ASTM A1010 stainless steel (referred to herein as A1010 steel) specifically is a dual-phase stainless steel consisting of martensite and ferrite and has a chromium content between 10.5 and 12.5 percent, corresponding to a nominal chromium content of 12 percent (Fletcher F. B., 2011). In 2017, the initial cost of A1010 steel was estimated to be approximately 1.6 times the cost of a galvanized steel alternative, considering only fabricator costs (Ault & Dolph, 2018), while a case study by the Virginia DOT estimated A1010 steel to be approximately 1.3 times the cost of a painted carbon steel alternative, considering fabrication and erection costs (Sharp, Provines, Moruza, Via, Jr., & Harrop, 2019). Efforts to decrease the initial cost of the corrosion-resistant steel by decreasing the chromium content to as low as 5 percent were made, but unlike A1010 steel, these potential alternatives did not demonstrate adequate impact toughness to be feasible for bridge construction (Fletcher F. B., 2011).

Despite its high initial cost, A1010 steel began to receive interest from the bridge community in the 2000s and 2010s because of its relatively high corrosion resistance, as demonstrated in laboratory testing and field exposure test sites. During marine site exposure testing conducted at Kure Beach, North Carolina, A1010 steel samples demonstrated a 4-year corrosion loss approximately one tenth that of weathering steel samples (Fletcher F. B., 2011). In a later laboratory evaluation, A1010 steel demonstrated approximately 10 times the corrosion resistance of weathering steel when exposed to a 5 percent NaCl solution and approximately 15 times the corrosion resistance of weathering steel when exposed to a 3 percent NaCl solution (Fletcher F. B., 2011). In a field exposure test in which coupons were placed on the flange of Moore Drive Bridge in Rochester, New York to compare performance in exposure to chloride-contaminated splash from underlying roadways, the A1010 steel coupons demonstrated a corrosion rate approximately a quarter of that of the weathering steel coupons (Phares, Shafei, & Shi, 2020). Based on the relatively high corrosion resistance demonstrated by A1010 steel in these studies, several DOTs have proceeded with experimental A1010 bridge construction.

The first bridge in the United States to be constructed with A1010 steel was built in 2004 carrying Fairview Road over the Glen-Colusa Canal in Colusa, California. The bridge was one of California's Innovative Bridge Research and Construction Program projects of 2002 and not only used the relatively unknown material A1010 Grade 50 steel for its girders, but also used an innovative structural design. The girder steel was only 4 mm thick with no corrosion allowance (Fletcher F. B., 2011). The next A1010 steel bridge was constructed in 2012 by the steel manufacturer ArcelorMittal over a local creek at their steel plant in Coatesville, Pennsylvania (ArcelorMittal, 2013). At approximately the same time, the Oregon DOT conducted a rigorous study of A1010 steel and investigated suitable welding procedures for the material before constructing the next two A1010 bridges in 2012 and 2013. The bridge constructed in 2012 carries Highway 138 over Dodge Creek and is located northeast of Stephens, Oregon. The bridge constructed in 2013 carries Columbia River Highway over Mill Creek, near Astoria, Oregon. The Iowa DOT later constructed a continuous steel girder bridge using both weathering steel (ASTM A709 Grade 50W) and A1010 steel in 2016. The bridge carries County Road K25 (275th St) over I-29 in Salix, Iowa. The two southernmost steel girders, including the flanges, webs, all splice plates, bearing stiffeners, intermediate diaphragm stiffeners, and sole plates at bearing locations, were ASTM A1010 steel while the remaining four girders were weathering steel. Based on the design drawings for the bridge and the current Iowa DOT Standard Specifications for Highway and Bridge Construction (2023), the A1010 steel components are in contact with other grades of steel or types of metals. The bolts and fasteners used at the A1010 field splices and the connections between the weathering steel diaphragms and the A1010 steel stiffener plates are hot-dip galvanized carbon or weathering steel. Carbon steel shear connector studs were welded to the tops of the girders. Additionally, while the weathering steel girders were painted over the length embedded within the concrete abutments and an additional foot beyond the face of the abutment concrete, the A1010 steel girders were not painted at any locations.

The Virginia DOT constructed its first A1010 steel bridge in 2016 in Waynesboro, Virginia. The bridge carries Route 340 (Main St) over South River, has a low water clearance, and is located downstream of a chemical plant. The site was selected as a trial location for A1010 steel because the existing steel girder bridge required replacement and conditions were unsuitable for uncoated weathering steel. Based on the experience, the Virginia DOT recommended that design guidance for A1010 steel be developed and A1010 steel girder bridges be selected for use in highly corrosive environments. The Virginia DOT has since constructed A1010 steel girder bridges in the Eastern Shore, a peninsula located between the Chesapeake Bay and Atlantic Ocean (Provines, Sharp, & Moruza, 2023).

The majority of the above trials focused on assessing the constructability and structural performance of A1010 steel and the development of guidance for its implementation, with good results. In 2017, ASTM A1010 steel was added to ASTM A709 as Grade 50CR, and ASTM A709 Grade 50CR steel has continued to be applied in or researched by other states, including Idaho, North Carolina, and Pennsylvania (Taylor, Ebrahimpour, Ibrahim, & Mashal, 2024; Short Span Steel and Bridge Alliance; NCHRP, 2024).

6.2.2.1.3 Duplex Stainless Steel

Duplex stainless steel is a dual-phase stainless steel consisting of austenite and ferrite and contains a relatively high chromium content between 22 and 27 percent (Cramer & Covino, Jr., 2003). Multiple duplex stainless steel alloys are available and several have been used in bridge applications, including UNS 32205, UNS 32304, and UNS 32101 (Ault & Dolph, 2018). The first duplex pedestrian bridge was constructed in Switzerland in 1999 while the first duplex vehicular bridge was constructed in Spain in 2005 (Provines, Sharp, Ozbulut, & Daghash, 2019). One of the first records of use of duplex stainless steel in a bridge application in the United States was for the Harbor Drive Pedestrian Bridge in San Diego in 2011. The Virginia DOT recently completed a study investigating the feasibility of using various types of corrosion-resistant steels, including ASTM A709 Grade 50CR, ASTM A1035CS[5], and duplex stainless steel, and recommended further study of duplex stainless steels as a result because of their relatively good mechanical properties compared to the other steels, successful implementation in past bridge applications, and the presence of pre-existing guidance for their design and fabrication for structural members (Provines, Sharp, Ozbulut, & Daghash, 2019). However, there has been relatively little investigation into the use of duplex stainless steel for bridges compared to A1010 steel and at this time, duplex stainless steel is considered a specialty material best used for projects with high architectural requirements or highly corrosive environments, especially marine environments (Ault & Dolph, 2018).

6.2.2.2 Protective Coatings

[5] ASTM A1035, *Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement*, is specific to reinforcing steel and not available for use in superstructures at this time, but the researchers were able to procure steel plates that met the chemical requirements of ASTM A1035 for study.

Protective coatings include paint systems, galvanizing, metallizing, and duplex coating systems. In the context of coatings, "duplex" is unrelated to duplex stainless steel and instead refers to the dual use of a paint coating on top of a metallic coating, i.e., galvanizing or metallizing.

6.2.2.2.1 Paint Systems

Paints consist of two primary types of components: (1) the vehicle, and (2) the pigmentation. The vehicle refers to the liquid components that facilitate the placement of the paint, including the resin or binder prior to cure and any added solvents. The resin is further responsible for cohesively holding the paint together and adhesively bonding the paint to the substrate after cure. The pigmentation refers to the insoluble raw materials carried by the vehicle, such as colorants. Azizinamini et al. (2014) provides a comprehensive overview of the components of a paint. The components used and their proportions greatly impact a paint's performance, from ease of application to effectiveness at protection to coating durability. Paints are commonly identified by the generic type of resin used since the resin system dictates much of the performance of the paint (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014).

Bridge owners generally apply a multi-layer paint system rather than a singular coat of paint. To characterize a paint system, the type and level of surface preparation, number of coats of paint, material used for each coat of paint, and thickness and function of each coat of paint need to be identified. Paint systems generally protect steel from corrosion by acting as a barrier between the steel and the exposure environment, i.e., moisture, oxygen, and chloride ions. Specific types of paint systems are designed to provide additional forms of protection as well, including inhibitive coatings and sacrificial coatings.

Historically, bridge owners used "red-lead" alkyd coatings from the 1870s to the mid-1960s or 1970s (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014; Ault & Dolph, 2018; Vinik, et al., 2016). These coatings were inexpensive, in part because they were used with little to no significant surface preparation and could provide protection for up to 20 years when properly installed and maintained (Ault & Dolph, 2018). However, these paints are no longer permitted to be used because they contain high amounts of lead and chromate, which negatively impact human and environmental health. While the nation switched to alternative paint systems for new steel bridges in the 1970s and the Iowa DOT stopped using lead-based paints in the mid-1970s (Iowa DOT, 2020), numerous existing steel bridges with the previous generation of red-lead alkyd coatings remained. For example, as of the late 1990s, the Iowa DOT had less than 35 bridges coated with red lead paint, but the cities and counties across Iowa had over 8,890 steel bridges and most were coated with red lead paint (Nahra, Walton, & Rost, 1999). As a result, there has been a large amount of research in paint systems that can overcoat existing paint systems to avoid the high expenses associated with contained removal of lead-containing paints; these paint systems are outside the scope of this project, which is focused on new construction.

For new steel, the red-lead alkyd coatings were largely replaced with paint systems consisting of abrasive blasting of the steel surface, a zinc-rich primer, and a vinyl topcoat over the zinc-rich primer (Vinik, et al., 2016). Zinc-rich coatings are classified as inorganic or organic and, based on SSPC-Paint 20, *Zinc-Rich Coating*, consist of at least 65 percent zinc by weight in the dried film. Zinc-rich primers are used for bridge steel because they not only provide barrier protection, but also sacrificial galvanic protection at locations where the steel is exposed, for example due to application defects in the coating such as pinholes, or locations of deteriorated or damaged coating. Zinc is less noble than steel and therefore corrodes preferentially to steel, as described in Section 6.2.1.3.4, Galvanic Corrosion. The zinc in the primer is assumed to be electrically connected to the steel such that, in the presence of an electrolyte that has penetrated the barrier protection offered by the coating, the zinc is sacrificed and corrodes instead of the exposed steel. The level of galvanic protection offered by a zinc-rich primer in practice will vary depending on the zinc content, particle size, and purity (Ault & Dolph, 2018).

The topcoat(s) over the zinc-rich primer serve several functions. First, from an engineering perspective, they lengthen the life of the coating by providing additional barrier protection and lowering the risk of pinhole defects in the paint system. The paints selected for topcoats also typically have improved weathering resistance compared to underlying paint materials such that they do not degrade from ultraviolet (UV) exposure as quickly. Second, from an architectural perspective, topcoats can be used to meet aesthetic requirements, such as requirements pertaining to color and gloss retention.

Vinyl paints contain relatively high volatile organic compounds (VOCs) compared to other types of paints, and as such are no longer used because of environmental regulations on VOC emissions; the Iowa DOT stopped using vinyl paints for topcoats in 1993 (Iowa DOT, 2020). Most state DOTs subsequently switched to the use of abrasive blasting and a three-coat system that retained the zinc-rich primer, but used an epoxy intermediate coat and a polyurethane topcoat. Polyurethane paints generally have enhanced weathering resistance compared to other types of paints; for example, many epoxy-based paints experience chalking when exposed to UV light and are not well suited to topcoat applications. However, the epoxy intermediate coat is beneficial because it is more compatible with the zinc primer and consequently facilitates coating adhesion (Ault & Dolph, 2018).

The three-coat system consisting of a zinc-rich primer, epoxy intermediate coat, and urethane-based topcoat is currently the most common paint system for new steel bridges (Vinik, et al., 2016; Ault & Dolph, 2018; Bowman, Hagan, & Hurdle, 2022). An inorganic zinc-rich (IOZ) primer is typically used for shop painting new steel and avoided when conducting painting in the field because IOZ primers are relatively sensitive to surface preparation and curing conditions (Ault & Dolph, 2018). Conversely, organic zinc-rich (OZ) primers are preferred in field applications. OZ primers are sometimes used for shop painting because they allow faster drying times and facilitate more uniform surface thickness, but are not widespread, because they are more expensive than IOZ primers (Bowman, Hagan, & Hurdle, 2022).

The life expectancy of common three-coat systems is 20 to 40 years, depending on installation quality, exposure environment, and maintenance practices (Ault & Dolph, 2018), and multi-coat systems using zinc-rich primers have consistently performed for 25 years or more (Vinik, et al., 2016). In paint systems of good installation quality, corrosion of the substrate is expected to occur eventually because these coatings have some level of permeability. As a result, they can become saturated if exposed to rain or other sources of water and will eventually permit moisture and any aggressive ions present to penetrate through the barrier protection to the metal substrate, at which point corrosion may initiate. The corrosion undercuts the paint system by causing loss of the metal substrate to which the paint system is bonded, resulting in deterioration of not only the metal element but the coating as well.

Paint systems of relatively poor quality often have a relatively high number of “holidays,” which are pinholes in the paint system where a very small area of the substrate is exposed to the environment. Holiday testing is conducted to verify coating quality and identify locations of holidays that should be repaired, but achieving a coating completely free of holidays is challenging. If holidays remain, ambient moisture and ions can reach the metal substrate more quickly at their locations.

6.2.2.2.2 Hot-Dip Galvanizing

There are several methods by which a steel component can be galvanized (Kogler, 2015). However, the primary method applicable to steel bridge elements is hot-dip galvanizing (HDG), which results in a metallic coating of zinc metallurgically bonded to the steel.

HDG is conducted by first cleaning and preparing the surfaces of the steel element, then dipping the element into a bath of molten zinc, and finally inspecting the galvanized piece to check the quality of the coating. The surface preparation typically consists of degreasing the element to remove organic contaminants, e.g., dirt, grease, oil, and paint markings, and then acid pickling the element to remove mill scale and iron oxides. Surface preparation procedures may also include abrasive blasting to help remove inorganic contaminants, such as welding slag or rust. The final step of surface preparation is fluxing, in which the steel element is dipped into a flux solution, i.e., zinc ammonium chloride solution, to remove any remaining oxides and prevent oxidation between the end of the cleaning process and the start of the galvanizing step (AGA, 2012). Proper surface preparation is critical to the success of HDG as the zinc will not bond to unclean steel.

The molten zinc bath used in HDG is required by ASTM A123, *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*, to consist of at least 98 percent pure zinc and is maintained at a temperature of approximately 840 °F (AGA, 2012). The kettles containing the molten zinc are typically around 40 feet long although some galvanizers have kettles up to 60 feet long; this limits the size of the elements that can be hot-dip galvanized (Kogler, 2015). Elements longer than the kettle can be dipped twice, one end at a time, and in cases where the element is still long enough that a gap exists between the galvanizing, metallizing may be conducted to complete the metallic coating, as described in the next section. Upon withdrawal from the bath, excess zinc is removed by draining or vibrating and the element may be air cooled or quenched using either water or a passivation solution (AGA, 2012). Once dipping is complete, the coating is inspected for appearance and thickness.

The thickness of the galvanizing coating depends on a variety of factors, including the chemical composition of the steel, the steel surface condition and any cold working of the steel prior to galvanizing, the bath temperature, immersion time, and withdrawal rate, and the steel cooling rate (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Steel composition is particularly influential. A carbon content less than 0.25 percent, phosphorus content less than 0.04 percent, and manganese content less than 1.35 percent are considered beneficial while a silicon content less than 0.04 percent or between 0.15 and 0.22 percent is considered desirable (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The range of silicon contents considered desirable corresponds to “non-reactive” steels, which galvanize relatively well compared to “reactive” steels, for which the silicon content is between 0.04 and 0.15 percent or above 0.22 percent (Kogler, 2015). While the galvanizing coatings that form on non-reactive and reactive steels provide effective corrosion protection, their features and characteristics differ.

In the case of a non-reactive steel, the coating that forms has four distinct layers: a pure zinc layer on the outermost surface followed by three distinct zinc-iron alloys, which increase in iron content with increasing proximity to the steel base metal, i.e., greater depth from the surface. While the zinc content decreases with increasing depth, all four alloys are predominantly zinc, with zinc contents ranging from 75 to 94 percent (AGA, 2012). The coating that forms generally has a thickness on the order of 4 mils, and this thickness cannot be increased by increasing the immersion or “dwell” time in the tank (Kogler, 2015).

In the case of a reactive steel, the coating that forms is entirely intermetallic, i.e., a zinc-iron alloy, with no pure zinc layer. The coating thickness is generally controlled by the dwell time and can be high, i.e., greater than 10 mils. The coating that forms additionally is at risk of becoming brittle, whereas the pure zinc outer layer that forms on non-reactive steel is relatively ductile (AGA, 2012; Kogler, 2015).

Because coating thickness is subject to multiple case-specific factors, many of which are outside of the control of the designer and the galvanizer, a target coating thickness cannot be specified (Kogler, 2015). ASTM A123 provides minimum thickness requirements that vary depending on the type of element, e.g., structural shapes, plates, pipes and tubing, etc., and the thickness of the steel element. The minimum average coating thickness ranges from 1.4 to 3.9 mils with the high end of this range being applicable for higher steel thickness. The standard notes that the minimum thickness requirements are generally considered obtainable, but certain steel materials may demonstrate marginal coating thickness.

Zinc galvanizing protects steel from corrosion through both barrier protection and cathodic protection. The galvanizing coating initially blocks exposure of the steel to moisture, oxygen, and corrosive ions not only through its physical thickness, but also through the development of a patina. The formation of the gray patina associated with weathered zinc is a multi-step process. Upon initial exposure to atmosphere, zinc oxide (ZnO) forms on the surface of the galvanized element. The zinc oxides convert to zinc hydroxides ($ZnOH_2$) when exposed to moisture, and the zinc hydroxides react with atmospheric carbon dioxide to form zinc carbonate ($ZnCO_3$), the protective corrosion product responsible for the patina. The patina requires approximately 6 to 24 months to form, after which corrosion of the galvanizing coating proceeds at a steady, slowed rate. In most atmospheres, zinc corrodes at least 10 times and up to 30 times slower than steel (Cramer & Covino, Jr., 2003; Revie, 2006). However, wetting and drying cycles are required for the patina to form properly. If moisture persists, then only zinc oxides and hydroxides form, which are powdery and nonadherent and therefore not capable of providing protection from further corrosion.

When the galvanizing coating is consumed such that the base steel metal is exposed, the galvanizing coating provides cathodic protection instead of barrier protection. The time at which the coating is fully consumed and bare steel is exposed will vary across the surface area of the element due to inherent variability in the coating thickness as well as exposure conditions across the surface area. Bare steel will appear first at locations where the coating is relatively thin and/or the local conditions are relatively aggressive, e.g., where chloride-contaminated water splashes onto the galvanized element, or moisture persists due to sheltered conditions and poor air flow. Once the zinc at a particular location is consumed, the remaining zinc coating adjacent to the bare area will provide cathodic protection by corroding preferentially to the exposed steel, at least for a limited time. As the amount of remaining zinc decreases and each area of exposed steel grows larger in size, the effectiveness of the cathodic protection decreases because of the need for the steel area and the zinc to be connected by an electrolyte. The localized corrosion rate of the zinc will also increase as the zinc is consumed because corrosion rates increase as the anode-to-cathode ratio, i.e., the area of the anode, in this case the zinc, over the area of the cathode, in this case the steel, decreases.

HDG can provide protection for 15 to 20 years in aggressive environments, such as marine environments, and, based on measured zinc corrosion rates, is predicted to provide protection for over 100 years in non-aggressive environments, such as rural environments (Ault & Dolph, 2018). Provided that the surface preparation is sufficient, galvanizing coatings also typically have relatively good coverage, particularly at sharp edges and corners where paint coatings tend to be thinner. Galvanizing additionally has a lower initial cost compared to metallizing and typical three-coat paint systems (Cramer & Covino, Jr., 2003; Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). However, the selection of HDG for corrosion protection can increase design costs due to the need to consider design details that facilitate producibility. For example, the designer will need to utilize geometry that will permit drainage of excess zinc during galvanizing and that will not distort unacceptably due to the HDG process (Ault & Dolph, 2018). Additionally, galvanized welds tend to have less fatigue life than ungalvanized welds, although this can be avoided by performing welding after galvanizing.

The first galvanized bridge in the United States was constructed in 1966 to carry Green Street over Stearns Bayou in Ottawa County, Michigan (Main, 1967). One research study reports that a particular galvanizer galvanized 123 state and county bridges in Ohio between 1966 and 1973. In their cost comparison between A1010 steel and other corrosion protection strategies for steel bridges, Sharp et al. (2019) reported the construction of 98 galvanized steel bridges by the Virginia DOT between 2008 and 2019. As of 2018, over 200 steel bridges across the United States were galvanized (Ault & Dolph, 2018), and the American Galvanizers Association (AGA) estimated that approximately 5 percent of steel bridges were galvanized (Collins, 2018). There is no current database of galvanized steel bridges, although this will change with the upcoming implementation of the Specifications for the National Bridge Inventory (SNBI) (2022). SNBI will replace the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges (1995) as the requirements for reporting routine bridge inspection data and is expected to be implemented between 2025 and 2028 (FHWA, 2022). SNBI requires that the “Span Protective System” be identified, wherein steel spans protected by HDG will be identified by the code C03 “Coating – hot dip galvanizing.”

6.2.2.2.3 Metallizing

Metallizing refers to the act of thermal spraying molten metal onto a substrate. The thermal spray process involves feeding material into a spray gun, using a heat source, such as electric arc, to melt the material, and then using compressed air to propel the melted material onto the substrate, where it solidifies upon landing. While the bridge industry generally uses metals when applying thermal spray coatings, hence the use of the term “metallizing,” certain ceramic and polymer coatings can be applied by thermal spray as well (Berndt & Berndt, 2003). Metallizing can be conducted on elements of any size, but requires that all surfaces to be metallized be accessible, and proper thickness may be challenging to achieve at corners, edges, recesses, and cavities. Metallizing is also relatively expensive compared to the other types of coatings; as of 2014, metallizing was estimated to cost 40 to 50 percent more than conventional painting (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014).

The metals predominantly used for metallizing steel bridges are: (1) pure zinc; (2) an 85 percent zinc, 15 percent aluminum alloy (referred to herein as 85/15 Zn/Al); and (3) pure aluminum. The metallized coating may consequently be referred to as “thermally sprayed zinc” (TSZ) or “thermally sprayed aluminum” (TSA). Both zinc and aluminum corrode preferentially to steel such that the metallized coating provides cathodic protection. Several studies completed in the 1950s and 1960s have shown that metallized aluminum coatings provide longer protection than metallized zinc coatings at equivalent thicknesses; however, metallized coatings typically use pure zinc or the 85/15 Zn/Al alloy rather than pure aluminum because zinc is faster and easier to apply than aluminum (Kogler, 2015).

While both galvanizing and metallizing result in metallic coatings, the microstructures of the two types of coatings are distinct in several ways. First, as described in Section 6.2.2.2.2, Hot-Dip Galvanizing, galvanizing results in a metallurgical bond between the coating and the steel substrate due to the formation of zinc-steel alloys. In contrast, the bond between a metallized coating and the steel substrate is purely mechanical and therefore dependent not only on the cleanliness of the substrate at the time of metallizing, but also the surface profile of the substrate, that latter of which is not needed for galvanizing (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Second, metallized coatings are lamellar, with the lamellae parallel to the surface of the substrate and approximately 0.04 to 0.1 mils thick (Berndt & Berndt, 2003). Third, metallized coatings are relatively porous compared to galvanizing, largely because of the stacking of the lamellae although other potential causes of porosity may occur.

The increased porosity of metallized coatings is not desirable because it results in relatively poor barrier protection of the steel substrate and facilitates greater rates of corrosion of the coating. To address this shortcoming, metallized coatings are regularly coated with a sealer that is intended to penetrate and seal the pores. Sealers traditionally consist of a low-viscosity, organic material, such as a low-viscosity epoxy or urethane-based paint, applied as a relatively thin coat compared to the paint systems discussed in Section 6.2.2.2.1, Paint Systems. A typical thickness for a sealer is about 1.5 mils (SSPC, 2011). More recently, more robust sealing systems, such as a sealer and a finish coat, with increased thickness and consequently improved barrier protection have been used. Such systems can be similar to the zinc-rich primer, epoxy intermediate coat, and urethane-based topcoat paint system that is currently preferred by many state DOTs, except the zinc-rich primer is replaced with the metallized coating (Kogler, 2015). In such circumstances, the sealed metallized coating more closely resembles a duplex coating system, in which a full thickness paint system is applied on top of a metallic coating. Because metallized coatings with traditional sealers and metallized coatings with full thickness paint systems are expected to perform differently and the former has a longer history of use such that the bridge and coatings industries have more experience with their performance, this document uses the term “sealed metallizing” exclusively to refer to metallized coatings with traditionally thin sealers. Metallizing sealed with a multi-layer, full thickness coating system is considered to be a duplex system, which is addressed in Section 6.2.2.2.4, Duplex Coating Systems (Ault & Dolph, 2018).

Sealed metallizing is typically used in relatively severe environments, such as marine environments or immersed conditions. In less severe environments, such as rural environments, the thickness of the metallizing may be enough protection such that a sealer is not necessary (Kogler, 2015). Unlike galvanizing, a metallized coating can be built up to meet a specified thickness, and the thickness of thermal spray coatings typically ranges between 2 and 20 mils (Berndt & Berndt, 2003). The thicknesses for metallizing in several atmospheric environments as well as the type of metal and need for a sealer as recommended by Ellor et al. (2004) in NCHRP Report 528, *Thermally Sprayed Metal Coatings to Protect Steel Pilings: Final Report and Guide* are presented in Table 36. In general, recommended thicknesses for metallized coatings have a safety factor applied such that a greater thickness is applied than needed (Kogler, 2015).

Table 36. Recommendations for Metallized Coatings in Atmospheric Environments (Ellor, Young, & Repp, 2004)

Atmospheric Environment	Metal Type	Thickness (mils)	Sealer
Rural	Zinc or zinc-aluminum	6 to 8	No
Industrial	Zinc or zinc-aluminum	12 to 15	Yes
Marine	Aluminum or zinc-aluminum	12 to 15	No

While sealers may not be needed in certain applications, in other circumstances, a sealer may not be permissible. For example, metallized bolted joints cannot be sealed because sealed metallizing does not meet Class B slip requirements (Ault & Dolph, 2018).

Even though metallizing is considered a mature technology, the history of the use of metallized coatings to protect steel bridges is poorly documented. According to Ault and Dolph (2018), metallic coatings, i.e., galvanizing or metallizing, have been used on bridges since the early 1900s while Berndt and Berndt (2003) states that metallizing has been used for infrastructure applications since the 1930s and 1940s. As with galvanized bridges, no database of metallized bridges currently exists, although this will change with the implementation of SNBI, wherein steel spans protected by metallizing will be identified by the code C04 “Coating – metalizing/thermal spray” under Item B.SP.07, Span Protective System.

Further, no comprehensive study of the performance of metallized steel bridges has been performed to date (Kogler, 2015). However, guidance for the service life that should be expected from metallized coatings is available. Based on the definition that service life is the time until 5 to 10 percent of the coated area has broken down and substrate rust is present and referencing ISO 12944-2 classifications, Helsel and Lanterman (2022) provide a relatively low service life estimate ranging from 16 years for an unsealed zinc metallized coating in a C5 environment to 33 years in a C2 environment, and 18 years for a sealed zinc metallized coating in a C5 environment to 35 years in a C2 environment. Other sources indicate that metallized coatings have demonstrated a minimum service life of 20 years in marine and urban environments while a service life of 50 years has been documented in rural environments (Berndt & Berndt, 2003; Kogler, 2015).

6.2.2.2.4 Duplex Coating Systems

Originally, the term “duplex coating” was used to refer to galvanizing with a coat of paint applied on top (Ault, 2023; Van Eijsbergen, 1994). Because galvanizing is not considered porous like metallizing and is capable of providing barrier protection to the underlying steel by itself, the primary benefit of the paint coating, besides architectural reasons, is the barrier protection offered to the galvanizing rather than the steel substrate. Today, the term “duplex coating” may be used to refer to galvanizing or metallizing with a full-thickness coat of paint or a multi-coat paint system on top. When applied to a metallized coating, the paint coating or paint system both seals the pores of the metallizing, thereby enhancing the barrier protection offered by the metallizing to the steel substrate, and offers barrier protection to the metallized coating. However, some organizations still refer to metallizing with either a full-thickness coat of paint or multi-coat paint system on top as sealed metallizing, despite the additional benefit of the more robust paint system (Ault, 2023). In this report, “duplex coating” is used to refer to any protective coating consisting of a coat of paint or a paint system on top of galvanizing or metallizing wherein the overlying paint is substantial enough to provide barrier protection to the metallic undercoat. “Sealed metallizing” is used to refer to metallizing with a paint coating on top that is too thin to provide reliable barrier protection for the metallized coating.

Duplex coatings are expected to offer protection for a much longer time than a metallic coating or paint system by itself. Compared to the zinc-rich primer in a conventional paint system, the metallic coating achieves a greater bond strength with the steel substrate and offers more cathodic protection at locations where the barrier protection of the topcoat(s) is breached (Ault, 2023). Examples of such breached locations include defects in the paint system, particularly holidays, which are pinholes in the coating that form during application and curing; and damaged locations, for example from handling during transport, construction, and erection or from long-term exposure during service. Meanwhile, the paint system lengthens the life of the metallic coating by protecting the metallic coating from general corrosion. Conservatively, the life of a duplex coating may be assumed to be the sum of the lives of the metallic coating and the paint system used. However, the metallic coating and the paint system act synergistically such that the life of the duplex coating is typically expected to be greater than the sum of its components (Van Eijsbergen, 1994).

To estimate the expected service life of a duplex coating, the sum of the service lives of the metallic coating and paint system is multiplied by a "synergy factor." In the context of coatings, "service life" typically refers to the time at which 5 percent of the surface area of the coating has broken down and active rusting of the substrate is present (Van Eijsbergen, 1994; Helsel & Lanterman, 2022). Lower synergy factors are expected in more severe environments, such as industrial or marine environments, and higher synergy factors are expected in less severe environments, such as rural environments. A synergy factor of 1.5 to 2.3 is commonly assumed for duplex coatings based on observed case histories compiled by van Eijsbergen (1994). However, current documentation of the field performance of duplex coatings is insufficient to corroborate this assumption.

In one analytical field performance review, Knudsen et al. (2019) investigated the performance of 61 steel bridges constructed between 1967 and 1995 in Norway with duplex coatings consisting of TSZ topped with four coats of an alkyd-based paint. Table 37 summarizes the bridge ages at which documentation showed the coatings were maintained, referred to as the time-to-maintenance in the table. The results are categorized based on the duplex coating specification in use at the time and the corrosivity of the bridge sites as defined by ISO 12944-2, *Paints and Varnishes-Corrosion Protection of Steel Structures by Protective Paint Systems. Part 2: Classification of Environments*, with C5 being the most corrosive environment and C2 being the least corrosive environment within the study. The time-to-maintenance was as little as approximately 20 years for some bridges, but these undesirably low lives were generally attributed to either low paint film thickness, the presence of holidays in the paint, coating defects due to "spitting" during thermal spraying, or saponification of the alkyd coating when in contact with adjacent concrete. The authors pointed out that many of these issues have been addressed by updated specifications, such as the replacement of alkyd-based paints with epoxy paints, which do not saponify in alkaline environments and for which areas of low thickness are easier to identify during application. Other duplex coatings were approaching 50 years with no signs of corrosion of the steel substrate at the time of the study. For reference, based on the expected lives of multi-coat alkyd paint systems and zinc metalizing from Helsel and Lanterman (2022) and the synergy factors from van Eijsbergen (1994), the expected life of the system would be approximately 40 years in marine environments (corrosivity class of C5) and approximately 120 years in rural environments (corrosivity class of C2). As an important consideration in interpreting the results, the authors noted that the actual time-of-maintenance in practice differed from the time at which maintenance was needed based on the end-of-life threshold of 5 percent area of coating degradation and substrate rusting that is commonly used. In some instances, only the paint system had broken down and no steel substrate corrosion was observed, but preventive maintenance of the duplex coating was performed, resulting in a falsely low datapoint. Conversely, in other instances, rust from corrosion of the steel substrate was visible and maintenance was needed; however, because maintenance had not been completed yet, the bridge age at the time of the study was taken as the "conservative minimum time-to-maintenance" for the bridge, resulting in a falsely high datapoint.

Table 37. Summary of Time-to-Maintenance Data for 61 Duplex Coated Bridges in Norway (Knudsen, Matre, Dorum, & Gagne, 2019)

Corrosivity Class	Built 1967 to 1977[1]			Built 1978 to 1995[2]		
	No. of Bridges:	Time-to-Maintenance (years):	Age of Unmaintained Bridges (years):	No. of Bridges:	Time-to-Maintenance (years):	Age of Unmaintained Bridges (years):
C5	2	25 to 42	n/a	2	19 to 23	n/a
C4	9	21 to 47	45	9	21 to 30	27 to 37[3]
C3	10	28 to 40	43 to 49	13	24	24 to 41[4]
C2	2	41	49	14	n/a	33 to 41

Notes: [1]*The specified duplex coating consisted of one 100-µm thick coat of TSZ, two 50-µm coats of an alkyd paint with zinc chromate, and two 50-µm coats of an alkyd paint.*

[2]*The specified duplex coating consisted of one 100-µm thick coat of TSZ, two 50-µm coats of an alkyd paint with zinc phosphate, and two 50-µm coats of an alkyd paint.*

[3]*Two bridges that had not undergone maintenance at the time of the study exhibited rusting of the steel substrate.*

[4]*One bridge that had not undergone maintenance at the time of the study exhibited rusting of the steel substrate.*

Recently, Ault (2023) completed a study of the performance of duplex coatings on steel bridges within the United States. The study included a field component in which the researchers visited select bridges with duplex coatings across several states, from which the researchers concluded that duplex coatings have provided good corrosion protection for 20 years thus far. Continued monitoring over time will be needed to determine if the duplex coatings meet service life and maintenance expectations. However, Ault (2023) also acknowledged that experiences across the state DOTs varied, with some having good success with duplex coatings and others needing to perform maintenance painting earlier than expected. The technical issues identified as causing short-lived performance of the duplex coatings include poor quality galvanizing or metallizing, contamination or improper cleaning of the surface of the metallic coating prior to application of the paint system, and poor compatibility between the metallic coating and the selected paints.

Overall, these experiences indicate that despite their history of use, duplex coating systems are relatively unfamiliar to state DOTs and immature within the bridge industry. Duplex coating systems have been used in various industries since before World War II and began to undergo more systematic study in the 1950s (Van Eijsbergen, 1994). The study by Knudsen et al. (2019) shows that duplex coating systems have been in use to protect steel bridges in Europe since at least the 1960s while the study by Ault (2023) shows that duplex coating systems have been used for steel bridges in the United States since at least the 1980s in several Northeastern states. However, further investigation into the proper design, specification, and implementation of duplex coatings is needed to decrease the risk of premature failure and the need for maintenance painting prior to 20 years of service.

6.2.2.3 Elimination of Expansion Joints

The deck of a bridge is sometimes likened to the roof of a structure because it protects the underlying elements from precipitation and, in the case of northern states that experience winter climates, deicing chemical exposure, at least from deicing chemicals applied to the roadway on the deck. However, expansion joints are breaks in the protection offered by the deck.

Expansion joints may be classified as “open” or “closed.” Open joints include finger joints and inherently allow chloride-contaminated water to flow through the joint onto any underlying elements. A drainage trough may be included underneath an open joint to catch and direct the water away from the superstructure and substructure elements, or a curtain system may be used to redirect water away from the underlying elements (ElBatanouny, Hawkins, & Krauss, 2021). However, both options require regular maintenance to ensure they are working effectively. Closed joints are sealed to block water and chlorides from passing through the joint and onto the underlying elements, but seals are relatively short-lived and commonly experience deterioration leading to joint leakage. As a result, expansion joints are generally expected to leak and cause accelerated deterioration of the underlying steel or reinforced concrete elements if they are present.

Jointless bridge decks are commonly used to eliminate this potential source of superstructure deterioration as a result. Expansion joints can be eliminated in the following ways:

- **Integral Abutments.** An integral abutment is an abutment wherein the steel or concrete girder ends are embedded in the abutment backwall. This causes the abutment to move with the girders as they expand and contract due to changes in temperature. The expansion joint, which in a non-integral design would be located over the beam ends to accommodate the relative movement of the girder ends with respect to the stationary abutment backwall, is consequently moved behind the abutment when an integral abutment is used. In integral designs, the expansion joint may be directly behind the backwall or located further away from the bridge at the end of an approach slab. At these locations, joint leakage is less of a concern since it will not cause corrosion of the bridge superstructure.
- **Continuous Designs.** Expansion joints at piers may be eliminated by using continuous spans instead of simple spans.
- **Link Slabs.** If continuous spans are not feasible, a continuous reinforced concrete deck may still be installed over a simple span superstructure with the aid of link slabs. Link slabs span girder ends and are pinned to the steel superstructure at their ends but left debonded from the superstructure in their center, as shown in Figure 57. The debonded area permits the girder ends to rotate without damaging the overlying riding surface.

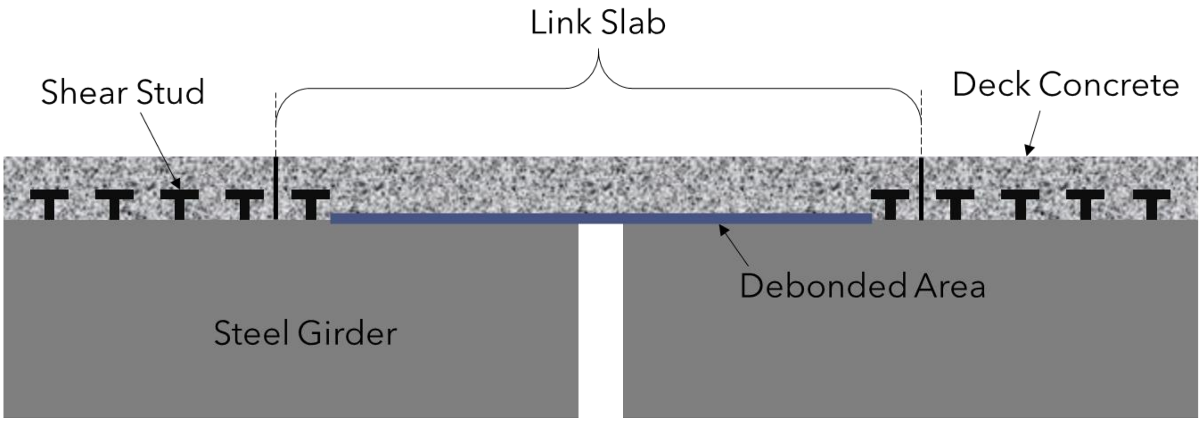

Figure 57. Schematic of a link slab identifying the debonded area between the slab and the underlying steel girders.

Joint elimination is not feasible for every bridge. For example, integral abutments are limited to sites where the subsoils can support deep foundations and the maximum requirements for bridge skew angle, bridge length, and longitudinal grade can be met (ElBatanouny, Hawkins, & Krauss, 2021). However, when joint elimination strategies are feasible and implemented, they effectively avoid accelerated, localized deterioration by avoiding the aggressive exposure conditions caused by joint leakage.

6.2.2.4 Proper Design Details

Proper design detailing of the steel superstructure as well as drainage and joints is needed to avoid the formation of aggressive "microclimates," which are localized areas that experience heightened levels of corrosion compared to the general area of the steel superstructure due to more severe exposure conditions. Desirable details avoid water ponding on the steel elements, divert chloride-contaminated runoff from the deck away from the underlying elements, and minimize the collection of debris, which retains moisture, on the steel. Ault and Dolph (2018) compiled a list of good design practices for mitigating the formation of aggressive microclimates from various sources, including (FHWA, 1989; Crampton, Holloway, & Fraczek, 2013; Kogler, 2015):

- Eliminating details that trap water and/or debris, e.g.:
  - Using narrow splice plates that prevent ponding water at the leading edge of girders,
  - Using coped stiffeners that do not trap water,
  - Minimizing the use of transverse and longitudinal stiffeners, and
  - Eliminating bottom flange lateral bracing,
- Minimizing the number of bridge deck scuppers.
- Installing water diverter plates on bottom flanges.
- Designing drainage paths to facilitate fast water flow and minimize the risk of standing water or clogging.
- Hermetically sealing box members to prevent the intrusion of moisture and air (i.e., oxygen) into the interior, or providing weep holes to allow proper drainage and air circulation and covering or screening openings in boxes that are not sealed.
- Avoiding closely-spaced girders or other superstructure elements.

- Using girders with narrow flange widths.
- Not using welded drip bars where fatigue stresses may be critical.

Guidance for proper detailing also recommends not using welded drip bars where fatigue stresses may be critical. While this practice focuses on mitigating the risk of fatigue rather than corrosion, it is typically acknowledged alongside corrosion mitigation guidance because of the role of drip bars in controlling water flow.

In addition, drain outlets should direct runoff a suitable distance away from superstructure components (and supporting substructures) to prevent splash or contaminant build-up. In circumstances where dissimilar metals are used, such as many bolted connections, designers usually include insulating elements to prevent electrical contact between the metal elements and consequent galvanic corrosion.

In addition to avoiding aggressive microclimates, proper detailing can facilitate the application of quality coatings. Minimizing the complexity of the surface is generally beneficial because it aids in achieving a clean surface, which is a critical aspect of surface preparation for the metallic coatings and paint systems discussed. Minimal complexity also improves the accessibility of the surface, which helps the applicability of paint systems and metallizing (Kogler, 2015). Reducing the number of edges is beneficial for paint systems and metallized coatings (Knudsen, Matre, Dorum, & Gagne, 2019) because these coatings tend to be thinner at sharp edges than the general surface area. The need to consider the impact of the galvanizing process on the integrity of the steel element and design appropriate geometry was acknowledged in Section 6.2.2.2.2 Hot-Dip Galvanizing.

6.2.2.5 Combinations of Protective Strategies

While the protective strategies that can be implemented were presented separately, owners commonly apply multiple corrosion protective strategies on the same bridge. For example, proper design detailing is considered a best practice that should generally be applied regardless of the type of steel and/or protective coating applied, and the elimination of deck joints where possible is typically included in discussions of proper detailing in literature (Ault & Dolph, 2018). The use of weathering steel with partial coatings, e.g., coating only under expansion joints, is a common combination that allows the owner to realize both initial cost savings through the reduction in painted area and long-term cost savings by addressing the poor performance of weathering steel in aggressive microclimates. As acknowledged previously, most protective coatings used on steel bridges are designed to offer sacrificial cathodic protection once the barrier protection of the coating is breached, and both galvanizing and metallizing may be used to protect elements that are too large for galvanizing alone. These examples highlight the fact that the corrosion protection strategies are not mutually exclusive and can instead be leveraged in combination to address the disadvantages or shortcomings of individual strategies, or even to provide longer-lived corrosion protection for steel superstructures.

An exception where combining strategies does not result in synergistic benefits is the use of full protective coatings on weathering steel. This may be implemented due to economics or the availability of weathering versus non-weathering steel, but the coating and the underlying weathering steel are not expected to have a synergistic effect. While some older field exposure testing suggests that paint systems on weathering steel can last longer than on non-weathering steel (Revie, 2006), recent accelerated laboratory testing has indicated that coated weathering steel and coated non-weathering steel are expected to perform similarly (AASHTO, 2020).

6.2.3 Current Standard Iowa DOT Practice for Typical Primary System Highway Bridges

The following documents were reviewed to identify the standard practice of the Iowa DOT pertaining to the use and corrosion protection of steel superstructures:

- *Iowa DOT LRFD Bridge Design Manual*, dated July 11, 2024
- *Iowa DOT Standard Specifications for Highway and Bridge Construction*, Series 2023
- 40' Roadway – 3 Span Rolled Steel Beam Bridge Standards, dated July 30, 2018 with latest revision date of August 2018

To reduce design costs, the Iowa DOT maintains standard superstructure design plans. For typical highway bridges, standard plans are currently available for the following types of superstructures, three of which are concrete superstructures and one of which is steel (Iowa DOT, July 2024):

- Three-span continuous concrete slab (CCS) superstructures;
- Single-span pretensioned prestressed concrete beam (PPCB) superstructures;
- Three-span PPCB superstructures; and
- Three-span rolled steel beam (RSB) superstructures.

The three-span RSB plans are intended for typical stream crossings and county road overpasses, and are an alternate to the standard three-span PPCB superstructure, intended for typical highway and stream crossings. The Iowa DOT currently has limited experience with RSB bridges and as a result requires the designer to lay out equivalent RSB and PPCB bridges for a cost comparison when an RSB bridge is under consideration (Iowa DOT, July 2024).

For county bridges, standard plans are currently available for CCS bridges, single- and three-span PPCB bridges, and concrete box beam bridges; no standard plans for county bridges using steel superstructures have been published (Iowa DOT, n.d.).

When a standard superstructure design cannot be used, the Iowa DOT prefers that a custom-designed PPCB or continuous welded plate girder (CWPG) superstructure be selected. CWPG superstructures are chosen when long spans are required (i.e., greater than 155 feet), minimum superstructure depth is necessary, or the horizontal alignment is sharply curved (Iowa DOT, July 2024).

To address the deterioration of steel superstructures due to corrosion, the Iowa DOT uses weathering steel, incorporates corrosion allowance in the size requirements for the steel sections, and applies protective paint systems. The corrosion protection strategies currently implemented by the Iowa DOT as part of its standard practice are described in more detail in the following sections.

6.2.3.1 Type of Steel

ASTM A709 Grade 50W weathering steel is called for by the standard plans for RSB superstructures. ASTM A709 Grades HPS-50W and HPS-70W are also permitted according to the Iowa DOT LRFD Bridge Design Manual (July 2024). Uncoated weathering steel is selected for CWPG superstructures unless site conditions are unfavorable, as described by Technical Advisory 5140.22 (FHWA, 1989). The Iowa DOT LRFD Bridge

Design Manual (July 2024) particularly cautions designers to consider painted weathering steel when the site is a grade separation and has all of the following characteristics:

- "Vertical clearance is 20 feet or less, because these bridges are more susceptible to 'tunnel-like' conditions
- Bridges over interstates in urban corridors, since deicer treatment in these areas is typically more concentrated
- ADTT = 10% or more under the bridge, since trucks generate more misting with deicers than cars do
- Posted speed limit is 55 mph or greater, since higher speeds generate more misting with deicers".

Under these conditions, the weathering steel superstructure may be fully painted or partially painted, i.e., painted only where the steel experiences the most aggressive exposure and is most vulnerable to corrosion. The Iowa DOT prefers the use of painted weathering steel over painted carbon steel under these circumstances because the initial cost difference between Grade 50 and Grade 50W steel is minimal. Otherwise, when a galvanized or painted steel superstructure is used, Grade 36 or Grade 50 steel is commonly selected (Iowa DOT, July 2024). The scenarios in which painted carbon steel is selected are not identified in the Iowa DOT LRFD Bridge Design Manual.

Various hardware used in steel superstructures, such as bolts, is also required to be weathering steel. The ASTM standard to which different types of hardware are required to adhere and the type or grade of steel called out by the Iowa DOT LRFD Bridge Design Manual are listed in Table 38.

Table 38. Steel Grades Used for Miscellaneous Hardware in Steel Superstructures (Iowa DOT, July 2024)

Element or Hardware Description	Specified Type of Steel
Fill plates, thickness ≤ 3/16 in.	ASTM A606 Type 4 (weathering steel)
Bolts	ASTM A325 Type III (weathering steel)
Nuts	ASTM A563 Grade DH3 (weathering steel)
Washers	ASTM F436 Type III (weathering steel)

To promote the formation of the protective oxide layer of the weathering steel, the Iowa DOT requires that uncoated weathering steel be blasted to SSPC-SP 6/NACE No. 3, Commercial Blast Cleaning, and then misted with water. Water mist is to be applied to the outside surfaces of the fascia girders at least three times, with the surface permitted to dry between each application (Iowa DOT, 2023).

6.2.3.2 Sacrificial Steel Thickness

"Sacrificial steel thickness" refers to the thickness of a steel member that is permitted to be lost to corrosion because the steel member can still provide the required capacity and stiffness at the reduced size. The Iowa DOT requires the designer to include a sacrificial steel thickness of at least 1/16 of an inch in the primary elements, i.e., web and flange plates as well as bolted field splice plates, of uncoated weathering steel superstructures, regardless of bridge location. Secondary elements, such as stiffeners and cross frames, are not required to have sacrificial steel (Iowa DOT, July 2024).

The Iowa DOT clarifies that when selecting a steel plate from the standard sizes offered by a steel mill: (1) the minimum design thickness is determined based on design resistance requirements; (2) the 1/16-inch sacrificial steel thickness is added to the minimum design thickness; and (3) lastly, the smallest plate (or section) size that meets the summed thickness requirement is selected (Iowa DOT, July 2024). This procedure prevents undesirable oversizing that might occur if the plate or section size was selected based on the minimum design thickness alone, and then increased to the next available size for corrosion allowance. By selecting size based on the sum of the minimum design thickness and the specified sacrificial steel thickness, the inherent corrosion allowance caused by the greater size of the available section relative to the minimum design thickness is counted towards the sacrificial steel thickness requirement.

6.2.3.3 Protective Coating Systems

As standard practice, “uncoated” weathering steel bridges are required to be partially coated. All weathering steel near expansion joints, i.e., within a distance of 1.5 times the girder depth, is required to be painted as well as girder ends embedded in concrete for the entire embedment length plus an additional 1.0 foot (Iowa DOT, 2023). The crevice between the embedded steel and the concrete is sealed by caulking the gap with a silicone material. When a median opening is present and is 30 feet wide or less, the fascias of the girders are painted to the limits shown in Figure 5.5.2.4.2 of the Iowa DOT LRFD Bridge Design Manual, reproduced in Figure 58 of this report. The purpose of this paint is to protect the girders from chloride-contaminated snow that is pushed over the bridge railing. Unless otherwise specified, bearing assemblies are also required to be painted, except for galvanized parts.

In addition to the above zones, if the design includes fracture-critical tub or box members, the interiors of the members are required to be painted white. The paint assists in finding fatigue cracking during inspections.

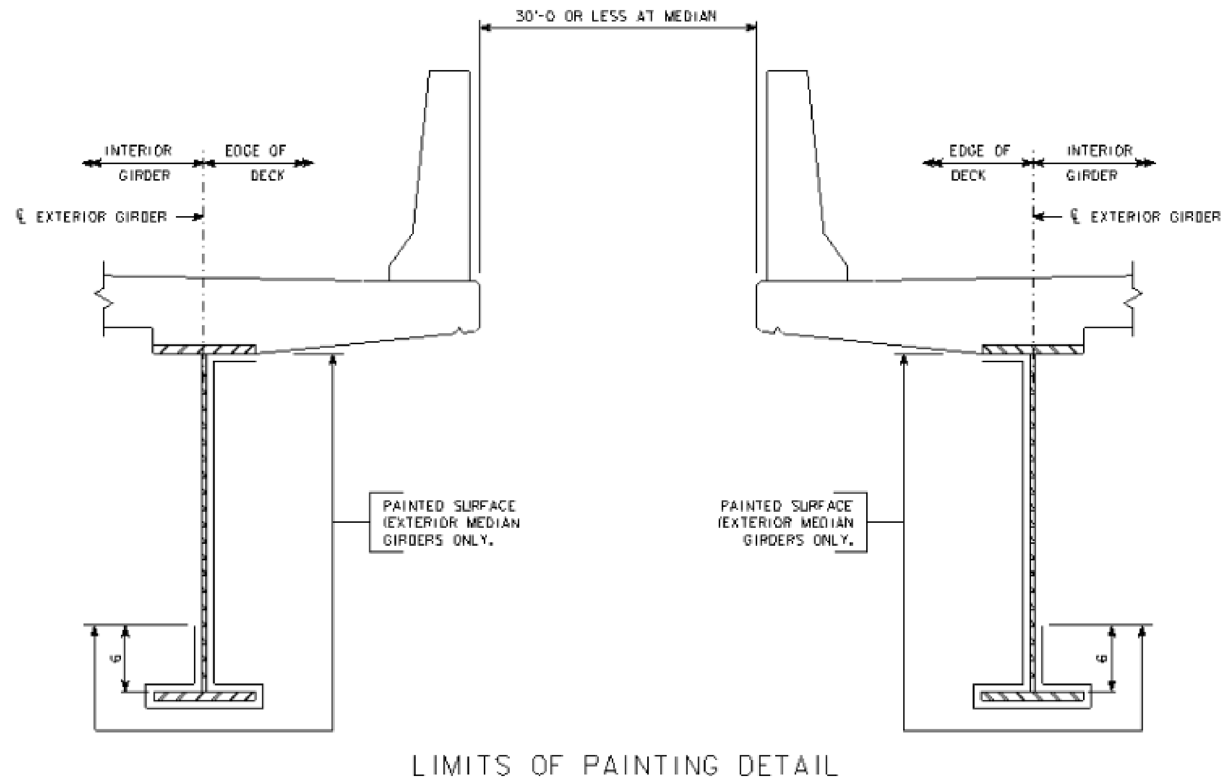

Figure 58. Figure 5.5.2.4.2 of the Iowa DOT LRFD Bridge Design Manual (July 2024) showing the limits of the paint system on the girders adjacent to the median opening when a median opening with a width of up to 30 feet is present.

For all new steel bridge construction, the Iowa DOT primarily uses a two-coat paint system applied on an abrasive blasted surface and consisting of a zinc silicate primer and a waterborne acrylic topcoat. The surface is blasted to a near-white finish per SSPC-SP 10/NACE No. 2, *Near-White Metal Blast Cleaning*, and a sharp, angular profile of 1.5 to 3 mils is required (Iowa DOT, 2023). The zinc silicate primer is shop-applied and is required to be applied within 16 hours of blasting. The target average dry film thickness for the primer is 4 mils, with spot measurements permitted to be 3 to 6 mils. The Iowa DOT maintains a list of approved products for the zinc silicate primer in I.M. 482.02, Appendix A – Structural Paint, Zinc-Silicate. When the zinc silicate primer is damaged due to transportation, handling, or construction in the field, a zinc-rich epoxy paint from the approved products list I.M. 482.02, Appendix C – Structural Paint, Zinc-Rich Epoxy is used to repair the primer in the field. The Iowa DOT Standard Specifications for Highway and Bridge Construction (2023) imply that for weathering steel bridges, a zinc-rich epoxy paint from I.M. 482.02, Appendix C may be used for the prime coat instead of a zinc silicate paint, although the conditions under which the zinc-rich epoxy paint is allowed to be used instead are unclear.

For carbon steel superstructures, a waterborne acrylic topcoat over the prime coat is not explicitly required, but heavily implied to be common practice. The topcoat may be applied either in the shop or in the field and is required to be selected from the approved products list for waterborne acrylic topcoats is maintained in I.M. 482.05, Appendix A – Structural Paint, Water Borne Acrylic. The section specifying the painting of carbon steel superstructures requires that the paints selected for the primer and the topcoat be from the same manufacturer. The waterborne acrylic topcoat is required to have a dry film thickness of at least 2 mils; a maximum dry film thickness for this coat is not specified in the Iowa DOT Standard Specifications for Highway and Bridge Construction (2023).

For weathering steel superstructures, a waterborne acrylic topcoat is required to be applied on top of the primer in the shop unless otherwise permitted by the engineer. Paints from either I.M. 482.05, Appendix A – Structural Paint, Water Borne Acrylic or I.M. 482.07, Appendix A – Structural Paint, Aliphatic Polyurethane are permitted to be used for the topcoat, although polyurethane paints are not discussed elsewhere. The Iowa DOT Standard Specifications for Highway and Bridge Construction (2023) do not provide requirements for the dry film thickness of this topcoat in the context of weathering steel bridges.

The coating system used for fasteners differs from the paint system used for the general steel surface. When a carbon steel superstructure is used, the fasteners are permitted to be galvanized. If galvanized fasteners are used, then no zinc silicate primer is applied to the fasteners, although a zinc-rich epoxy may still be needed for field repair of any damage to the galvanized layer. If the carbon steel superstructure is topcoated with the waterborne acrylic paint in the shop, then galvanized fasteners are to be bolted prior to topcoating. If the fasteners are not galvanized, then they are primed after erection using a zinc-rich epoxy and then field topcoated with the rest of the superstructure, if a topcoat is specified. Requirements for surface preparation of the non-galvanized fasteners in the field are not provided by the standard specifications.

When a weathering steel superstructure is used, the exposed surfaces of fasteners within areas to be painted are prepared for painting after erection. A specific surface preparation standard is not identified, but hand tools, mechanical tools, or blasting equipment may be used, depending on which equipment is most suitable. The fasteners are primed using a zinc-rich epoxy and then topcoated with a waterborne acrylic.

As noted in Section 6.2.3.1, Type of Steel, the Iowa LRFD Bridge Design Manual references the use of galvanized steel bridges in Iowa (July 2024). Galvanizing of rolled, pressed, and forged steel shapes, plate, bars, and strip that is at least 1/8-inch thick is required to be conducted according to ASTM A123, *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products* (Iowa DOT, 2023). However, the circumstances under which a galvanized coating is chosen for a steel superstructure over a paint system are not defined in the Iowa LRFD Bridge Design Manual (July 2024).

6.2.3.4 Joints and Design Details

The Iowa DOT generally follows good design detailing practices for steel superstructures. Integral abutments that avoid joints are used where possible, as described in Section 6.3. Per the standard drawings for rolled steel beam bridges, deck drains are designed to extend at least 1 foot below the bottom of the adjacent steel girder, as shown in Figure 59. Connection plates have coped and clipped corners, as shown in Figure 60. Lastly, flange deflectors are used on the outer sides of the exterior beams; the plan view for the flange deflector is shown in Figure 61.

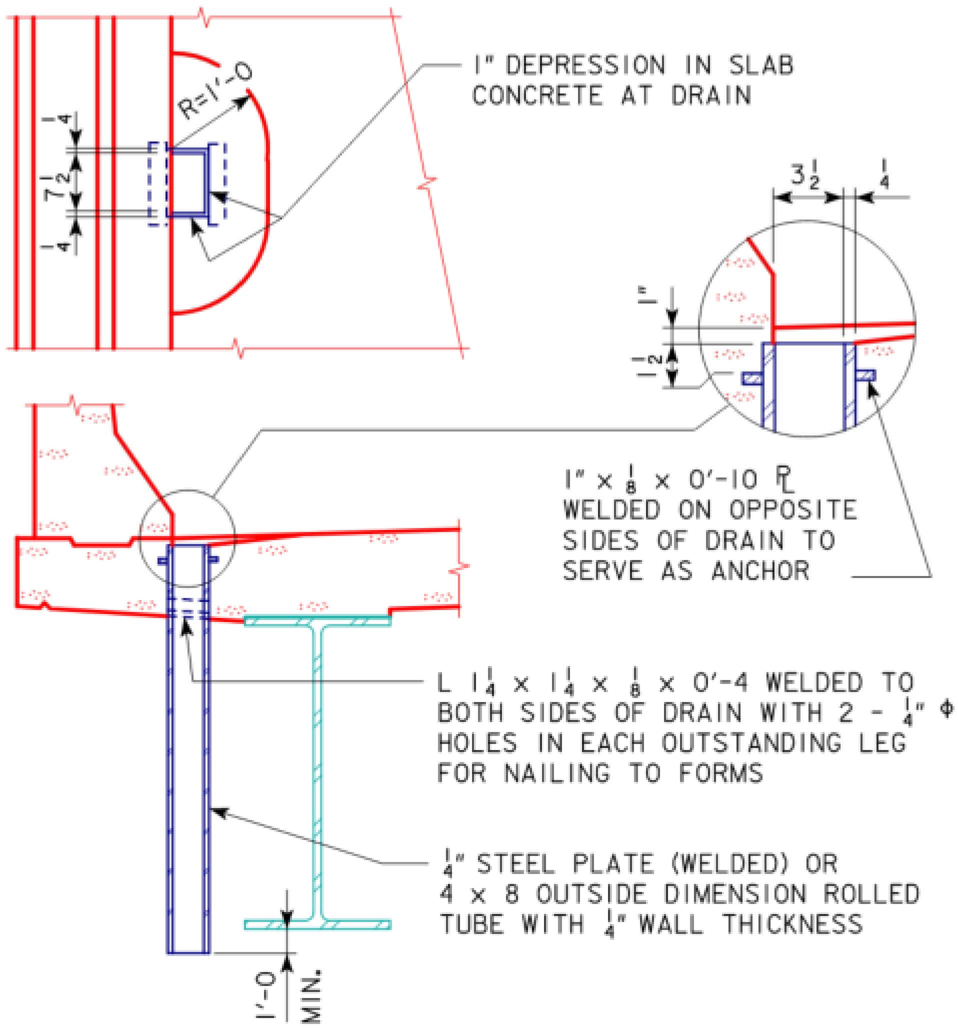

Figure 59. Deck drain design per standard plans by Iowa DOT for rolled steel beam bridges (Iowa DOT, 2018).

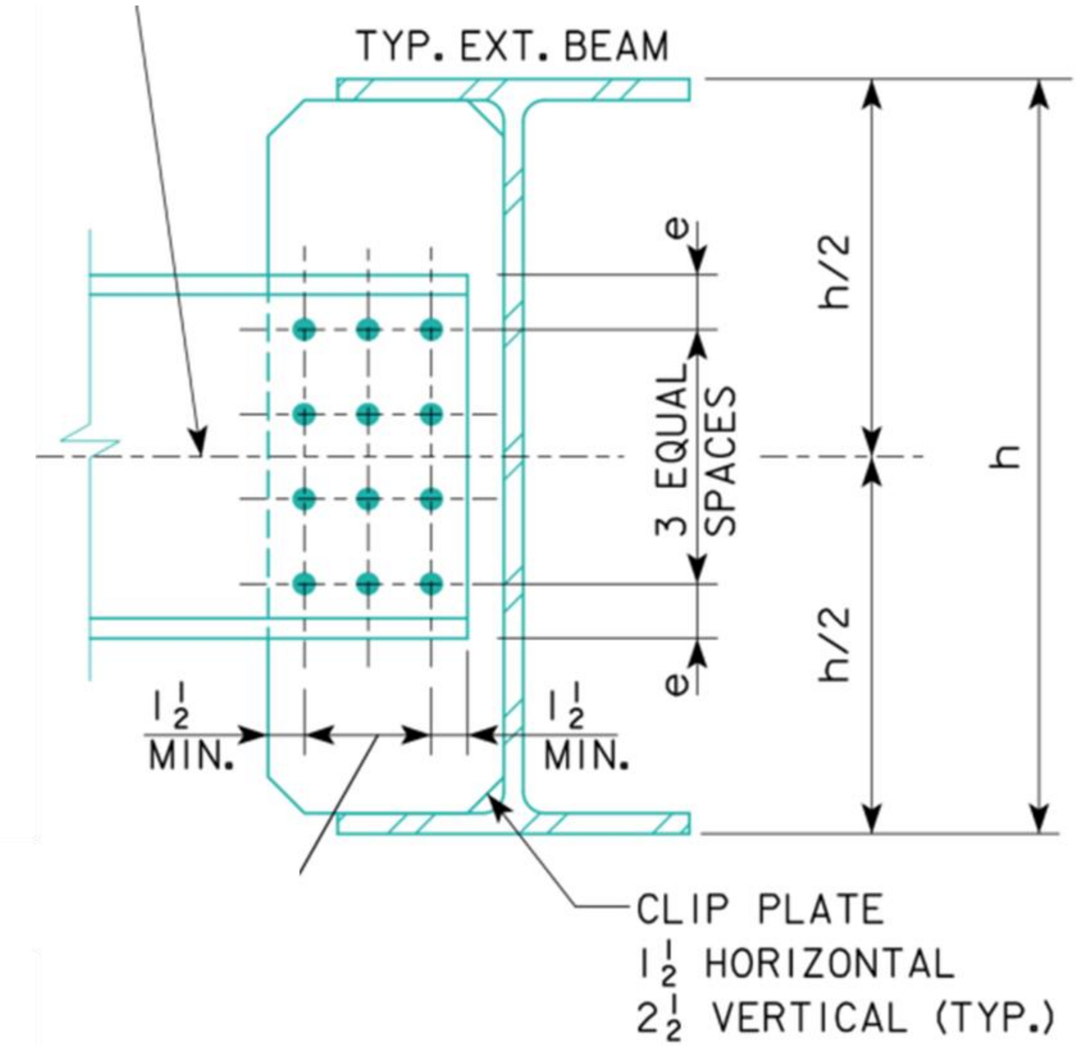

Figure 60. Example of a connection plate with coped or clipped corners per standard plans by Iowa DOT for rolled steel beam bridges (Iowa DOT, 2018).

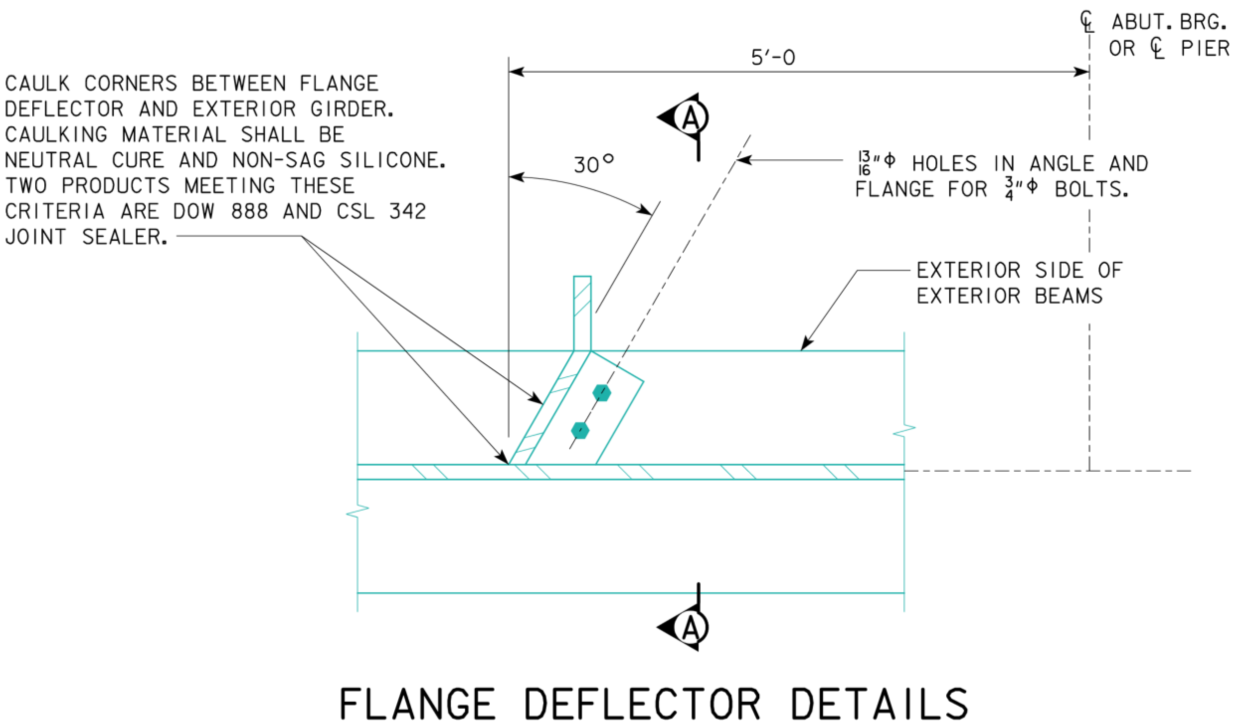

Figure 61. Plan view of flange deflector detail per standard plans by Iowa DOT for rolled steel beam bridges (Iowa DOT, 2018).

6.2.3.5 Steel Bridge Washing Practice

The Iowa DOT generally does not conduct routine pressure washing of steel superstructures except for border bridges, i.e., bridges jointly owned by the Iowa DOT and a neighboring state. For these bridges, regular maintenance washing is conducted to remove surface chlorides, dirt, and debris. However, annual washing is not practical for the entire inventory of steel bridges in Iowa, particularly for bridges over areas of high traffic because of traffic control requirements (Iowa DOT, July 2024).

6.2.4 Expected Performance of Steel Corrosion Protection Strategies in Iowa

The performance of a steel superstructure and its corrosion protection strategies depends on the exposure severity. Performance predictions therefore often identify an expected deterioration rate or life based on the general exposure conditions at the bridge site. However, actual exposure to moisture, chlorides, and other aggressive agents varies across the surface area of the superstructure such that deterioration does not occur at a uniform rate. The presence of construction defects or variable construction quality also contributes to variations in deterioration rate across the superstructure. As discussed previously, good design detailing decreases the risk of locally accelerated deterioration by avoiding aggressive local conditions and/or using more robust protective systems at weak points known to deteriorate relatively quickly. However, the risk of accelerated deterioration at these locations is rarely fully eliminated and they will often require maintenance earlier than the general area of the superstructure. As a result, performance prediction and the selection of a durable design requires an understanding not only of the expected performance of the system under the general site exposure conditions, but also the expected performance of the points of the system that are vulnerable to accelerated deterioration.

This section introduces the exposure conditions that steel superstructures are expected to experience in Iowa and then discusses the expected performance of corrosion-resistant steels and protective coatings in the identified environments. Each discussion considers both general atmospheric exposure conditions and locations where accelerated deterioration typically occurs. Quantitative guidance for performance is usually available for general exposure conditions, but relatively sparse for accelerated deterioration due to local conditions. As a result, discussion of the latter topic is usually qualitative.

6.2.4.1 Exposure Conditions in Iowa

Characterization of the exposure of a bridge requires consideration of both the general site exposure conditions and the potential presence of relatively aggressive conditions across the bridge elements.

6.2.4.1.1 General Site Exposure Conditions

In the context of steel superstructures, variables such as annual average temperature and precipitation and atmospheric concentrations of pollutants like sulfur dioxide are relevant when characterizing general site exposure conditions. The atmospheric corrosivity categories from ISO 9223 that were introduced in Table 34 consider these variables and are well suited for summarizing and categorizing the general exposure conditions of sites. The atmospheric corrosivity categories that are typically expected to apply across Iowa and the general circumstances under which they are assumed to apply are defined for the purposes of this project as:

- **C2 (Low) –** For bridges in rural areas or small towns that are not expected to be exposed to chlorides because they do not cross roadways, therefore avoiding underside chloride exposure, and the deck and barriers protect the superstructure from topside chloride exposure.
- **C3 (Medium) –** For bridges in urban areas regardless of the type of feature crossed and excluding bridges in urban areas that meet the requirements for C4 (High) exposure, and bridges in rural areas or small towns that are expected to experience some effect of chlorides because the feature crossed is a roadway. “Some effect of chlorides” is assumed to occur if a roadway is crossed and the minimum vertical underclearance is greater than 20 feet.
- **C4 (High) –** For bridges in urban and rural areas that are expected to experience a “substantial effect of chlorides” because the bridge crosses a roadway and the minimum vertical underclearance is 20 feet or less. Also for bridges near enough to industrial factories that emissions are expected to affect the air quality of the bridge site.

The definitions of the locations where each corrosivity category is assumed to apply are generally based on the qualitative descriptions from ISO 9223 that were presented in Table 34. However, the qualitative descriptions do not address when chlorides from deicing chemicals are expected to have “some effect” (C3, Medium) versus “substantial effect” (C4, High). Because the deck is generally assumed to protect the superstructure from chlorides from deicing chemicals applied to the deck of the bridge, except as described in Section 6.2.4.1.2, Potential Aggressive Conditions, chloride effects are primarily expected to occur on superstructures because of exposure to chloride-contaminated splash from traffic on underlying roads. In splash exposure, the quantity of chlorides deposited on the superstructure depends on multiple variables, including the vertical underclearance; traffic volume, percentage of truck traffic, and traffic speed on the crossed roadway; and quantity of deicing chemicals applied to the crossed roadway. The severity of chloride splash exposure as a function of these variables, particularly on steel structures, has not been well studied in the United States. Therefore, for simplicity, only the vertical underclearance was used to distinguish between "some effect" and "substantial effect."

A vertical underclearance threshold of 20 feet was selected based on several sources. First, a vertical underclearance of up to 20 feet is one of the requirements currently used by the Iowa DOT to identify relatively aggressive conditions that justify the use of coated weathering steel instead of uncoated weathering steel, as described in Section 6.2.3.1, Type of Steel. Second, a maximum threshold of 20 feet is similar to the maximum threshold of 18 feet identified by McConnell et al. (2024) as one of the characteristics of Inferior Environments 2 and 3, in which weathering steel is expected to exhibit accelerated corrosion, as was discussed in Section 6.2.2.1.1, Weathering Steel. And lastly, a threshold of 20 feet also coincides with the height at which chloride splash exposure for overhead concrete elements becomes relatively low, based on the salting practices of the Iowa DOT and the equation suggested by *fib* Bulletin 34 for modeling splash chloride exposure on concrete elements. *Fib* Bulletin 34 and the referenced equation were introduced previously in Section 6.1. To reiterate here, the equation predicts the surface chloride concentration of concrete elements as a function of the maximum surface chloride concentration at the source roadway, the horizontal distance from the edge of the roadway, and the vertical height above the roadway. When the crossed roadway has a chloride exposure of 6,000 ppm, assumed to represent "High" exposure in Iowa currently, the splash chloride exposure decays with height above and distance from the roadway as shown by the contour map in Figure 62. At a height of 20 feet, the maximum surface chloride concentration is approximately 1,000 ppm, which corresponds to a relatively low chloride exposure. Therefore, a vertical underclearance threshold of 20 feet appears to generally be appropriate for distinguishing between sites with a corrosivity category of C3 (Medium) and C4 (High).

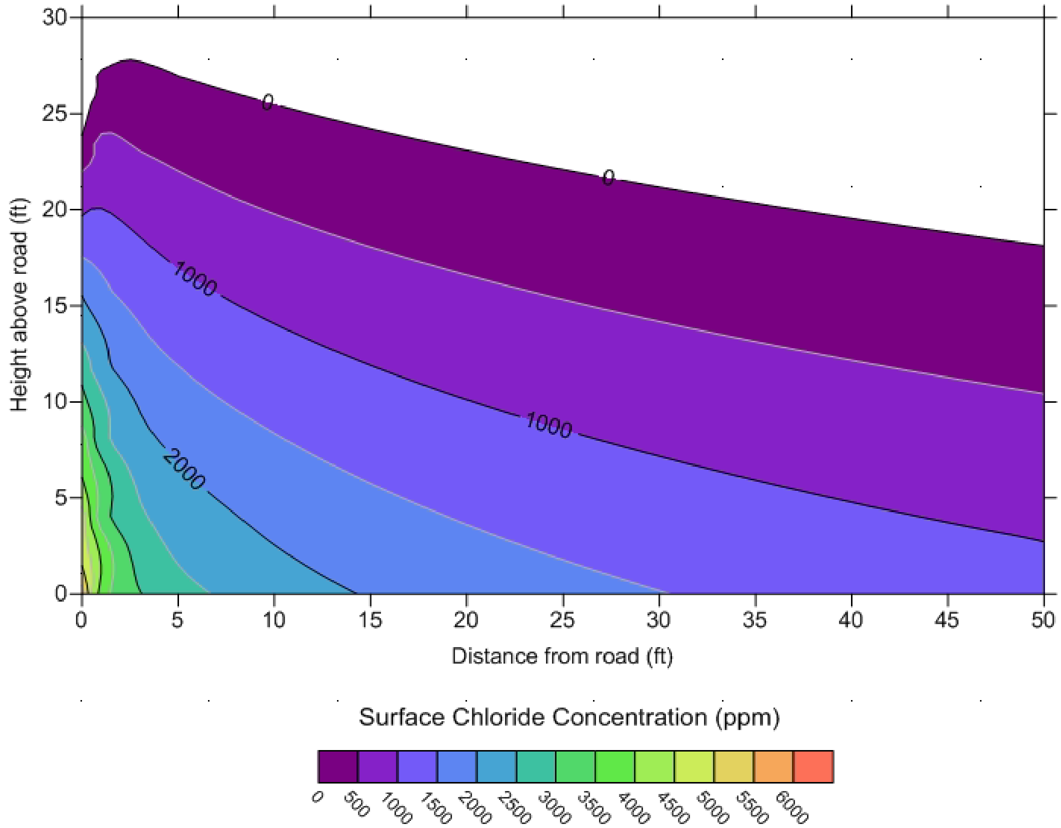

Figure 62. Contour map of the surface chloride concentration experienced by concrete elements near roads per *fib* Bulletin 34, assuming the road experiences a surface chloride concentration of 6,000 ppm.

6.2.4.1.2 Potential Aggressive Conditions

Accelerated deterioration commonly occurs, or is at risk of occurring, at the following locations:

- **Bottom Flanges.** The bottom flanges of steel superstructures are the part of the steel superstructure that is most exposed to contaminated splash from underlying roadways because they are closest to the roadway. Additionally, because of their horizontal orientation, the bottom flanges do not always shed water effectively and are more prone to catching debris than other faces of the superstructure. As a result, the bottom flanges often deteriorate more quickly than the top flanges and webs of the girders.
- **Exterior Faces of Exterior Girders.** The exterior faces of the exterior girders are relatively more exposed compared to the rest of the superstructure. While a deck overhang can help mitigate exposure, exterior faces often experience wind-driven rain and UV radiation, the latter of which is more relevant to the degradation of polymeric coatings. The exterior faces of exterior girders also tend to experience more splash exposure than the webs of interior girders because the bottom flanges and the exterior girders can help block the splash.
- **Sheltered Areas.** While sheltered areas do not experience as much exposure to driving rain and chloride-contaminated splash water as exposed areas, they also do not experience as much air

circulation. As a result, elements within sheltered areas can experience prolonged moisture conditions and greater times of wetness than exposed elements. Elements in the areas directly adjacent to abutments have been known to experience prolonged moisture conditions and consequently accelerated corrosion in the past. This can be mitigated through good design practices, but may not always be fully eliminated.

- **Bolted Connections.** Bolted connections are inherently susceptible to crevice corrosion. Crevices can form between faying surfaces, between bolt heads and the structural member, between the bolt shank and the structural members, and in other locations. Designing tight connections and sealing bolted connections can help minimize the risk of crevice corrosion (Albrecht, Coburn, Wattar, Tinklenberg, & Gallagher, 1989).
- **Leaking Drains.** While it is good practice to design drainage to carry water and chlorides away from the bridge such that the superstructure and substructure elements are not exposed to the effluent, drains are expected to become plugged with debris or deteriorate in other ways such that water leakage eventually occurs.
- **Leaking Joints.** When an expansion joint cannot be avoided, a sealed or "closed" joint is commonly designed to prevent water leakage onto the underlying elements. Unfortunately, seals are relatively short-lived compared to the desired service life of the bridge and the only way to prevent leakage at this time is to proactively replace joint seals before deterioration allowing leakage takes place, which is not often feasible for DOTs. Consequently, joints are always expected to leak.
- **Leaking Deck Cracks.** Through-depth deck cracks in concrete bridge decks that are left unsealed are pathways through which water can drip onto the underlying bridge elements. Leaking cracks can commonly be identified by the presence of efflorescence on the deck underside.
- **Embedded Beam Ends.** While integral abutments, in which the steel beam ends are embedded in the concrete abutment, are advantageous because they avoid joints, the steel immediately adjacent to the concrete is at risk of accelerated corrosion because of a corrosion mechanism that was not introduced in Section 6.2.1.3, Types of Corrosion. The steel within the concrete is subject to passive conditions because of the high pH of the concrete. The exposed steel immediately adjacent to the concrete is not passivated and therefore corrodes preferentially to the passivated steel at the boundary. This corrosion mechanism is the reason why embedded steel beam ends are coated.

6.2.4.2 Performance of Weathering Steel

The primary benefit of using weathering steel is the avoidance of the need for a paint system. For a marginal increase in steel cost, the initial cost of the paint system is avoided, as well as future costs and traffic disruptions associated with maintenance of the paint system. However, this benefit only occurs in environments suitable for weathering steel, i.e., environments where the development of the protective patina occurs. In environments unsuitable for weathering steel, the weathering steel corrodes at a rate similar to that of carbon steel such that painting and its associated costs are necessary regardless of the steel type.

Partially painted weathering steel is advantageous at sites that are generally suited to weathering steel, but have localized conditions along the superstructure where the patina would be unlikely to form or provide effective protection. These systems are designed to minimize the costs of both the paint system and its future maintenance needs, and the costs of future maintenance of the steel elements themselves.

This section first summarizes the environments and sites that are unsuitable for uncoated or partially coated weathering steel. The section then discusses the performance expected of uncoated or partially coated weathering steel at suitable sites within Iowa.

6.2.4.2.1 Sites Unsuitable for Uncoated Weathering Steel

Sites that are unsuitable for uncoated weathering steel are those that have very high exposure to moisture, which will prevent the patina from forming. High levels of chlorides or industrial atmospheric pollutants will also compromise the ability of the weathering steel to develop a patina, although the threshold levels above which the patina will not form vary from site to site depending on the amount of moisture present. For example, weathering steel in relatively arid locations is expected to be able to tolerate greater amounts of chlorides than weathering steel in relatively wet environments.

Specific, quantitative thresholds for environmental variables and site characteristics, or combinations of environmental variables and site characteristics, that make a site unsuitable or marginally suitable for uncoated weathering steel were compiled from FHWA guidance (FHWA, 1989), recent research on the performance of uncoated weathering steel bridges in the United States (McConnell, Shenton III, Bai, & Rupp, 2024), and Iowa DOT experience (Iowa DOT, July 2024). The atmospheric corrosivity categories in ISO 9223 are not referenced in the discussion because they are too general to be useful for distinguishing between suitable and unsuitable environments for weathering steel.

Sites unsuitable for uncoated or partially coated weathering steel bridges are:

- **High Moisture Exposure.** Environments with “Frequent High Rainfall, High Humidity or Persistent Fog (Condensing Conditions)” were identified as unsuitable for uncoated weathering steel bridges by the FHWA in 1989 (FHWA, 1989). The commentary within Technical Advisory 5140.22 elaborated that caution should be used if the time of wetness (ToW) exceeds 60%. The recent work conducted by McConnell et al. (2024) included the development of a database of uncoated weathering steel bridges across the nation and their environmental exposure, including ToW. This database is publicly available through InfoBridge, which is a web portal developed and managed by the Federal Highway Association (FHWA) Long-Term Bridge Performance (LTBP) Program. The locations of uncoated weathering steel bridges identified in Iowa by McConnell et al. (2024) are shown in Figure 63. The greatest ToW experienced by the existing population of uncoated weathering steel across Iowa, which is generally representative of the geographic area across the state, was 46%, based on a 30-year average of annual values as of 2012, and was experienced by the cluster of bridges in the northwest corner of the state, north of Sheldon, IA. Based on the ToW threshold estimated by the FHWA in 1989 and the climate data compiled by McConnell et al. (2024) in 2012, uncoated weathering steel bridges are not expected to be removed from consideration for sites in Iowa on the basis of time of wetness. Technical Advisory 5140.22 also cautions designers when using uncoated weathering steel at sites of “Low-Level Water Crossings,” defined as crossings with an underclearance of 10 feet or less over stagnant, sheltered water or an underclearance of 8 feet or less over moving water (FHWA, 1989). During their review of the performance of the current inventory of uncoated weathering steel bridges, McConnell et al. (2024) noted that the amount of underclearance needed to avoid increased humidity and time of wetness likely depends on the size of the body of water as well, and that uncoated weathering steel bridges with as little as 6 feet of underclearance over water have reportedly demonstrated satisfactory performance in environments with "low potential" for flooding and without excessive humidity. Based on this performance, the researchers recommended that the clearance threshold for moving water be removed. However, they also recommended that quantitative guidance for qualifying environments with a "low potential for flooding" be developed as frequent or long-term flooding can cause excessive moisture conditions and debris build-up, and consequently accelerated corrosion.

Iowa contains some wetlands within its geographic area, where relatively high humidity and time of wetness may be expected compared to the rest of the state because of the size of the body of water, the large amount of vegetation, and the lack of water movement. The climate data compiled by McConnell et al. (2024) indicates that the wetlands do not elevate moisture conditions to a time of wetness that would preclude uncoated weathering steel from use entirely, but greater bridge underclearances may be justified in such areas. Further work would need to be done to identify the extent of these areas and determine the acceptable minimum underclearance for uncoated weathering steel bridges.

Water crossings over creeks, streams, and rivers are common in Iowa and subject to the discussion of low-level crossings over moving water. The majority of the creeks and streams are expected to be small bodies of water such that their impact on relative humidity and time of wetness is also small and a relatively low underclearance is necessary. However, these water features are at risk of flooding, and further work would need to be done in order to comprehensively determine where uncoated weathering steel bridges should be avoided due to flood heights and frequency across the state.

- **High Chloride Exposure.** Technical Advisory 5140.22 advises designers to be cautious when considering uncoated weathering steel for "Marine Coastal Areas" or "Grade Separations in "Tunnel-Like" Conditions" due to the presence of high levels of chlorides (FHWA, 1989). The commentary explains that grade separations consisting of a narrow, depressed roadway with narrow shoulders located between deep abutments or vertical retaining walls and with low vertical clearances experience a "tunnel effect," in which salt spray raised by traffic on the underlying roadway is not dissipated by air currents. As a result, relatively high quantities of chloride-contaminated water are sprayed onto the steel superstructure of the overhead bridge. While the technical advisory provides a qualitative description of when the tunnel effect is expected to occur, it does not provide any quantitative guidance, such as ranges of vertical clearances, horizontal clearances, traffic volumes, and/or truck traffic volumes, that could be used to identify when tunnel effect is a risk.

  To address the need for quantitative guidance, McConnell et al. (2024) analyzed the performance of uncoated weathering steel bridges across the nited States and identified "inferior environments" where uncoated weathering steel bridges had exhibited relatively accelerated deterioration compared to the general population of uncoated weathering steel bridges. Two inferior environments associated with marine exposure and three inferior environments associated with deicing chemical exposure were defined. The latter group, reproduced in Table 35, apply to highway overpass, I-girder bridges and are defined by the vertical underclearance, ADT of the crossed roadway, average annual snowfall, and/or atmospheric chloride concentration:

- **Inferior Environment 1** is characterized by very high traffic volume (an ADT of at least 100,000 vehicles) on the highway under the bridge. This level of traffic is rare in Iowa, but I-235 to the west of Des Moines currently has an ADT of approximately 100,000 vehicles or greater when considering two-way traffic. Very high traffic corridors may exist in other cities, such as Cedar Rapids, Omaha, and Sioux City, as well. Inferior Environment 1 is also defined by an average annual snowfall of at least 18 inches, which is generally applicable across all of Iowa based on the annual average snowfall normal reported by the National Weather Service for 1981 to 2010.
- **Inferior Environment 2** is characterized by a lesser amount of traffic volume (an ADT of at least 10,000 vehicles) on the highway under the bridge, but more aggressive conditions due to less vertical underclearance (no more than 18 feet) and a greater average annual snowfall (at least 22 inches), which indicates greater quantities of deicing chemicals. Figure 64 shows the bridges in Iowa that have an ADT of at least 10,000 vehicles according to current NBI data available on InfoBridge. Even though the reported ADTs are for bridges instead of roadways, this information can be used to infer which routes have an ADT of at least 10,000 vehicles. Based on this information, new bridges in urban areas or new bridges crossing I-35, I-235, or State Route 218 are likely to meet the ADT criterion for Inferior Environment 2. Many of these bridges only carry one directional traffic, and additional routes are likely to meet the ADT criterion if two-way traffic is considered, e.g., if a bridge crosses over both directions of traffic and each direction has an ADT of at least 5,000 vehicles. While select areas in the south of the state may not meet the snowfall criterion, the majority of Iowa has an annual average snowfall of at least 22 inches. The vertical underclearance can be controlled by the designer to a much greater extent than traffic and climate exposure, but NBI data shows that many existing bridges over I-35, I-235, and State Route 218 have a vertical underclearance of 18 feet or less, implying that such underclearances are common practice. Inferior Environment 2 is therefore expected to be relatively common in Iowa.
- **Inferior Environment 3** is similar to Inferior Environment 2, except it has a lesser amount of traffic volume (an ADT of at least 4,000 vehicles) and greater chloride exposure, as indicated by the minimum atmospheric chloride concentration of 0.1 ppm. While atmospheric chlorides are typically discussed in the context of marine environments, the source of the chlorides is inconsequential to qualification for Inferior Environment 3. Based on the climate data compiled by McConnell et al. (2024) as part of the uncoated weathering steel database and accessible through InfoBridge, the atmospheric chloride concentration in Iowa is typically 0.07 or 0.08 ppm, but the same group of bridges identified as experiencing a relatively high ToW of 46% are also experiencing a greater chloride concentration of 0.11 ppm. Therefore, Inferior Environment 3 is not expected to be a typical environment in Iowa but may occur.

While the work by McConnell et al. (2024) is intended to provide quantitative clarification of the 1989 guidelines pertaining to where weathering steel should not be used, the research team did not necessarily recommend prohibiting the use of uncoated weathering steel in the inferior environments identified. Because the inferior environments were likely to be aggressive to any available grade of steel and protective coating, a corrosion allowance of 1/8 of an inch in the bottom flanges of the I-girders was recommended for uncoated weathering steel bridges in these environments instead of the selection of alternative materials. However, the recommended corrosion allowance was based on measured corrosion rates of up to 70 mils within 46 years, which corresponds to approximately 1.5

mils per year on average and indicates failure of the weathering steel to form a protective patina as discussed in Section 6.2.4.2.2, Corrosion Rates of Weathering Steel in Suitable Sites. Lastly, the Iowa DOT has also identified a set of site criteria under which the tunnel effect is too severe for uncoated weathering steel to be considered. The criteria have some similarities to the inferior environment criteria identified by McConnell et al. (2024); the bridges must cross over interstates, although in urban corridors, and have a vertical underclearance of 20 feet or less. However, the sites that are unsuitable for uncoated weathering steel per current Iowa DOT policy must also have an ADTT of at least 10% on the interstate under the bridge and the posted speed limit must be at least 55 miles per hour, which are variables that were not identified by McConnell et al. (2024). The Iowa DOT does not require that a fully painted steel structure be used when a site fulfils the identified list of requirements, but requires a paint system to be applied to vulnerable surfaces of the weathering steel superstructure at a minimum, thereby increasing the extent of the partial painting compared to standard practice.

- **Exposure to Industrial Pollutants.** Technical Advisory 5140.22 also advises caution when using uncoated weathering steel in “Industrial Areas where concentrated chemical fumes may drift directly onto the structure.” Atmospheric concentrations of industrial pollutants have decreased substantially across the United States since the 1970s because of federal regulations such as the Clean Air Act of 1970. As a result, industrial pollutants and their impact on the corrosion rates of exposed metals are generally considered negligible today. However, there may still be unique bridge sites where industrial pollutants will cause accelerated corrosion, such as a site located immediately downwind of a factory. While such sites are not expected to be common in Iowa, they are expected to occur infrequently.

In summary, multiple bridge sites across Iowa may not be suitable for weathering steel for a variety of reasons. The most common reason is expected to be classification of the site under Inferior Environment 2, as defined by McConnell et al. (2024). A smaller number of sites are expected to be unsuitable because they classify under Inferior Environment 1, as defined by McConnell et al. (2024). The number of sites that classify under Inferior Environments 1 and 2 is expected to grow with time as traffic volumes increase. Additionally, bridge sites may be deemed unsuitable for uncoated weathering steel because of frequent or long-term flooding and/or the inability to provide enough clearance to avoid low-level water crossings; further work is needed to determine the extent of the geographic area where flooding is expected to influence the performance of uncoated weathering steel. Lastly, bridge sites within a relatively small area in the northwest corner of the state may classify under Inferior Environment 3, as defined by McConnell et al. (2024), because of elevated atmospheric chloride levels compared to the rest of the state.

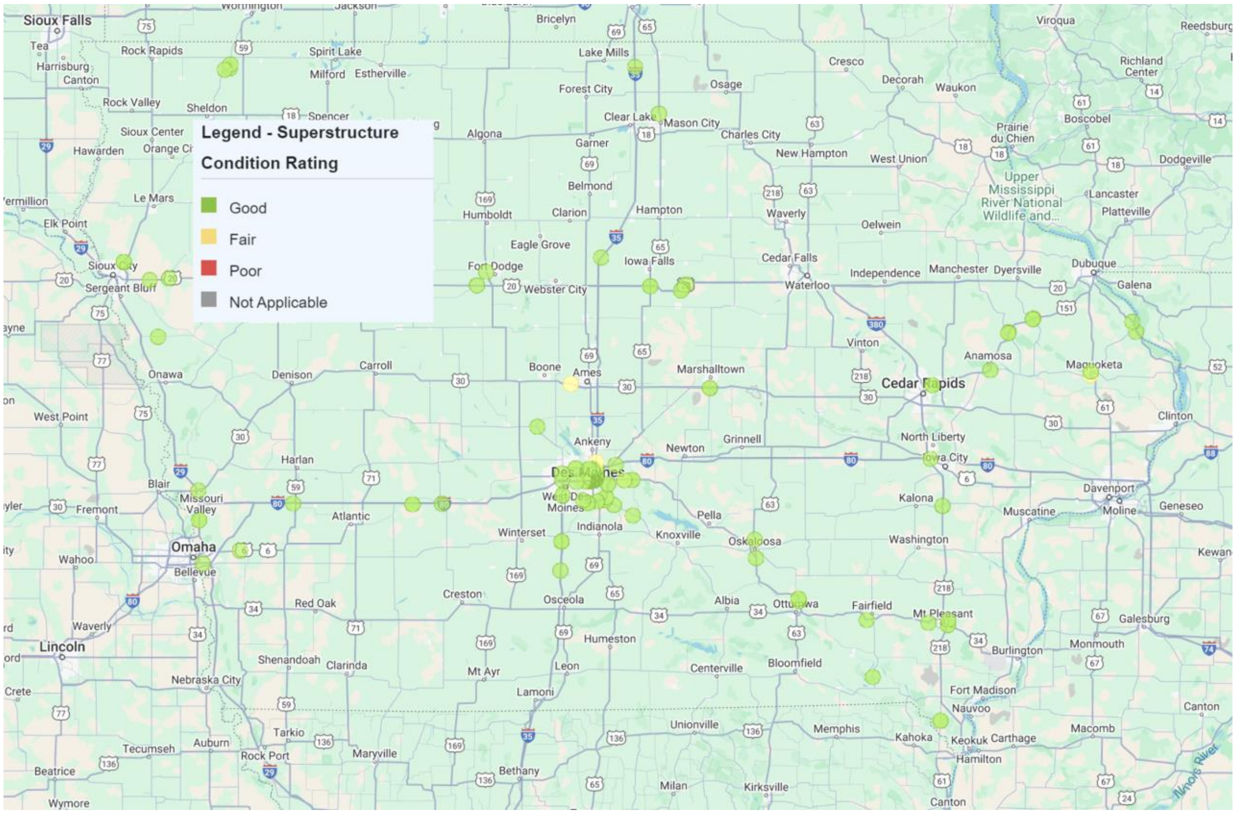

Figure 63. Locations of uncoated weathering steel bridges in Iowa, based on the unocated weathering steel database developed by McConnell et al. (2024) and retained in InfoBridge. The bridges are color-coded according to the superstructure condition rating with green indicating “good” condition and yellow indicating “fair” condition. No bridges are in “poor” condition, represented by red.

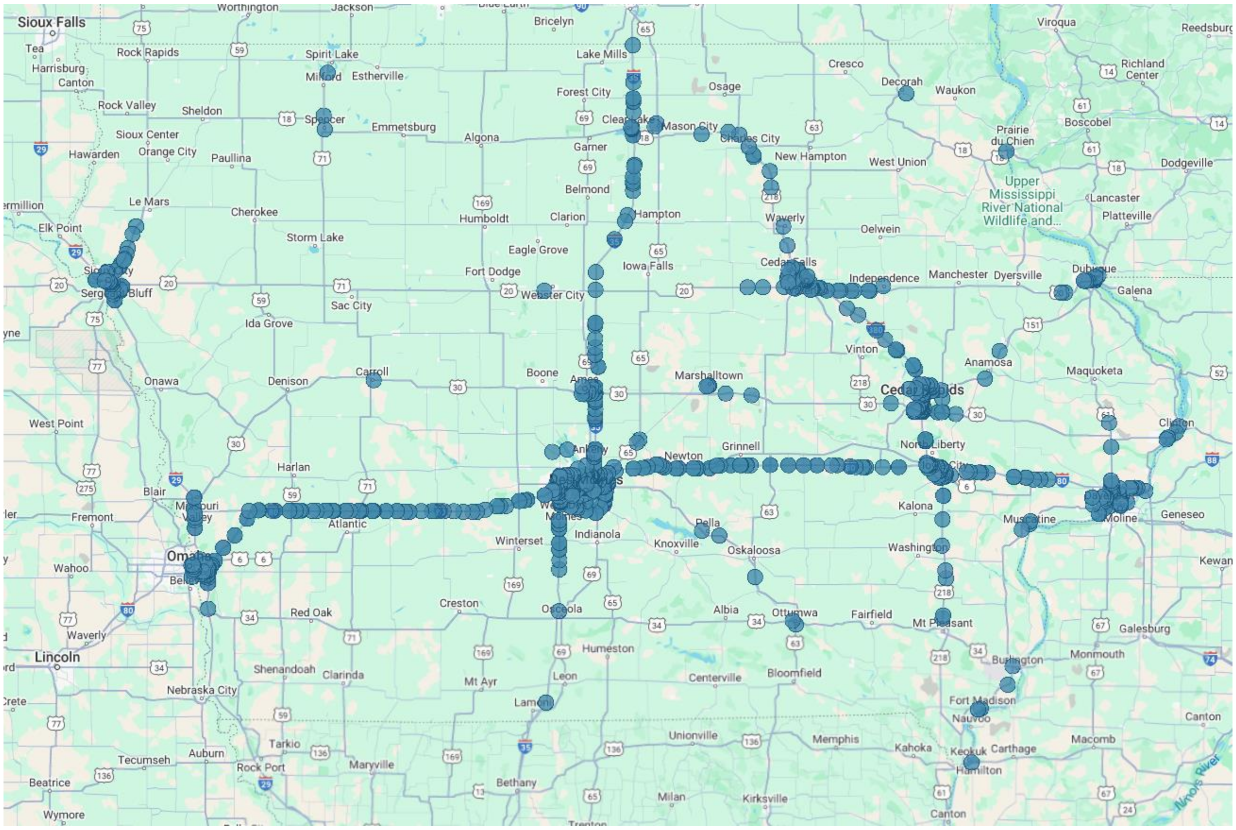

Figure 64. Bridges in Iowa that have an ADT of at least 10,000 vehicles based on 2024 NBI data within InfoBridge.

6.2.4.2.2 *Corrosion Rates of Weathering Steel in Suitable Sites*

Even though multiple bridge sites in Iowa are expected to be unsuitable for uncoated weathering steel, uncoated weathering steel is still expected to be able to develop an effective patina and be an effective corrosion protection strategy at many other bridge sites in Iowa. Based on research conducted in the 1980s, an uncoated weathering steel surface with a protective patina, i.e., located at a “suitable” site, is expected to corrode at a rate between 0.12 and 0.3 mils per year. Higher corrosion rates indicate that the protective patina will not form (Albrecht, Coburn, Wattar, Tinklenberg, & Gallagher, 1989).

These corrosion rates align with the values published by ISO in 1988 in a draft proposal for the first version of standard ISO 9224, *Corrosion of metals and alloys – Corrosivity of atmospheres – Guiding values for the corrosivity categories*. The annexes of ISO 9224 provide expected long-term corrosion rates for standard metals, e.g., carbon steel and zinc, when exposed to atmospheres of the different corrosivity categories identified in ISO 9223. The 2012 version of ISO 9224 does not provide rates for weathering steel, but according to Albrecht et al. (1989), the 1988 draft proposal expected weathering steel to experience steady-state corrosion rates after development of the patina of:

- 0.004 to 0.04 mils per year in atmospheres of “Low” corrosivity,
- 0.04 to 0.2 mils per year in atmospheres of “Medium” corrosivity, and
- 0.2 to 0.4 mils per year in atmospheres of “High” corrosivity.

The lower and upper bounds of the range of corrosion rates provided by Albrecht et al. (1989) are the median steady-state corrosion rates for the “Medium” and “High” corrosivity categories, respectively. While rural areas are expected to have atmospheres with “Low” corrosivity based on the 2012 version of ISO 9223, corrosion rate data compiled by Albrecht et al. (1989) showed that weathering steel in rural Pennsylvania could experience steady-state corrosion rates of up to 0.4 mils per year, which is instead in alignment with the “High” category.

On the one hand, the weathering steel corrosion data compiled by Albrecht et al. (1989) from exposure sites in the United States justifies the assumption of “Medium” to “High” corrosivity of atmospheres in the United States. On the other hand, the corrosion data has limited applicability to modern weathering steel bridges in the United States and particularly in Iowa. Improvements in air quality and steel composition since the 1970s mean that corrosion rates for modern weathering steel bridges are expected to be lower than corrosion rates measured in the 1970s and 1980s. The rural corrosion rate data compiled by Albrecht et al. (1989) is also specific to sites in Pennsylvania, which is expected to have had relatively poor air quality at the time compared to rural sites in Iowa today because of the industrial history of the Mid-Atlantic region. Additionally, Albrecht et al. (1989) cautioned readers that much corrosion rate data was based on exposure testing of small weathering steel coupons, for which “bold” exposure to sun radiation and weather was, and still is, specified. The relatively large weathering steel elements used in bridge superstructures are expected to corrode at different rates because of their different thermal behavior and different exposure conditions, such as relatively sheltered conditions or exposure to deicing chemicals. However, since the 1980s, there has been limited research in weathering steel corrosion rates. This topic was identified as a research need to NCHRP in 2017, but was not selected for funding. As a result, the range of expected corrosion rates provided by Albrecht et al. (1989) is still commonly assumed.

As acknowledged earlier, the rates offered by Albrecht et al. (1989) are steady-state corrosion rates. Within the first few years of corrosion, weathering steels generally corrode at rates similar to those of carbon steels and this should be accounted for in corrosion prediction models. Albrecht et al. (1989) referenced the models shown in Equations 1 and 2 for predicting upper and lower bounds, respectively, for corrosion loss (C, in mils) of a weathering steel surface as a function of time of exposure (t, in years):

$$C = 2.0 + 0.3(t - 1) \qquad \text{Equation 7}$$

$$C = 1.0 + 0.12(t - 1) \qquad \text{Equation 8}$$

According to these equations, weathering steel is estimated to experience 1 to 2 mils of corrosion in the first year before experiencing steady-state corrosion at rates of 0.12 to 0.3 mils per year. These estimates are consistent with the current version of ISO 9223, which defines atmosphers of “Medium” and “High” corrosivity as atmospheres in which carbon steel experiences 1.0 to 2.0 mils and 2.0 to 3.1 mils of corrosion in the first year of exposure, respectively. This current report subsequently assumes that corrosion loss of uncoated weathering steel located at sites in Iowa where the protective patina will form will fall between the upper bound presented in Equation 3 below and the lower bound that was presented in Equation 2:

$$C = 3.1 + 0.3(t - 1) \qquad \text{Equation 9}$$

The above discussion established the expected rate of metal loss. In order to predict expected life, the acceptable amount of metal loss needs to be determined. The “true” amount of metal that can be lost without compromising the ability of the superstructure to provide the required capacity depends on the demand-to-capacity ratio (DCR), governing failure limit state, location of the metal loss on the superstructure, design sacrificial steel thickness, and amount of unintended corrosion allowance that occurs due to selection of the nearest available section of larger size. As a result, detailed structural analyses are needed to understand the true amount of metal loss that can be accommodated across the surfaces of a steel superstructure. In lieu of a bridge-specific analysis, the design sacrificial steel thickness is commonly and conservatively assumed to be the threshold of allowable metal loss across the entire surface area of the superstructure.

As identified in Section 6.2.3.2, Sacrificial Steel Thickness, the girders of steel superstructures in Iowa have a design sacrificial steel thickness of 1/16 of an inch. Assuming two-sided corrosion of an I-section, each surface has a design sacrificial steel thickness of 1/32 of an inch, or approximately 31 mils. Based on Equations 2 and 3, the design sacrificial steel thickness is expected to be consumed after 95 to 253 years of exposure.

Secondary members of the steel superstructure, such as cross frames and stiffeners, do not have a design sacrificial steel thickness. A design sacrificial steel thickness for these types of elements was likely deemed unnecessary because: (1) their replacement is relatively easy, inexpensive, and non-disruptive because it does not require the bridge to be closed the way repair or replacement of the girders would, although it could be disruptive to traffic on an underlying roadway; and (2) the amount of corrosion allowance due to selection of the next-largest available section is expected to be relatively high. However, because a design sacrificial steel thickness is not specified, the assumption that the threshold of allowable metal loss equals the design sacrificial steel thickness cannot be applied as it was for the primary members of the superstructure. Further, the corrosion allowance permitted by the selection of the next-largest available section cannot be reliably estimated, and as a result, a prediction of the time at which the threshold of allowabl metal loss is exceed cannot be developed for the secondary members of steel superstructures in Iowa.

6.2.4.2.3 Risk of Locally Accelerated Corrosion Based on Performance of Iowa Bridge Inventory

Even at suitable sites, a poor patina and accelerated corrosion rates are expected at locations along the superstructure with relatively aggressive conditions, as discussed in Section 6.2.4.1.2, Potential Aggressive Conditions. Accelerated corrosion rates associated with these highly localized conditions are extremely variable, and their prediction is subject to very high uncertainty such that estimating the time at which maintenance is needed and/or structural capacity is insufficient is challenging. The Iowa DOT avoids accelerated corrosion in several key locations known for having aggressive conditions by painting the steel elements at the following locations or on the following surfaces:

- Under expansion joints, which are expected to leak;
- Beam ends embedded in concrete integral abutments;
- Beam length for a distance from the abutment, where there is a greater risk of sheltered conditions and high moisture exposure; and
- The exterior faces of girders adjacent to median openings up to 30 feet in width, which are expected to experience high quantities of chloride-contaminated spray.

The expected life of the areas of painted weathering steel is addressed in Section 6.2.4.3, Performance of Paint Systems.

Common or potential locations of accelerated corrosion that do not receive a protective coating when following the standard practices of the Iowa DOT are:

- Bottom flanges;
- Exterior faces of exterior girders;
- Bolted connections;
- Areas adjacent to drains; and
- The top faces of elements that may experience water leakage from deck cracks.

Instead of estimating accelerated corrosion rates at these local areas based on values reported in literature, the performance of the existing inventory of uncoated or partially coated weathering steel bridges across Iowa was evaluated to determine whether or not accelerated corrosion at these locations has caused inspectors to report maintenance needs.

The existing inventory was identified based on the uncoated weathering steel bridge inventory developed by McConnell et al. (2024) circa 2012. The histogram in Figure 65 shows the range of ages of uncoated or partially coated weathering steel bridges in Iowa. As a caveat to this analysis, the majority of the 133 uncoated or partially coated weathering steel bridges in Iowa are between 10 and 30 years of age, which is younger than the desired 50-year period of minimum maintenance. However, this is old enough for the patina to have developed and steady-state corrosion to be achieved such that any issues associated with poor patina formation would be identified.

Three of the bridges in the existing inventory of uncoated or partially coated weathering steel bridges in Iowa are relatively old and were constructed between 1970 and 1972, all prior to the FHWA TA 5140.22 (1989) and prior to the pause in the use of uncoated weathering steel bridges in Iowa. These bridges do not appear to have had their superstructures replaced since their construction and as such were kept in the analysis. The inventory developed by McConnell et al. (2024) identified three additional uncoated weathering steel bridges in Iowa, but these were removed from the analysis. One of the additional bridges was constructed in 1984, after the Iowa DOT had paused its use of uncoated weathering steel bridges, and the other two additional bridges were constructed in 1963, prior to the use of weathering steel superstructures in the United States. Further datamining determined that the original bridges had coated steel superstructures, but were widened with weathering steel girders in later years. Because the superstructures are not predominantly uncoated or partially coated weathering steel and because of the complicated history of the superstructures, these three bridges were not included in this analysis.

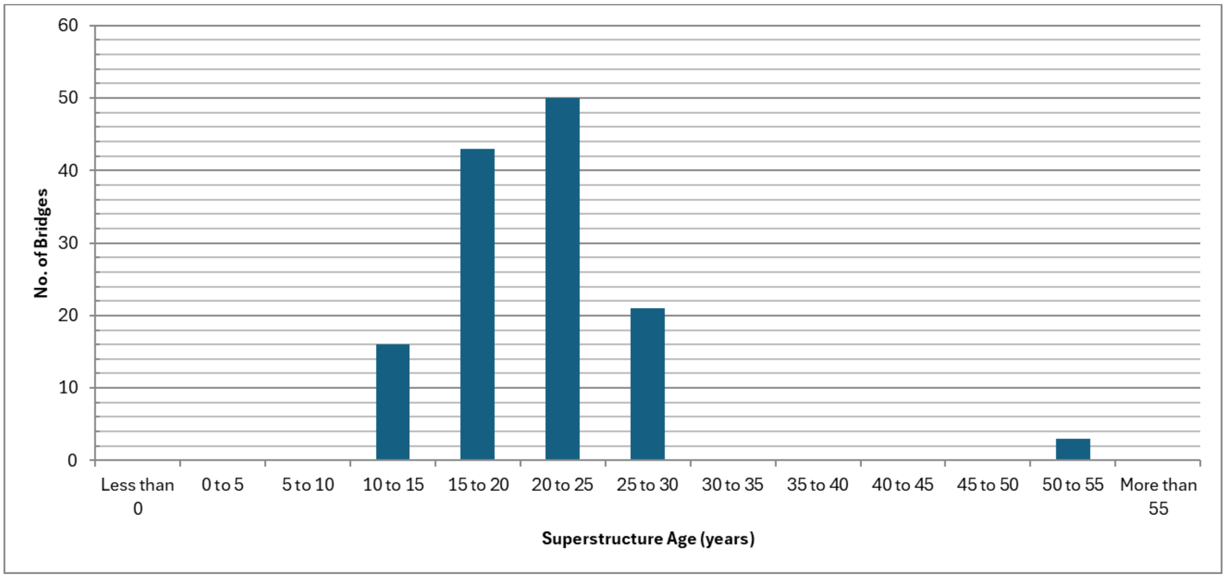

Figure 65. Histogram of the age of uncoated or partially coated weathering steel bridges in Iowa as of 2024.

As of 2024, only three of the 133 uncoated or partially coated weathering steel bridges in Iowa have a superstructure NBI condition rating of "6 – Satisfactory," defined as "widespread minor or isolated moderate defects" by the Specifications for the National Bridge Inventory (2022), referred to as SNBI. The remaining 130 bridges have superstructure NBI condition ratings greater than 6, which correspond to better conditions (as defined in SNBI (2022)), The three bridges with superstructure NBI condition ratings of 6 are:

- FHWA No. 609280, which is 21 years of age;
- FHWA No. 606750, which is 27 years of age; and
- FHWA No. 015225, which is 52 years of age.

Based on a review of the past inspection records and maintenance histories of these bridges, steel corrosion appears to be contributing to the relatively low condition rating only for No. 606750. The superstructure condition rating history for No. 606750, as reported by InfoBridge, is shown in Figure 66. This bridge was constructed in 1997 and carries W Summit St, classified as an urban minor arterial route, over State Route 61 in Maquoketa, Iowa. The average daily traffic of US-61 is approximately 3,550 vehicles per direction such that the ADT of the feature crossed by No. 606750 is 7,100 vehicles. The average daily truck traffic of US-61 at this location is 22%, the posted speed limit of US-61 is 65 mph, and the minimum vertical underclearance of the bridge is 16.6 ft. The ToW at the site is 30% and the atmospheric chloride concentration is 0.07 ppm. This site is generally open, as shown by the elevation view in Figure 67.

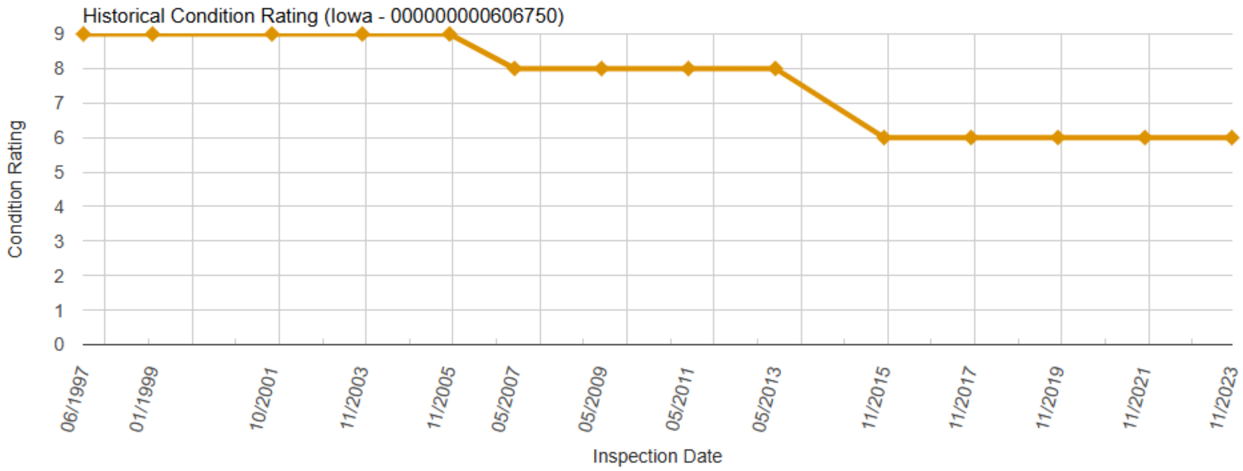

Figure 66. Superstructure condition rating of No. 606750 since the year the bridge was constructed (1997), from InfoBridge.

Figure 67. Elevation view of No. 606750 as of August 2024 facing north, from Google Maps.

Element-level inspection data from 2015, the year in which the general condition rating of the superstructure decreased to 6, show that the entire superstructure of the bridge was classified as belonging to Condition States 2 or 3 due to the presence of corrosion. Similarly, 56% of the quantity for the weathering steel protective coating (Element 851) was classified as Condition State 3 and 44% was classified as Condition State 2. Photographs from visual inspections show that the bottom flange has a poor patina (Figure 68) and tape tests on the beam webs demonstrated that the patina is not providing protection (Figure 69). This performance can be explained by the low clearance of the bridge over a principal arterial route with relatively high traffic and truck traffic volumes and a high posted speed limit in an urban environment, even though the site does not meet the criteria for an inferior deicing chemical environment, as defined by McConnell et al. (2024), nor the set of site requirements set by the Iowa DOT to qualify the bridge for a more extensive paint system. Additionally, areas adjacent to the beam ends embedded in the concrete abutment are experiencing corrosion (Figure 70) and paint peeling (Figure 71). The relatively sheltered conditions at the abutments are likely contributing to this observed deterioration.

Figure 68. Photograph of the typical appearance of the floor system of No. 606750, dated December 12, 2023, from NBI inspection records.

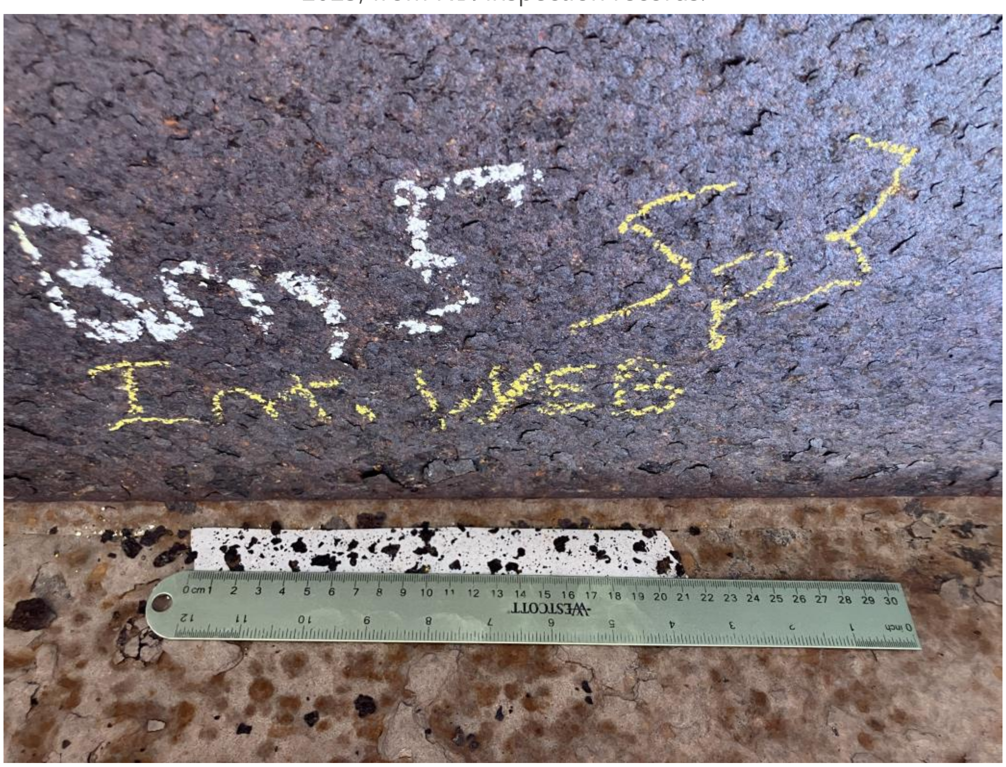

Figure 69. Photograph of a tape test conducted on the web of an interior beam belonging to No. 606750, dated December 12, 2023, from NBI inspection records.

Figure 70. Photograph of rust products implying corrosion at an abutment belonging to No. 606750, dated May 6, 2015, from NBI inspection records.

Figure 71. Photograph of peeling paint adjacent to an abutment belonging to No. 606750, dated May 6, 2015, from NBI inspection records.

For No. 609280, inspection records indicate that the weathering steel patina is relatively poor, but the beams were not experiencing any concerning corrosion as of 2020, the year in which the general condition rating of the superstructure decreased to 6. The superstructure condition rating history for No. 609280, as reported by InfoBridge, is shown in Figure 72. Visibly poor patina quality on the bottom flanges of the superstructure was noted in 2010 (Figure 73). The element-level data from the 2018 inspection show that the entire weathering steel protective coating (Element 851) was in Condition State 2 or Condition State 3, but the steel girders were reportedly in Condition State 1, with no corrosion defects reported. The 2018 inspector noted that the beams crossing over traffic had "some loose rust on bottom and top of bottom flanges only." Element-level condition data did not change between the 2018 and 2020 inspections, indicating that while the poor patina on the bottom flanges of the girders may be contributing to the general condition rating of 6, corrosion is generally not driving the decrease.

No. 609280 was constructed in 2003 and carries SE Corporate Woods Dr, classified as an urban minor arterial route, over US Interstate 35 in Des Moines, Iowa. The site is also located next to Ankeny Regional Airport. Nearby bridges that carry I-35 indicate that each traffic direction has an average daily traffic of 45,700 vehicles and an average daily truck traffic of 12%, resulting in an average daily traffic of 91,400 vehicles and average daily truck traffic of 10,968 trucks under No. 609280. The posted speed limit at this location on I-35 is 65 mph and the minimum vertical underclearance is 16.9 ft. The ToW is 19% and the atmospheric chloride concentration is 0.07 ppm. The site is generally open, as shown by the elevation view in Figure 74.

The relatively poor performance of the bottom flange is generally expected, as the site meets the requirements of the Iowa DOT to qualify the superstructure for either full painting or painting only in the most vulnerable locations. Despite the aggressive environment and the poor quality patina on the bottom flange, the bridge has not been recommended for preventive maintenance such as cleaning or painting at this time.

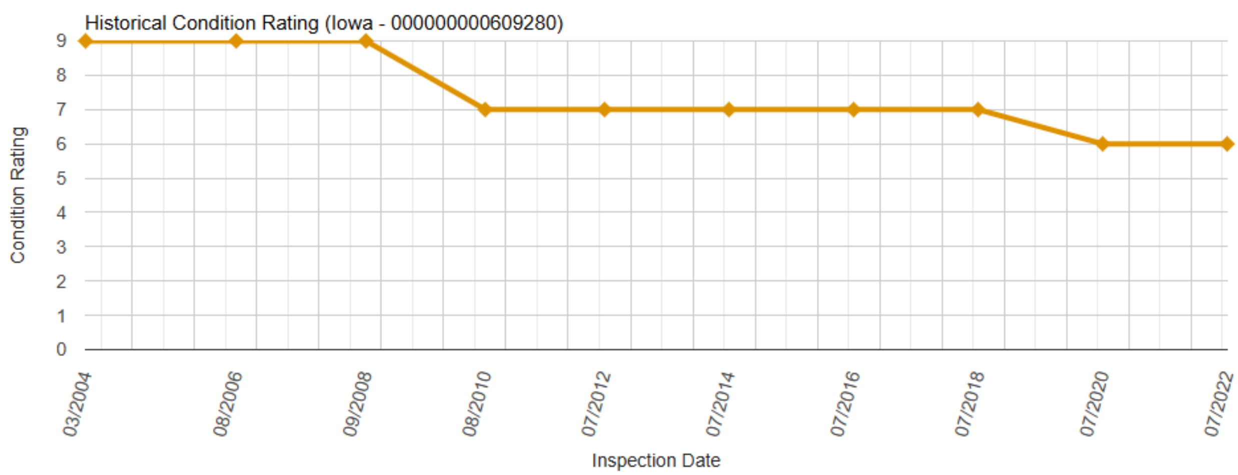

Figure 72. Superstructure condition rating of No. 609280 since the year the bridge was constructed (2003), from InfoBridge.

Figure 73. Photograph of typical “light flaking rust above roadways” on underside of a bottom flange of a girder belonging to No. 609280, dated September 29, 2010, from NBI inspection records.

Figure 74. Elevation view of No. 609280 as of September 2024 facing north, from Google Maps.

The third uncoated or partially coated weathering steel bridge in Iowa that currently has a general condition rating of 6 for its superstructure, No. 015225, is of greatest interest because it is 52 years of age. The available superstructure condition rating history for No. 015225, as reported by InfoBridge starting in 1982, is shown in Figure 72. The superstructure general condition rating was briefly “7 – Good,” defined as “[s]ome minor defects” by SNBI (2022), in the later 1980s but was restored to an “8 – Very Good,” defined as “[s]ome inherent defects” by SNIBI (2022) in 1989. The reason for this change is unknown, and may be caused by maintenance on the superstructure or by variation in the general condition ratings assigned by different inspectors. The true reasons cannot be determined because of the sparsity of digital records during this time period, but because the superstructure remained in “good” condition, commonly defined as a general condition rating of 7 to 9, until 2023, this brief decrease is not considered a reflection of poor performance of the weathering steel.

The general condition rating of the superstructure decreased to a 6 in 2023, but records indicate that this was caused by collision damage rather than corrosion of the weathering steel. Element-level data from the 2023 inspection reported that the entire quantity of the weathering steel patina was in Condition State 2 and no corrosion defects were reported for the steel girders and other elements making up the steel superstructure. The collision damage reported in the 2023 inspection was absent in the 2021 inspection report.

No. 015225 carries eastbound IA 930, a rural principal arterial route, over US 30, classified as an urban principal arterial route, to the west of Ames, Iowa. Based on the two-way traffic data for the culvert carrying US 30 over Honey Creek, US 30 has an average daily traffic of 8,800 vehicles and an average daily truck traffic of 13%. The posted speed limit of US 30 is 65 mph, the minimum vertical underclearance of the bridge is 17.4 ft, the ToW is 36%, and the atmospheric chloride concentration is 0.07 ppm. Based on these qualities, the site does not classify as an interior environment based on the definitions by McConnell et al. (2024) nor based on the criteria of the Iowa DOT.

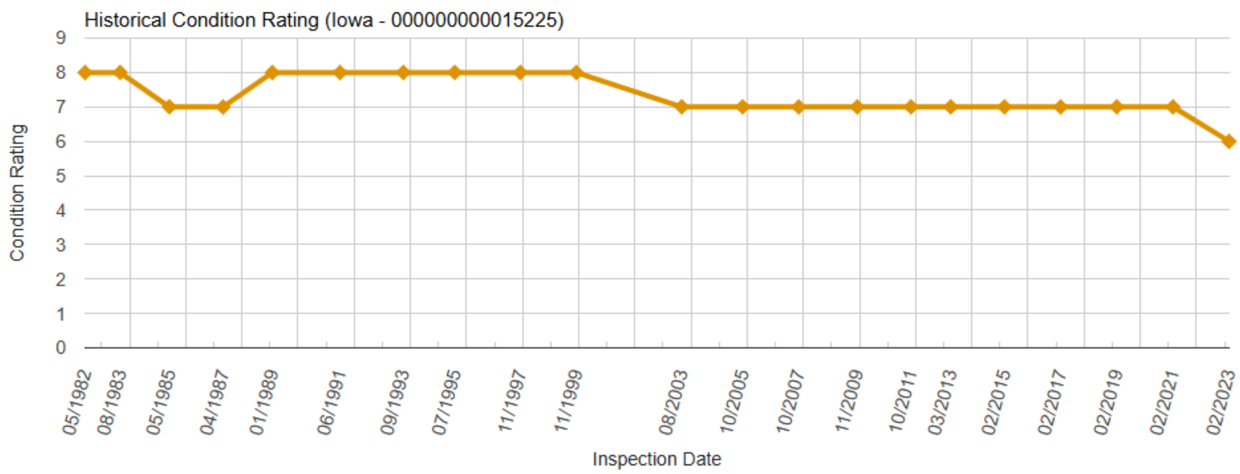

Figure 75. Superstructure condition rating of No. 015225, constructed in 1972, for which NBI records are available from InfoBridge.

In addition to the assessment of the general condition ratings of the uncoated and partially coated weathering steel bridges across Iowa and reasons for comparatively poor performance, the documented maintenance needs of the inventory were compiled to identify if accelerated corrosion at locations along the superstructure potentially subject to aggressive conditions was triggering maintenance requests. Superstructure maintenance recommended by inspectors included:

- **Superstructure Cleaning.** Cleaning was recommended for two bridges, No. 042761 and No. 041451. No. 042761 was constructed in 2003 and was recommended for routine washing in 2011 at 8 years of age because of the presence of flaking rust on a girder, noted to be caused by snow removal and chloride exposure. No. 041451 was constructed in 1999 and was recommended for pressure washing in 2015 to remove smoke soot from the girders and the underside of the deck.

- **Repair of Loose Flange Deflectors.** Loose flange deflectors requiring maintenance, shown in Figure 76, were reported for two bridges, No. 606750, which is the weathering steel bridge experiencing relatively accelerated corrosion, and No. 600200. Loose flange deflectors were reported on No. 606750, which was constructed in 1997, at an age of 18 years and on No. 600200, which was constructed in 1970, at an age of 50 years. For No. 606750, the inspector noted that the bolts connecting the deflector plates to the bottom flange were rusted, and pitting and approximately 1/8 of an inch of section loss was present on the tops of the bottom flanges where the plates were set. The observations of the inspector are consistent with crevice corrosion.
- **Painting.** Only one uncoated or partially coated weathering steel bridge, No. 609710, was recommended for zone painting of the superstructure, and painting of the faying surfaces of the diaphragms and bolts at the abutments was completed at an age of 6 years. Five other uncoated or partially coated weathering steel bridges, all along the same corridor, are programmed for bridge painting in the near future. However, the notes for one of the bridges states that elements within the substructure are to be painted and the superstructure appears to be out of the scope of the painting.

Figure 76. Photograph of a loose deflector plate on No. 606750, dated December 23, 2019, from NBI inspection records.

In summary, the overall performance of 133 known uncoated or partially-coated weathering steel bridges in Iowa, which range in age from 13 to 54 years, was analyzed. An in-depth review of bridges that currently have low general condition ratings of 6 compared to the rest of the inventory and the superstructure maintenance recommended for the 133 bridges was performed to determine if localized, accelerated corrosion has been causing the weathering steel superstructures to require maintenance, and the timing of such maintenance if so.

Based on the findings, the risk of maintenance needs associated with accelerated corrosion caused by locally aggressive exposure conditions is low. Of the 133 bridges, only three had superstructures at a general condition rating of 6 while the remaining bridge superstructures had greater general condition ratings of 7 or 8. Further, the relatively low condition rating of only one superstructure (No. 606750) could be attributed to localized accelerated corrosion at vulnerable locations along the superstructure despite the fact that the site is considered suitable for uncoated weathering steel based on current industry and DOT guidance. The general condition rating of the superstructure of No. 606750 decreased to a 6 at an age of 18 years.

The second superstructure with a relatively low condition rating of 6 (No. 609820) is located at a site that is considered aggressive towards uncoated weathering steel by the current practice of the Iowa DOT and for which coating of additional vulnerable areas or even full painting of the superstructure is encouraged by the Iowa DOT. The general condition rating of the superstructure of No. 609820 decreased to a 6 at a similar age of 17 years and although the decrease cannot be attributed to the presence of steel corrosion, the patina on the surfaces of the bottom flanges of the girders is reportedly of poor quality. This superstructure is an example of the deterioration rate that may be expected of uncoated or partially uncoated weathering steel bridges in aggressive environments.

The general condition rating of the final superstructure (No. 015225) decreased to a 6 at an age of 51 years, but the decrease appears to be caused by collision damage rather than corrosion. This superstructure is therefore an example of the long life that can be achieved with uncoated weathering steel superstructures when used at a suitable site.

Additionally, only five bridges had records of superstructure maintenance recommendations, of which four were associated with addressing and/or preventing corrosion. The maintenance recommendations and the timing of each recommendation are summarized in Table 39. One of the recommendations appeared to be purely preventive instead of triggered by corrosion. Cleaning triggered by the presence of nonprotective rust products was recommended for one bridge at an age of 8 years, and correction of loose flange deflectors was recommended for two bridges, one at 18 years of age and the other at 50 years of age. Bridge No. 606750, for which locally accelerated corrosion caused it to deteriorate more rapidly than the majority of the uncoated or partially coated weathering steel bridges in Iowa as discussed above, was the bridge for which corrective maintenance consisting of reattachment of flange deflectors was recommended at an age of 18 years.

Table 39. Summary of superstructure maintenance recommendations for uncoated or partially coated weathering steel bridges in Iowa

FHWA No.	Recommended Maintenance	Age at Time of Recommendation	Reason for Recommendation
609710	Zone painting of diaphragm connections at abutments	6 years	None given; appears to be preventive based on lack of record of corrosion issues and superstructure general condition rating of 8
042761	Superstructure cleaning	8 years	Presence of nonprotective, flaking rust associated with chloride exposure on a girder
606750	Correction of loose flange deflectors	18 years	Presence of loose flange deflectors, corrosion of bolts securing the deflectors, and signs of accelerated corrosion on top of bottom flanges underneath the deflector plates
600200	Correction of loose flange deflectors	50 years	Presence of loose flange deflectors

In total, four of the 133 uncoated or partially coated weathering steel bridges in Iowa, corresponding to 3.0%, have experienced relatively high deterioration rates and/or required unintended maintenance due to corrosion of uncoated weathering steel within their lifetimes so far based on available records. This percentage is not the likelihood that an uncoated or partially coated weathering steel superstructure in Iowa will experience unexpectedly high deterioration due to corrosion or need corrective maintenance addressing corrosion; such a value would be site- and bridge-specific and is too complicated to be developed by this analysis. Instead, this percentage demonstrates that the current policies of the Iowa DOT have a high rate of success in avoiding accelerated corrosion and associated maintenance needs for uncoated and partially coated weathering steel bridges in Iowa, although the analysis has two primary limitations. First, 98% of the uncoated or partially coated weathering steel bridges in this analysis were at an age of 29 years or less, with only three such superstructures at an age of 52 to 54 years. As a result, the corrosion-associated maintenance needs of uncoated or partially coated weathering steel superstructures in Iowa at ages greater than 29 years remains largely unknown. Second, this analysis assumed that no corrosion-associated maintenance was needed if no record of the need or maintenance exists, but detailed inspection data and maintenance records prior to approximately 2010 are generally unavailable. Therefore, this assumption is a potential source of inaccuracy, particularly for the oldest superstructures in the inventory. Continued monitoring of the performance of the existing inventory of uncoated and partially coated weathering steel bridges in Iowa would be needed to verify the results of this analysis and characterize corrosion performance and maintenance needs at over 30 years of age.

6.2.4.3 Performance of Paint Systems

As described previously in Section 6.2.2.2.1, Paint Systems, the primary manner in which paint systems protect underlying metal from corrosion is by serving as a physical barrier between the metal and the corrosive environment. However, paint systems have some amount of porosity and permeability such that moisture and ions, such as chlorides, will eventually penetrate the system, reach underlying metal, and cause corrosion. When a zinc-rich primer is present, as in the two-coat paint system currently used by the Iowa DOT, the zinc within the primer will corrode instead of the steel substrate, at least until the zinc is consumed. However, regardless of whether zinc or steel is corroding, the corrosion will generate corrosion products and cause disbondment, bubbling, and peeling of overlying coats of paint. This type of paint failure is commonly observed in paint systems on steel bridges and identified under Defect 3420 – Peeling/Bubbling/Cracking, applicable for Steel Protective Coatings, during element-level inspection. Paint systems may also experience other degradation mechanisms, such as chalking, which is a form of degradation caused by exposure to ultraviolet radiation, and other types of distress, such as loss of gloss or poor color retention, which are generally aesthetic concerns.

Paint systems are vulnerable to accelerated deterioration in many of the same locations where steel is expected to experience accelerated corrosion because more severe exposure to moisture and chlorides will cause these agents to penetrate the paint layers, initiate corrosion, and cause disbondment more rapidly. Paints are also at risk of degrading more quickly at element edges. During application, the paint naturally pulls away from edges when it is in a liquid state. As a result, the dry film thickness tends to be relatively thin at edges compared to the general surface area away from edges. The risk of decreased thickness along edges is commonly addressed by applying an additional coat of paint only along edges and corners, referred to as a stripe coat. Stripe coating may also be conducted at connections where crevice corrosion is considered a risk. Stripe coats may be applied using the primer prior to the application of the general paint system, or after the paint system has been applied, in which case the paint used for the topcoat may be used for stripe coating.

The expected lives of paint systems are of interest because it is often the deteriorated condition of the paint system that triggers maintenance of painted steel superstructures rather than corroded steel conditions. While a bridge owner could permit the paint system to degrade since it does not directly impact structural capacity, the restoration of the paint system and the protection that it offers is generally accepted to be a worthwhile corrosion protection strategy, even though maintenance painting and particularly full removal and replacement of the paint system, which may be necessary if the existing paint system has extensive and/or severe enough deterioration, can be expensive to the agency and disruptive to traffic. Therefore, the identification of long-lived paint systems or alternative coatings with minimal maintenance needs is highly desirable.

When considering a paint system for new construction, the “expected service life” of the paint system is usually expressed as the time-to-maintenance. This approach acknowledges the fact that paint systems are frequently maintained in practice to extend their life, and that actual service life is difficult to predict because it depends in large part on the maintenance policies and actions of the owner. While the extent and/or severity of degradation that justifies maintenance of the paint system is up to the owner, a threshold of 5% coating breakdown is a typical assumption.

For expected service lives of paint systems, practitioners often refer to the “estimated practical maintenance times” identified by Helsel and Lanterman (2022) for various steel protective coating systems in their paper “Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work.” Helsel and Lanterman (2022) define the “practical maintenance time” as the time at which 5 to 10% coating breakdown occurs and active rusting of the substrate is present, at which point maintenance painting is assumed to be triggered. Estimated practical maintenance times for various generic coating systems, identified by the type of surface preparation (i.e., hand/power tools or abrasive blasting), generic coating type for each coat (e.g., inorganic zinc/epoxy/polyurethane), number of coats, and minimum total dry film thickness, are provided in Table 1A of Helsel and Lanterman (2022) for three different atmospheric exposure environments. The atmospheric exposure environments are defined by ISO 12944-2, "Paints and varnishes – Corrosion protection of steel structures by protective paint systems – Part 2: Classification of environments" and described as follows (Helsel & Lanterman, 2022):

- Mild (rural)/C2 – "Low – Atmospheres with low levels of pollution; mostly rural areas,"
- Moderate (industrial)/C3 – "Medium – Urban and industrial atmospheres, moderate sulfur dioxide pollution; coastal areas with low salinity," and
- Severe (heavy industrial)/C5 – "Very High – Industrial areas with high humidity and aggressive atmosphere and coastal areas with high salinity."

The atmospheric environments across Iowa, in the context of coatings, are expected to generally fall under "Mild (rural)/C2" or "Moderate (industrial)/C3" and are not expected to fall under "Severe (heavy industrial)/C5." The typical paint system used by the Iowa DOT, consisting of abrasive blasting to a near-white metal finish, an inorganic zinc primer with a minimum thickness of 3 mils, and a waterborne acrylic topcoat with a minimum thickness of 2 mils, is sufficiently uncommon that it is not listed in Table 1A. However, estimates for a 1-coat system consisting only of an inorganic zinc coat on an abrasive blasted surface and a 3-coat system consisting of an inorganic zinc coat and two coats of waterborne acrylic paint on top of an abrasive blasted surface are provided. The estimated practical maintenance times for these paint systems are shown in Table 40 and reportedly vary from approximately 15 to 24 years in Mild (rural)/C2 and Moderate (industrial)/C3 environments. The performance of the 2-coat system used by Iowa may be assumed to be similar.

The estimated practical maintenance times of the paint systems most commonly used across the United States for new steel bridge construction, as identified by Azizinamini et al. (2014), are also shown in Table 40 for comparison. These systems have longer expected times-to-maintenance of 20 to 34 years in Mild (rural)/C2 and Moderate (industrial)/C3 environments compared to inorganic zinc and waterborne acrylic systems.

Table 40. Estimated practical maintenance times of paint systems used for new bridge construction in the United States in various atmospheric environments (Helsel & Lanterman, 2022)

Coating Systems (primer/midcoat/topcoat)	Surface Preparation Method	No. of Coats	Minimum Dry Film Thickness (mils)	C2	C3	C5
Paint Systems Similar to Those Used in Iowa						
Inorganic zinc	Blast	1	3	21	15	5
Inorganic zinc/acrylic waterborne/acrylic waterborne	Blast	3	7	24	17	12
Paint Systems Commonly Used in the United States Outside of Iowa						
Inorganic zinc/epoxy/polyurethane	Blast	3	11	30	21	16
Epoxy zinc/epoxy/polyurethane	Blast	3	10	29	20	14
Organic zinc/epoxy/polysiloxane	Blast	3	12	30	21	15
Epoxy zinc/epoxy/fluorinated polyurethane	Blast	3	10	34	24	18
Zinc-rich MCU/MCU/MCU[1]	Blast	3	9	29	20	14

Notes: [1]*MCU refers to a moisture-cured urethane.*

6.2.4.4 Performance of Metallic Coatings

The expected life of fully metallic coatings, i.e., galvanizing and unsealed metallizing, can be estimated by using an assumed corrosion rate and coating thickness. The selection of a single corrosion rate and a single coating thickness results in a basic deterministic approach for predicting the life of metallic coatings and only provides an estimate of the general age at which the metallic coating is expected to have been fully consumed. A more sophisticated model would account for:

- Variability in the metallic coating corrosion rates, caused by both inherent uncertainty in the corrosion rates that will actually occur at the site and variable exposure conditions across the superstructure due to localized aggressive conditions, the former of which is acknowledged by quantitative ranges of potential corrosion rates and the latter of which is discussed qualitatively; and
- Variability in the coating thickness, which is of greater risk in metallizing wherein the coating thickness is relatively sensitive to the expertise of the operator compared to galvanizing, and which has not been addressed qualitatively or quantitatively in the basic approach applied in this section.

Because the basic deterministic approach does not quantitatively account for variability, it cannot provide insight regarding the estimated practical maintenance time of the metallic coatings as is typically considered for the paint systems, although experience-based estimates are provided by Helsel and Lanterman (2022) for metallizing, both sealed and unsealed. However, the minimum time of general failure that is obtained by following the deterministic approach described in this section is sufficient for comparison to the performance of other potential corrosion control strategies for steel superstructures in Iowa.

6.2.4.4.1 Assumed Corrosion Rates of Galvanizing and Metallizing

Zinc corrosion rates based on exposure testing of zinc coupons may be assumed in the calculation, but with the understanding that they are subject to several sources of inaccuracy, especially when applied to zinc galvanizing or metallizing. First, as acknowledged in Section 6.2.4.2.2, Corrosion Rates of Weathering Steel in Suitable Sites, the rate of corrosion experienced by a coupon during standardized field exposure testing, i.e., oriented in a standard orientation with “bold” exposure, will differ from the rate of corrosion experienced by structural bridge elements. Second, the zinc coupons will generally have a different chemical composition than galvanizing or metallizing. While the topmost layer of galvanizing is purely zinc, deeper layers of the galvanizing are zinc-iron alloys. In the case of metallizing, even though metallizing can be done with pure zinc, an 85/15 zinc-aluminum alloy is more commonly used, which corrodes more slowly than pure zinc metallizing. And third, also in the case of metallizing, the microstructure of the metallic coating is relatively porous such that the coating is expected to retain moisture for greater amounts of time and experience relatively high corrosion rates compared to galvanizing or pure zinc metal, unless a sealer is applied. Despite these inaccuracies, zinc corrosion rates according to field exposure testing are assumed to be suitable for predicting the life of the metallic coatings, particularly in the absence of data specific to galvanizing and metallizing.

The zinc corrosion rates that correspond to the corrosivity categories C2, C3, and C4, as defined in ISO 9223, are shown in Table 41. The metal loss in the first year (r_{Zn1}) is used by ISO 9223 to define the corrosivity categories. The average long-term zinc corrosion rates over the first 10 years of exposure (r_{Zn10})

or the first 30 years of exposure (r_{Zn30}) are provided by ISO 9224 as guidance for predicting long-term metal loss.

Table 41. Corrosion rates expected to be experienced by zinc in atmospheric corrosivity categories defined by ISO 9223 that may be encountered in Iowa (ISO, 2012; ISO, 2012).

ISO 9223 Corrosivity Category	Zinc Corrosion in First Year of Exposure (r_{Zn1}), µm/year	Avg. Zinc Corrosion Rate During First 10 Years (r_{Zn10}), µm/year	Avg. Zinc Corrosion Rate During First 30 Years (r_{Zn30}), µm/year
C2 – Low	$0.1 < r_{Zn1} \le 0.7$	$0.07 < r_{Zn10} \le 0.5$	$0.05 < r_{Zn30} \le 0.4$
C3 – Medium	$0.7 < r_{Zn1} \le 2.1$	$0.5 < r_{Zn10} \le 1.4$	$0.4 < r_{Zn30} \le 1.1$
C4 – High	$2.1 < r_{Zn1} \le 4.2$	$1.4 < r_{Zn10} \le 2.7$	$1.1 < r_{Zn30} \le 2.2$

As a point for comparison, the American Galvanizers Association has published guidance pertaining to the expected corrosion of galvanizing, but expressed in terms of the time to first maintenance instead of zinc corrosion rates (AGA, n.d.). This guidance defines the time to first maintenance as the time at which 5% of the surface is experiencing rusting and corrosion of the steel substrate, in alignment with typical practice for paint systems as described earlier. The time to first maintenance or TFM chart presents the time to first maintenance as a function of the average thickness of the zinc, based on the assumption of a linear long-term corrosion rate. The long-term corrosion rates assumed by the TFM chart and the locations on which they are based are presented in Table 42. The article notes that the estimated times are conservative for galvanizing in the 21st century because of the improvement in air quality, implying that the corrosion rate data is relatively old.

Table 42. Long-term zinc corrosion rates assumed by the TFM (time to first maintenance) chart published by the AGA for determining time to first maintenance based on average zinc thickness (AGA, n.d.).

Atmospheric Environment	Long-Term Zinc Corrosion Rate (µm/year)	Data Locations
Rural	0.77	Boise, ID; Las Cruces, NM; Fargo, ND; Little Rock, AK; Macon, GA
Suburban	1.1	Vallejo, CA; Tucson, AZ; Cedar Rapids, IA; Jackson, MS; Harrisburg, PA; Columbia, SC
Temperate Marine	1.2	Seattle, WA; San Francisco, CA; Milwaukee, WI; Norfolk, VA; Atlantic City, NJ; Boston, MA
Tropical Marine	1.3	Miami, FL; Corpus Christi, TX; San Diego, CA; Cancun, Mexico; Mazatlan, Mexico
Industrial	1.4	Pocatello, ID; Los Angeles, CA; Chicago, IL; Dallas, TX; New York, NY; Knoxville, TN

Like steel, zinc experiences increased corrosion rates when exposed to greater amounts of moisture and chlorides. Therefore, relatively rapid zinc corrosion compared to the general zinc corrosion rate is expected at many, although not all, of the same locations identified as being at risk for accelerated steel corrosion in Section 6.2.4.1.2, Potential Aggressive Conditions. The protective oxide film on the zinc surface may not necessarily break down entirely in these environments, as the patina of weathering steel would under perpetual moisture conditions and/or aggressive chloride conditions, but would at least be of reduced effectiveness and permit greater passive corrosion rates. Interestingly, a study published by

ASTM in 1968 demonstrated that the ratio of the corrosion rate of zinc to the corrosion rate of steel decreases with increasing environmental exposure severity (Corrosiveness of Various Atmospheric Test Sites as Measured by Specimens of Steel and Zinc, 1968). In other words, even though zinc and steel corrosion rates both increase with increasing exposure severity, the increase in the steel corrosion rate outpaces the increase in the zinc corrosion rate such that the benefit of a zinc-based protective coating increases with increasing exposure severity. The data supporting this behavior was intended to characterize the corrosion rates of zinc and steel across different exposure sites, but logically, this behavioral trend could be expected to apply to microclimates of varying exposure severity within the same bridge site as well.

Of the locations identified as being at risk for accelerated corrosion in Section 6.2.4.1.2, Potential Aggressive Conditions, the embedded beam ends are a unique case when considering zinc corrosion. Steel passivates in concrete due to its high pH (provided the concrete pH has not been decreased by carbonation and the concrete does not contain sufficient amounts of chlorides to initiate corrosion). However, zinc may not passivate when exposed to concrete alkalinity, particularly when the concrete has a pH of approximately 13 or greater (Andrade & Alonso, 2004). As a result, the corrosion mechanism driving accelerated steel corrosion adjacent to the embedded beam ends does not necessarily apply to zinc. However, because the zinc coating on the length of steel embedded in concrete may not develop a passive layer like the zinc exposed to atmosphere, the zinc within the concrete may be expected to have greater corrosion rates than the zinc exposed to atmosphere.

6.2.4.4.2 Assumed Thickness of Galvanizing and Metallizing

The metallic coating thickness assumed depends on the type of metallic coating under consideration. Preemptively predicting the thickness of a galvanized coating is challenging because the coating thickness will depend on the steel composition and, in some cases, the dwell time within the zinc bath, as explained in Section 6.2.2.2.2, Hot-Dip Galvanizing. As a result, the specified minimum thickness of the metallic coating is assumed to be a reasonable, if conservative, thickness for coating life prediction. In contrast, designers and contractors have much greater control over the thickness of metallizing such that the desired coating life can be used to specify a desired metallizing thickness, although the selected thickness needs to be checked for practicality once it has been determined. Debonding may occur if the thickness of the metallizing is too high.

6.2.4.4.3 Expected Life of Galvanizing

The ranges of the expected life of galvanizing on steel superstructures in Iowa according to the deterministic approach described in this section are presented in Table 43. The specified minimum thickness of the galvanizing is from ASTM A123-24, *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*, and ASTM A153-23, *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware*. As shown in the table, different galvanizing thicknesses are specified depending on the type of item or shape and the size of the item. ASTM A123 provides requirements for the minimum average thickness of the galvanizing for structural shapes, strips and bars, plates, pipes and tubing, wires, and reinforcing bars. If an element falls under different categories because its thickness varies, e.g., if the web and the flange of an I-beam have different thicknesses that cause different minimum average galvanizing thicknesses to apply, then the element is considered to have multiple “items” and the minimum average galvanizing thickness for each individual item according to its size and shape applies. While minimum requirements for the average galvanizing thickness are provided, ASTM A123 does not identify a minimum thickness requirement for individual measurements.

ASTM A153 provides requirements for the minimum thickness of galvanizing on castings, fasteners, washers, and other rolled, pressed, or forged articles. The standard provides both a minimum requirement for the average galvanizing thickness of an inspection lot, as identified in Table 43, and a minimum requirement for any individual specimen, which is not shown in the table. Assuming the minimum for an individual specimen is considered overly conservative for the purposes of this analysis.

Table 43. Estimated time-to-consumption of zinc galvanizing coatings in Iowa based on minimum average coating thicknesses specified by ASTM A123 and ASTM A153 and ranges of average 30-year zinc corrosion rates provided by ISO 9224

Item Thickness, t, Diameter, d, or Length, L (in.)	Min. Avg. Galvanizing Thickness[1] (μm)	Time-to-Consumption in ISO 9223 C2 Exposure (yrs)	Time-to-Consumption in ISO 9223 C3 Exposure (yrs)	Time-to-Consumption in ISO 9223 C4 Exposure (yrs)
Structural Shapes				
t < 1/16	35	88 to >200	32 to 88	16 to 32
1/16 ≤ t < 1/8	60	150 to >200	55 to 150	27 to 55
1/8 ≤ t < 1/4	65	163 to >200	59 to 163	30 to 59
1/4 ≤ t	85	>200	77 to >200	39 to 77
Plates				
t < 1/16	35	88 to >200	32 to 88	16 to 32
1/16 ≤ t < 1/8	60	150 to >200	55 to 150	27 to 55
1/8 ≤ t < 5/8	65	163 to >200	59 to 163	30 to 59
5/8 ≤ t	85	>200	77 to >200	39 to 77
Castings				
Not applicable	86	>200	78 to >200	39 to 78
Fasteners				
d ≤ 3/8	43	108 to >200	39 to 108	20 to 39
3/8 < d	53	133 to >200	48 to 133	24 to 48
Washers				
t < 3/16	43	108 to >200	39 to 108	20 to 39
3/16 ≤ t	53	133 to >200	48 to 133	24 to 48
Rolled, Pressed, and Forged Articles[2]				
5/8 ≤ t and 15 < L	86	>200	78 to >200	39 to 78
t < 5/8 and 15 < L	66	165 to >200	60 to 165	30 to 60
L < 15	56	140 to >200	51 to 140	25 to 51

Notes: [1]*The minimum average galvanizing thickness is according to ASTM A123-24 for structural shapes and plates and according to ASTM A153-23 for the other elements listed in the table, which are generally considered hardware.*

[2]*Other than the fasteners and washers already identified in the table.*

The estimated times at which the galvanizing would be fully consumed based on the range of average 30-year zinc corrosion rates identified in ISO 9224 for the atmosphere corrosivity classes expected to be found in Iowa are presented in the three rightmost columns of Table 43. For C2 exposure, a coating life of 88 to over 200 years is expected for the possible types and sizes of items. However, the use of structural shapes and plates less than 1/16 inch in thickness is highly unlikely, in which case the galvanizing on the structural elements is expected to be present for at least 150 years and the galvanizing on the fasteners and other hardware is expected to be present for at least 108 years. For C3 exposure, a coating life of 32 to over 200 years is expected and for C4 exposure, a coating life of 16 to 77 years is expected.

For comparison, the time to first maintenance (TFM) of galvanizing with the average thicknesses listed in Table 43 based on the TFM chart published by the AGA and corresponding zinc corrosion rates is shown in Table 44. Estimates for rural, suburban, temperate marine, and industrial environments are presented since much of Iowa is expected to be rural; data from Cedar Rapids, Iowa and Milwaukee, Wisconsin was used to inform the assumed corrosion rates for suburban and temperate marine exposure, respectively; and an industrial environment is the most severe estimate provided by the AGA.

The TFM of the galvanizing in rural environments per the AGA resource is much lower than the time-to-consumption based on the long-term zinc corrosion rates offered by ISO 9224 for C2 exposure. This may be expected because of the different end conditions, i.e., the TFM assumes 5% of the coated area is experiencing rusting of the steel substrate while the time-to-consumption assumes the galvanizing has been consumed to a depth equal to the average coating thickness across 100% of the coated area. Additionally, the AGA acknowledged that the data used for establishing zinc corrosion rates in the TFM chart is from a time period of relatively poor air quality and as a result, the estimates of TFM may be conservatively low. However, the TFM in “temperate marine” environments, which align with the qualitative definitions of C4 or C5 exposure per ISO 9223, is consistently within the range of the estimated time-to-consumption presented in Table 43 for C4 exposure. This contradicts the explanation for the differences in expected performance between “rural” and C2 environments. The comparison between the TFM estimated based on guidance from the AGA and the time-to-consumption estimated based on guidance from ISO highlights the importance of selecting a corrosion rate that is appropriate to assume for the site(s) under consideration and understanding both the data and the analysis or interpretation used to develop corrosion rate guidance.

Table 44. Estimated time to first maintenance of galvanizing of various average zinc thicknesses based on zinc corrosion rates published by the AGA (AGA, n.d.).

Avg. Zinc Thickness (μm)	TFM in Rural Environments (years)	TFM in Suburban Environments (years)	TFM in Temperate Marine Environments (years)	TFM in Industrial Environments (years)
35	45	35	30	26
43	56	42	37	32
53	69	52	45	39
56	72	54	48	41
60	78	58	51	44
65	84	63	55	47
85	110	82	72	62

Ideally, the expected life of galvanized steel bridges and their maintenance needs could be validated through field performance data. However, as acknowledged earlier in Section 6.2.2.2.2, Hot-Dip Galvanizing, no comprehensive field performance study of galvanized bridges has been conducted in the United States to date. Kogler (2015) identified several studies that investigated the field performance of a limited number of galvanized steel bridges. One study in Ohio reportedly assessed two galvanized steel bridges located in rural environments with at least 10 years of exposure prior to 1983 in detail and determined that the galvanizing was expected to have a total life of 60 years for one bridge and 90 years for the other. This case study aligns more closely with the TFM predicted by AGA for rural environments or the time-to-consumption assuming a C3 environment per ISO 9223.

Lastly, localized areas of a superstructure where the galvanizing may be consumed more quickly because of relatively aggressive exposure conditions have already been discussed qualitatively. However, an additional vulnerability of galvanizing is the location of any repairs to the galvanizing where the coating may have been damaged during transport, handling, and construction. Damaged galvanizing coating may be repaired by metallizing, in which case similar performance between the undisturbed galvanizing and the metallizing repair may be expected. However, damaged galvanizing is also sometimes repaired using an organic zinc-rich paint, as the Iowa DOT uses for repairing damage to the galvanizing on fasteners. While the surrounding galvanizing may affect the service life of the paint repair, an organic zinc-rich paint or a paint system with an organic zinc-rich primer is generally expected to last approximately one to three decades depending on the exposure severity, which is generally a much shorter life than that expected of the galvanizing. A galvanized coating consequently may require maintenance earlier than expected at locations of paint-based repairs.

6.2.4.4.4 Expected Life of Metallizing

The thickness of metallizing can be controlled to a much greater extent than galvanizing. Therefore, instead of estimating the time-to-consumption or time to first maintenance of metallizing based on a minimum specified thickness, the minimum metallizing thicknesses needed to achieve 50, 75, 100, 125, and 150 years before the metallizing is consumed based on different corrosion rates are presented in Table 45, which shows time-to-consumption according to average 30-year zinc corrosion rates from ISO 9224, and Table 46, which shows TFM according to the corrosion rates assumed by the AGA in the TFM chart. The discussion comparing the values developed following guidance from ISO 9223 and ISO 9224 and the values developed following guidance from AGA that was presented in Section 6.2.4.4.3, Expected Life of Galvanizing, applies to these tables as well since they simply reflect a different application of the same sets of corrosion rates.

Reportedly, a metallizing thickness of 10 to 16 mils (254 to 406 μm) is generally recommended for a service life greater than 40 years, which includes a safety factor of at least two (Kogler, 2015), and a thickness of 20 mils (508 μm) is commonly achievable (Cramer & Covino, Jr., 2003). Based on these ranges, the required metallizing thicknesses presented in Table 45 and Table 46 are achievable.

Table 45. Metallizing thickness required for various target times-to-consumption based on average 30-year zinc corrosion rates published by ISO 9224.

Avg. Zinc Thickness for a Time-to-Consumption of:	ISO 9223 C2 Exposure	ISO 9223 C3 Exposure	ISO 9223 C4 Exposure
50 years	< 25 µm	< 25 µm to 55 µm	55 to 110 µm
75 years	< 25 to 30 µm	30 to 83 µm	83 to 165 µm
100 years	< 25 to 40 µm	40 to 110 µm	110 to 220 µm
125 years	< 25 to 50 µm	50 to 138 µm	138 to 275 µm
150 years	< 25 to 60 µm	60 to 165 µm	165 to 330 µm

Table 46. Metallizing thickness required for various target times to first maintenance based on zinc corrosion rates published by the AGA (AGA, n.d.).

Avg. Zinc Thickness for a TFM of:	Rural	Suburban	Temperate Marine	Tropical Marine	Industrial
50 years	39 µm	51 µm	59 µm	66 µm	68 µm
75 years	58 µm	78 µm	88 µm	98 µm	103 µm
100 years	77 µm	105 µm	118 µm	131 µm	137 µm
125 years	96 µm	131 µm	147 µm	164 µm	172 µm
150 years	116 µm	158 µm	177 µm	197 µm	206 µm

Additionally, Helsel and Lanterman (2022) provide estimated practical maintenance times, which are generally equivalent to the TFM, for both sealed and unsealed metallizing. The estimated practical maintenance times are presented in Table 47. The minimum metallizing thickness assumed by Helsel and Lanterman (2022) is 8 mils (203 µm). If the long-term corrosion rates within the TFM chart for galvanizing are applied to this thickness, then the estimated TFM varies from 263 years in rural environments to 147 years in industrial environments – approximately 8 or 9 times the estimated practical maintenance times identified by the work by Helsel and Lanterman (2022).

The almost order-of-magnitude difference between the experiential estimates from Helsel and Lanterman (2022) and the estimated TFM based on zinc galvanizing corrosion rates from the AGA calls the validity of the necessary thicknesses presented in Table 46, and by extension those presented in Table 45, into question. On one hand, unsealed metallizing is expected to corrode more quickly than galvanizing because of its greater porosity. On the other hand, exposure testing of metallized coupons supports the position that metallizing can last for the many decades predicted by following the deterministic model (Kogler, 2015). Additionally, the inherent differences between metallized and galvanized coatings may be contributing to the difference between the estimated practical maintenance times and the estimated TFM. Specifically, a metallized coating is expected to have more locations vulnerable to shorter service life than a galvanized coating; for example, metallizing is typically thinner along edges while galvanizing is not. As a result, even if a metallized coating and a galvanized coating corrode at the same rate, the metallized coating may reach the 5 to 10% steel substrate rusting threshold more quickly than the galvanized coating because of the greater and more extensive variability in its thickness.

Table 47. Estimated practical maintenance times of metallizing-based coating systems in various atmospheric environments (Helsel & Lanterman, 2022)

Coating Systems	Surface Preparation Method	Minimum Dry Film Thickness (mils)	Mild (rural)/C2	Moderate (industrial)/C3	Severe (heavy industrial)/C5
Metallizing (min. 85% zinc)	Blast	8	33	22	16
Metallizing/sealer	Blast	9	35	25	18

6.2.4.5 Performance of Duplex Coatings

Duplex coating systems are uncommon for bridge structures in the United States and therefore have an increased risk of premature failure caused by improper design and construction associated with lack of experience, as well as potential deterioration caused by "unknown unknowns." However, if premature failure can be avoided, the long-term performance of a duplex coating is expected to be predicted by Equation 4 below (Van Eijsbergen, 1994):

$$D_{duplex} = f_s(D_{zinc} + D_{paint}) \quad \text{Equation 10}$$

Where D_{duplex} is the "durability in years of outdoor exposure until not more than five percent of the underlying steel surface has rusted," f_s is the synergistic factor between 1.5 and 2.3, and D_{zinc} and D_{paint} are "the durability factors for the zinc coating and the paint coating, respectively, when directly applied to the steel surface." The "durability" of the duplex coating, the zinc, and the paint is assumed to correspond to the estimated practical maintenance time or TFM of each coating or coating system. The synergistic factor depends on the exposure severity, with smaller factors expected in more severe environments, although van Eijsbergen (1994) does not provide quantitative guidance for the factors to assume in various environments beyond the given range.

The expected TFMs or estimated practical maintenance times of individual zinc-based coatings and select paint systems considered to be potential components of a duplex coating system are presented in Table 48. The assumed galvanizing thickness was chosen based on the assumption that the steel elements within the superstructure may have thicknesses as low as 3/8 of an inch. In the case of structural shapes, a 3/8-inch thickness corresponds to galvanizing with a minimum average thickness of 3.3 mils, but in the case of plates, a 3/8-inch thickness corresponds to galvanizing with a minimum average thickness of 2.6 mils. Therefore, the expected TFMs of galvanizing with a thickness of 2.6 mils are shown. For metallizing, the practical maintenance times for a "metallizing/sealer" system is shown instead of unsealed metallizing because of anticipated challenges in achieving a quality installation when applying commonly used paint systems directly on top of unsealed metallizing. The sealer is expected to improve the constructability of the system.

SSPC Guide 19, *Selection of Protective Coatings for Use Over Galvanized Substrates*, was used to identify potential paint systems for overcoating the galvanizing or sealed metallizing (SSPC, 2012). When coating new galvanized surfaces without signs of steel deterioration, i.e., rust staining, the guide recommends brush-off blast cleaning the surface. For weathered galvanizing without rust staining, the guide states that brush-off blast cleaning is not recommended because it would compromise the zinc patina, which should be preserved as much as possible. The guide further identifies acrylics, epoxies, and several other generic types of coatings that are less commonly used for steel bridge superstructures as potentially suitable materials for overcoating the galvanizing. The paint systems identified in Table 48 as potential components of duplex coatings and the practical maintenance times reflect this guidance. A three-coat acrylic system and a two-coat surface tolerant epoxy/polyurethane system are shown. Helsel and Lanterman (2022) provide different estimated practical maintenance times for these systems depending on whether the surface was prepared by hand or power tool cleaning, or by abrasive blasting to a commercial or near white finish. The surface conditions that result from brush-off blast cleaning are considered more similar to those achieved by hand or power tool cleaning, and as a result, the practical maintenance times for these systems on hand or power tool cleaned surfaces are shown.

The estimated TFMs of various combinations of the zinc-based coatings and paint systems in duplex coating systems (D_{duplex}) based on Equation 4 are included in the same table. For the purposes of this analysis, a "Mild (rural)/C2" environment per ISO 12944-2 and a "rural" environment as identified by the AGA are assumed to be similar. A "Moderate (industrial)/C3" environment, per ISO 12944-2, and a "temperate marine" environment, per AGA, are assumed to be similar, and a "Severe (heavy industrial)/C5" environment, per ISO 12944-2, and an "industrial" environment, per AGA, are assumed to be similar. These assumptions are required in order to combine the TFMs for galvanizing, which are based on AGA data and analysis, with the estimated practical maintenance times presented by Helsel and Lanterman (2022) for metallizing and paint systems. A synergistic factor f_s of 2.3 is assumed for "rural" or "Mild (rural)/C2" exposure, and 1.5 for "industrial" or "Severe (heavy industrial)/C5" exposure. For "temperate marine" or "Moderate (industrial)/C3" exposure, the midpoint of the potential range identified for the synergistic factor, 1.9, was selected.

Based on Equation 4 from van Eijsbergen (1994), duplex coatings are expected to provide over 100 years of service before needing maintenance in mild, rural environments. In severe, industrial environments, duplex coatings that use galvanizing are expected to require maintenance after approximately 80 years and duplex coatings that use metallizing are expected to require maintenance after approximately 35 to 40 years. For moderate environments, duplex coatings that use galvanizing are expected to provide over 100 years of service before requiring maintenance and duplex coatings that use metallizing are expected to provide approximately 60 to 70 years of service before requiring maintenance.

Table 48 also shows estimated practical maintenance times provided by Helsel and Lanterman (2022) for duplex coating systems consisting of metallizing with at least 2 coats of paint on top. The lives predicted by Helsel and Lanterman for duplex coating systems are much smaller than the lives predicted by the equation from van Eijsbergen (1994). For example, the "metallizing/sealer/surface tolerant epoxy/polyurethane" paint system for which practical maintenance times were estimated based on Equation 4 is considered similar to the "metallizing/sealer/epoxy/polyurethane" system for which practical maintenance times estimated by Helsel and Lanterman (2022). However, the estimates by Helsel and Lanterman vary from 37% to 64% of the estimates given by Equation 4 and the assumed synergistic factors. This conflict in expectations indicates that further study is required to understand the expected performance of duplex coating systems.

The definition of the trigger for maintenance may be contributing to differences in expectations. As identified previously, 5% steel rusting is commonly used for a paint system or metallic coating. For a duplex coating system, this would theoretically require both failure of the paint system, full consumption of the underlying zinc metal, and loss of cathodic protection in an exposed area by continued corrosion of adjacent, intact zinc. However, if aesthetics are a concern or if the owner agency practices preventive maintenance painting, as was observed by Knudsen et al. (2019), then 5% bubbling, peeling, or cracking of the paint, or a greater percent area, may trigger maintenance long before the underlying zinc coating is consumed.

Table 48. Estimated time to first maintenance of duplex coating systems in various atmospheric environments.

Coating System	Minimum Avg. Thickness (mils)	"Rural" or "Mild (rural)/C2"	"Temperate Marine" or "Moderate (industrial)/C3"	"Industrial" or "Severe (heavy industrial)/C5"
Zinc Coatings				
Galvanizing[1]	2.6	84	55	47
Metallizing/sealer[2]	9	35	25	18
Paint Systems				
Waterborne acrylic/waterborne acrylic/waterborne acrylic[2]	6	12	8	5
Surface tolerant epoxy/polyurethane[2]	7	15	10	7
Duplex Coatings, based on Equation 4 (van Eijsbergen, 1994)				
Galvanizing/waterborne acrylic/waterborne acrylic/waterborne acrylic	8.6	>200	120	78
Metallizing/sealer/waterborne acrylic/waterborne acrylic/waterborne acrylic	15	108	63	35
Galvanizing/surface tolerant epoxy/polyurethane	9.6	>200	124	81
Metallizing/sealer/surface tolerant epoxy/polyurethane	16	115	67	38
Duplex Coatings, reported by Helsel and Lanterman (2022)				
Metallizing/sealer/polyurethane	12	39	28	22
Metallizing/sealer/epoxy/ polyurethane	15	42	31	24

Notes: [1]*Values are TFMs based on the TFM chart by the American Galvanizers Association.*
[2]*Values are estimated practical maintenance time from Helsel and Lanterman (2022).*

Currently, the US's experience with duplex coating systems does not conflict with the estimated lives predicted following the equation by van Eijsbergen (1994) or those from Helsel and Lanterman (2022). While duplex coating systems in the US generally have not yet reached the service lives predicted by these references and therefore do not strongly support these estimates, they have generally provided good performance for approximately 20 years thus far such that they have not refuted these estimates either, provided that premature failure due to poor design or construction quality was avoided (Ault, 2023). Improvements in design and implementation practices are needed to prevent premature maintenance needs of these systems. Data for duplex coating systems in Norway has shown that maintenance of duplex coating systems has sometimes been needed after approximately 20 to 40 years, regardless of the corrosivity class of the environment, but some duplex coating systems have lasted almost 50 years without maintenance (Knudsen, Matre, Dorum, & Gagne, 2019). However, the systems in Norway use 100 µm (approximately 4 mils) of metallizing followed by 4 coats of alkyd-based paints and while the performance of these duplex coating systems in Norway is a good point of reference for potential performance, this performance is not directly applicable to performance in the United States due to the different environments and the different coating systems used.

6.2.4.6 Performance of ASTM A1010 (ASTM A709 Grade 50CR) Stainless Steel

There has been little study of the corrosion resistance of ASTM A1010 steel in deicing chemical environments. ASTM A1010 steel is commonly referenced as having 10 times the corrosion resistance of weathering steel based on marine exposure and accelerated laboratory testing (Fletcher F. B., 2011). The relative corrosion resistance of ASTM A1010 steel to weathering steel demonstrated by exposure testing at Kure Beach, North Carolina (Fletcher, Townsend, & Wilson, 2003), expressed in terms of the ratio between the average annual corrosion rates of weathering steel coupons and those of ASTM A1010 coupons, is shown in Table 49. The data generally supports the assertion that ASTM A1010 steel is expected to corrode at approximately a tenth of the rate of weathering steel, although longer-term test data at the more severe lot of Kure Beach, North Carolina, indicated a smaller benefit.

The steel continuous Moore Drive Bridge in Rochester, New York has been used as a site for coupon exposure testing to compare the corrosion resistance of ASTM A1010 steel and weathering steels (Fletcher F. B., 2011). The Moore Drive Bridge (FHWA No. 4443820) carries Moore Road over the Erie Canal and both directions of I-390. Exposure testing was conducted in the 2000s, at which time I-390 had an ADT of 39,004 vehicles, resulting in a total ADT of approximately 78,000 vehicles, and an ADTT of 9% in both directions. The minimum vertical underclearance was 15.5 feet, and the number of snowfall days is 81 days. ASTM A1010 coupons located on the bottom flange of the bridge's superstructure reportedly exhibited a corrosion rate of 0.58 mpy over 4 years of exposure (2005 to 2009) while weathering steel coupons exhibited corrosion rates of 2.03 or 2.43 mpy, depending on the steel grade. First, this bridge falls under Inferior Environment 2 as defined by McConnell et al. (2024), indicating that the environment is severe for uncoated weathering steel, and the corrosion rates of the weathering steel coupons confirm that the environment is in fact unsuitable for uncoated weathering steel. Second, the reported corrosion rates indicate that in a severe deicing chemical environment where uncoated weathering steel is expected to experience accelerated corrosion rates, ASTM A1010 steel may be expected to last approximately 4 times as long as weathering steel.

Table 49. Relative corrosion resistance of ASTM A1010 steel compared to weathering steel based on marine exposure testing by Fletcher, Townsen, and Wilson (2003).

Site	No. of Years of Exposure	Ratio of Weathering Steel Corrosion Rates to ASTM A1010 Corrosion Rates (mpy/mpy)
Kure Beach, NC, 200-meter lot	2 years	10 to 12
Kure Beach, NC, 200-meter lot	4 or 5 years	14 to 16
Kure Beach, NC, 25-meter lot	2 years	9 to 11
Kure Beach, NC, 25-meter lot	4 or 5 years	4 to 5

Further testing to assess the corrosion resistance of ASTM A1010 steel in severe deicing chemical environments that are marginally suitable or unsuitable for weathering steel, particularly at additional locations that may experience locally aggressive exposure conditions and accelerated corrosion, should be conducted to better understand the suitability of this steel for such environments and pre-emptively identify any corrosion protection needs beyond the inherent corrosion resistance of the material. The Iowa DOT has one weathering steel bridge in which two girders were constructed with ASTM A1010 steel instead of weathering steel (FHWA No. 053651). Constructed in 2016, the superstructure general condition rating was decreased to a 7 in 2022 because tape testing indicated the patina on the undersides of the bottom flanges of the weathering steel girders crossing over traffic was of limited effectiveness. Correspondingly, in the element-level condition data for the weathering steel protective coating (Element 851), 85% of the area of patina on these girders was decreased to condition state 2 due to oxide film degradation (Defect 3430). In contrast, no deterioration of the ASTM A1010 steel elements has been identified in the NBI reports thus far.

This Iowa bridge has the potential to assist in identifying unanticipated causes of poor performance of ASTM A1010 steel in real-world bridge environments and in characterizing field corrosion rates of ASTM A1010 in environments that are relatively severe to uncoated weathering steel, as evidenced by the poorly performing patina of the adjacent weathering steel girders. However, the NBI and element-level data currently available for this bridge does not contain the data needed to predict appropriate corrosion rates for ASTM A1010 steel in deicing chemical environments. Therefore, the 0.58-mpy corrosion rate from the Moore Drive Bridge in Rochester, New York was used to identify the one-sided corrosion allowances needed to achieve a 50-, 75-, 100-, 125-, and 150-year life. The necessary one-sided corrosion allowances are shown in Table 50.

Table 50. Expected corrosion loss of ASTM A1010 steel in a severe deicing chemical environment marginally suitable for uncoated weathering steel for a range of service life targets.

No. of Years	Expected One-Sided Corrosion Loss of ASTM A1010 Steel
50 years	29 mils
75 years	43.5 mils
100 years	58 mils
125 years	72.5 mils
150 years	87 mils

6.2.4.7 Performance of Duplex Stainless Steel

The strong passivation behavior of stainless steels and their very low passive corrosion rates leads many bridge-oriented practitioners to assume that stainless steel exposed to atmosphere will experience negligible metal loss for the life of the structure. While stainless steels do not experience meaningful general corrosion and consequently a sacrificial steel thickness is not necessary, stainless steels are at greater risk of aggressive localized corrosion due to pitting corrosion and, by extension, crevice corrosion, and duplex stainless steels in particular are at risk of local loss of corrosion resistance due to welding.

6.2.4.7.1 Risk of Pitting Corrosion of Duplex Stainless Steel Bridges

Pitting corrosion may simply be an aesthetic concern due to its unsightliness or can be a structural concern when the pitting is severe enough and at a location along the superstructure where loss of capacity cannot be tolerated. Different grades of stainless steel have different levels of resistance to pitting corrosion, which is commonly predicted by calculating the Pitting Resistance Equivalent Number (PREN) based on the chemical composition of the stainless steel. The PREN is typically based on the chromium (Cr), molybdenum (Mo), and nitrogen (N) contents of the stainless steel, although some formulas for the PREN of super duplex stainless steels include the element tungsten (W). The most commonly used formula for the PREN is shown in Equation 5:

$$PREN = Cr + 3.3Mo + 16N \qquad \text{Equation 11}$$

Other formulas may use different factors for Mo and N (BSSA, n.d.).

The PREN is useful for comparing the relative resistance of various grades of stainless steel to pitting, but has a limited role in the selection of the grade of stainless steel. The PREN has not been tied to specific service applications and a "suitable PREN range" for given environments does not exist. While the PREN is a useful indicator, guidance usually focuses on directly identifying the stainless steel grades suited for an environment. For example, the Nickel Institute provides guidance for selecting between Grades 316, 304, and 430 and "highly alloyed" stainless steels for architecture and building construction in various atmospheric environments. The guidance acknowledges that the surface finish prescribed by the designer and the commitment, or lack of commitment, of the owner to regular maintenance washing impact performance and therefore can impact the material selection (Nickel Institute, 2014). The significance of factors other than steel chemical composition, which is the only consideration of the PREN, is one of the reasons why the PREN has limited utility in material selection.

The Virginia DOT has begun investigating the feasibility of using duplex stainless steel for plate girder bridges and identifying the resources and construction practices needed for its successful implementation (Provines, Sharp, Ozbulut, & Daghash, 2019). Their assessment of the corrosion resistance of duplex stainless steels consisted of: (1) calculating the PRENs of potential grades of duplex stainless steels and comparing them to the pitting resistance of other available bridge steels, particularly ASTM A1010 steel, since PRENs for carbon and weathering steels is meaningless; and (2) reviewing literature for stainless steel corrosion rates according to long-term exposure coupon testing, in which two marine exposure studies that included Type 316 and Type 410 stainless steel coupons, neither of which are duplex grades, were identified and their general corrosion rates were presented. As expected, the general corrosion rates of the stainless steel coupons were negligible, i.e., on the order of 0.01 mils/year or less. Beyond calculating PRENs, the study did not address the risk of pitting corrosion in bridge environments, whether marine or deicing chemical, where the steel will be sheltered from natural washing due to rain, which the coupons in the marine exposure testing experienced. Lack of regular washing, either by rain or maintenance personnel, allows chlorides to build up to greater concentrations and consequently leads to much more aggressive environmental exposure than the general exposure of the site, and this phenomenon needs to be considered when selecting the grade of stainless steel.

While the PREN and laboratory testing can help inform a designer of the resistance of a grade of stainless steel to pitting, documented field performance monitoring is still needed to verify the suitability of the grade for the environment under consideration. Currently, duplex stainless steel is known to have been selected for use in two bridges in the United States: (1) as the primary structural members of the Harbor Drive Pedestrian Bridge in San Diego in 2011, because of the marine environment; and (2) as the structural steel hangers used in the West 7th Street Bridge in Fort Worth, Texas in 2013, because of the corrosion resistance of the steel although this bridge is not located in a marine environment nor a heavy deicing chemical environment. Unfortunately, performance monitoring of these bridges is of limited usefulness to designers of common roadway bridges because both structures have singular designs. The Harbor Drive Pedestrian Bridge is a single-cable, self-anchored suspension bridge with a curved deck suspended along one side (Iconic pedestrian bridge features stainless steel, 2012). The West 7th Street Bridge is a precast concrete network arch bridge (IMOA, 2018). In both cases, the duplex stainless steel elements are exposed such that they experience regular washing by rain instead of sheltered from rain. The performance of these bridges consequently cannot be used to better understand how duplex stainless steel will perform as I-beam elements or box girders sheltered under bridge decks, even ignoring the differences in the general chloride exposure of San Diego, California; Fort Worth, Texas; and locations across Iowa.

The use of duplex stainless steel for primary bridge superstructure elements has been explored to a greater extent in Europe, and other nations including Singapore and Australia have also constructed duplex stainless steel bridges (Ault & Dolph, 2018). With respect to grade selection, guidance for the selection of appropriate grades of stainless steel is included in Annex A of EN 1993-1-4, *Eurocode 3: Design of Steel Structures-Part 1-4: General Rules-Supplementary Rules for Stainless Steels*. With respect to field performance verification, Mameng et al. (2018) conducted inspections of seven duplex stainless steel bridges between 4 and 12 years of age located across Sweden, Spain, and the United Kingdom to assess their performance. Cosmetic micro-pitting was observed on only one bridge, specifically sheltered beams belonging to a bridge in Sweden located less than 0.25 km (0.16 miles) from the coast and adjacent to an oil refinery. The researchers noted that the duplex stainless steel was of a lower grade than that recommended by EN 1993-1-4 Annex A for the environment, but that the pitting was not impacting the structural performance and as such the duplex stainless steel was performing adequately. Overall, the researchers concluded that the study validated the assumption that duplex stainless steel requires little maintenance (Mameng, Backhouse, McCray, & Gedge, 2018).

While the United States can leverage the guidance of and lessons learned in other nations, the field performance study by Mameng et al. (2018) and others like it will have limited applicability for developing expectations of the performance of duplex stainless steel bridges in the United States. On the one hand, Mameng et al. (2018) investigated bridges with duplex stainless steel elements that were sheltered from washing by rain, which would be expected for many designs in the United States. On the other hand, the chloride exposure from deicing chemicals that was experienced by the studied bridges is expected to be different from that encountered across the United States and particularly in Iowa. Chlorides from deicing chemicals was only acknowledged for one bridge, and in the context of splash from an underlying road. Differences between the bridge design and maintenance practices of the states and those of European and other nations are particularly expected to cause differences in the "vulnerabilities" of the superstructure, i.e., which locations may be subject to relatively aggressive conditions.

The performance of duplex stainless steel superstructures and their maintenance needs in Iowa will be governed by their durability within aggressive microclimates found on superstructures within Iowa.

Currently, there is not enough field data to predict the suitability of the available grades of duplex stainless steels for bridge superstructures in Iowa, specifically their performance at locations identified as having the potential to experience relatively aggressive exposure conditions.

6.2.4.7.2 Risk of Local Loss of Corrosion Resistance Caused by Welding

Duplex stainless steels can experience loss of their corrosion resistance at welds because of changes to their microstructures during the welding process. This risk was identified by the Virginia DOT during their evaluation of the constructability of duplex stainless steels (Provines, Sharp, Ozbulut, & Daghash, 2019). The Virginia DOT noted that structural duplex stainless steel construction has a long history in the United States in engineering applications outside of the bridge industry such that local expertise in welding of duplex stainless steel is expected. Further, guidance for welding stainless steels is available. However, welders that work primarily in bridge construction are not expected to have expertise in welding duplex stainless steel, and experts in the stainless steel community reportedly mentioned that the use of a specialized stainless steel fabricator with pre-existing expertise in welding duplex stainless steels would likely be advantageous (Provines, Sharp, Ozbulut, & Daghash, 2019). In WJE's experience, despite the long history of duplex stainless steel construction in the United States, the loss of corrosion resistance due to welding remains a common cause of failure of duplex stainless steel structures.

6.2.5 Recommendations

The goal of this work has been to identify the steel superstructure designs and corrosion protection strategies that will minimize the need for maintenance of steel superstructures in Iowa in the first 50 years of service. As described previously in Section 5.1, Target Service Life Design Approach for Alternative Element Designs, the risk of maintenance needs at an undesirably young age is assumed to decrease as the service life for which the structure is designed from a durability perspective increases. As a result, target service lives of 75 years (Normal), 100 years (Enhanced), and 125 years (Maximum) are considered in this report.

In the context of steel superstructure corrosion, the service life of the steel superstructure is considered to be the time at which the factored capacity of the steel elements can no longer meet the factored design loads because of corrosion loss. The extent of corrosion loss at which this occurs is assumed to be the design sacrificial thickness, as discussed in Section 6.2.4.2.2, Corrosion Rates of Weathering Steel in Suitable Sites, in the analysis of the expected performance of uncoated weathering steel in Iowa. This definition and criterion are not necessarily suitable for stainless steel superstructures, but no alternative definition and criterion were developed because stainless steel superstructures are currently limited to bridge sites with unique performance requirements and designed on a case-by-case basis. The analysis presented above was conducted in order to identify the steel superstructure designs predicted to provide the target service lives with minimal maintenance in the first 50 years of service, based on current understanding of their performance.

The Corrosion Protection Strategy Selection Table (provided as Table 51) includes recommended corrosion protection strategies to meet the target service life of each category and minimize maintenance needs within the first 50 years of service within the atmospheric environments encountered across Iowa, when used in conjunction with good design detailing and joint elimination practices. Supporting details that can aid in understanding the table and its development are presented in Section 6.2.5.2, Supporting

Discussion. Additional actionable recommendations for the Iowa DOT pertaining to corrosion protection of steel superstructures are provided in Section 6.2.5.3, Additional Recommendations.

6.2.5.1 Corrosion Protection Strategy Selection Table

Table 51 is limited to deterioration caused by various forms of corrosion; other deterioration mechanisms or risks, such as fatigue cracking, are not considered. The designs are characterized by:

1. The type of bridge steel, i.e., weathering steel (WS), or ASTM A709 Grade 50CR steel (50CR). Duplex stainless steels (DSS) are not included in the table because their performance in Iowa cannot be predicted at this time based on current bridge literature. Carbon steels (CS) are also not included in the table because they are assumed to require regular maintenance.
1. The sacrificial steel thickness (SST), typically either 1/16 inch or 1/8 inch, understood to apply to all elements in the steel superstructure and the entire structural shape (if structural shapes are used) unless stated otherwise, e.g., bottom flange only.
2. The general type of coating, i.e., galvanizing (G), metallizing (M), or a duplex coating system (D). Stand-alone paint systems are not included in the table because they are expected to require at least one round of spot repair and touch up maintenance within 50 years, even in the least severe exposure environments in Iowa.
3. The extent of the coating, i.e., partial coating to the limits currently specified by Iowa DOT standard practice (PC-STP) or full coating (FC).

The Corrosion Protection Strategy Selection Table does not consider the availability of materials in Iowa. Further, it includes some technologies unlikely to be ready for widespread implementation in Iowa, either because they are unfamiliar to Iowa or generally experimental in the United States at this time. Specific technologies include ASTM A709 Grade 50CR steel; duplex coating systems; and galvanizing or metallizing of corrosion-resistant steels. The designs in the table are therefore recommended to the Iowa DOT for consideration, and the table is expected to: (1) define reasonable performance expectations for and the limitations of the designs and corrosion protection strategies currently available to the Iowa DOT; and (2) aid the Iowa DOT in choosing the technologies and practices to invest in and develop by identifying their ability to provide desirable durability.

The Corrosion Protection Strategy Selection Table also does not consider options in which steel elements are preserved through frequent maintenance and regular replacement of their protective coatings. In theory, this strategy could slow the deterioration of the structural elements to a negligible rate. However, such regular maintenance is challenging to implement in practice due to limited resources and understood to be in conflict with the goal of this project to design for minimum maintenance.

Table 51. Corrosion Protection Strategy Selection Table: Type of steel, sacrificial steel thickness, and general type and extent of coating(s) expected to achieve target service lives for steel superstructures in general exposure environments across Iowa with minimum maintenance needs.[1,2,3]

General Exposure Environment	Exposure Subcategory	Normal Service Life Category[4]	Enhanced Service Life Category[4]	Maximum Service Life Category[4]
C2 (Low)[5]	All other sites	WS, SST of 1/16", PC-STP using G, M, or D.	WS, SST of 1/16", PC-STP using G, M, or D.	WS, SST of 1/16", PC-STP using G, M, or D.
	Low-Level Water Crossings[6]	—	—	—
C3 (Moderate)[5,7]	All other sites	WS, SST of 1/16", PC-STP using M or D.	WS, SST of 1/16", PC-STP using M or D.	WS, SST of 1/16", PC-STP using M or D.
	Low-Level Water Crossings[6]	—	—	—
C4 (High)[5]	All other sites	WS, SST of 1/16", PC-STP using D.	WS, SST of 1/8", PC-STP using D.	WS, SST of 1/8", PC-STP using D.
	Inferior Environments (Deicing)[8]	**Option 1.** 50CR, SST of 1/8", PC-STP using D. **Option 2.** 50CR, SST of 1/16", FC using G or M and PC-STP using D.[9,10] **Option 3.** WS, SST of 1/8", FC using M and PC-STP using D.[9,10] **Option 4.** WS, SST of 1/8", FC using D.[10]	**Option 1.** 50CR, SST of 1/8", PC-STP using D. **Option 2.** 50CR, SST of 1/16", FC using M and PC-STP using D.[9,10] **Option 3.** 50CR, SST of 1/16", FC using D.[10]	**Option 1.** 50CR, SST of 3/16", PC-STP using D. **Option 2.** 50CR, SST of 1/8", FC using G or M and PC-STP using D.[9,10] **Option 3.** 50CR, SST of 1/8", FC using D.[10]
	Low-Level Water Crossings[6]	—	—	—
	Industrial Sites	—	—	—

Notes: [1]*Abbreviations are as follows: WS = weathering steel. 50CR = ASTM A709 Grade 50CR steel. SST = sacrificial steel thickness. G = galvanizing. M = metallizing. D = duplex coating. Partial coating to the limits specified by Iowa DOT standard practice = PC-STP. Full coating = FC.*

[2]*A "—" indicates that a corrosion consultant should be engaged to aid in selecting a design that will meet the goals of the service life category with minimum maintenance needs.*

[3]*The coatings included in this table were selected to minimize the likelihood of coating maintenance within the first 50 years of service, but are expected to require maintenance and potentially removal and replacement within the target service life of the superstructure. While the table provides guidance as to the type of coating system that should be selected, not all coatings within the coating category will provide the desired performance and the user should be selective when choosing a specific coating system within the recommended category.*

[4]*The target service life is 75 years for the Normal service life category, 100 years for the Enhanced service life category, and 125 years for the Maximum service life category.*

[5]*As defined in Section 6.2.4.1.1.*

[6]*As defined in Technical Advisory 5140.22 (FHWA, 1989). Wherever possible, the clearance needed to avoid this classification should be incorporated into the design.*

[7]*Galvanizing is not identified as a potential coating for bridges in this general exposure environment because the ability to achieve the galvanizing thickness needed to avoid maintenance for 50 years at locations of the superstructure with relatively aggressive exposure conditions depends on many project-specific variables. However, users may consider conducting galvanizing trials on a project-by-project basis to identify the achievable galvanizing thickness expected to be achieved and evaluate whether or not that thickness is sufficient to meet the goal of minimum maintenance for 50 years.*

[8]*Defined as the inferior environments identified in Table 35 based on the work of McConnell et al. (2024) and environments considered too aggressive for uncoated weathering steel by the Iowa DOT as described in Section 6.2.3.1, Type of Steel (Iowa DOT, July 2024). Where possible, these environments should be avoided by providing greater minimum vertical underclearance.*

[9]*The intent of the proposed coating system is to provide a metallic coating across the entire surface area of the superstructure, and then apply a paint system on top to form a duplex coating system at the locations susceptible to more aggressive exposure conditions as described by current standard Iowa DOT practice.*

[10]*Both a full coating and a sacrificial steel thickness are included because it is assumed that the full coating will not be maintained. If the full coating is maintained such that the corrosion experienced by the steel elements has a negligible impact on the ability of the superstructure to provide the required capacity, then the sacrificial steel thickness is not needed.*

6.2.5.2 Supporting Discussion

The following is a step-by-step discussion of how the recommendations of Table 51 were developed.

6.2.5.2.1 Capabilities of Uncoated Weathering Steel with 1/16-Inch Sacrificial Steel Thickness

Starting with the materials and practices with which the Iowa DOT has experience, uncoated weathering steel with a sacrificial steel thickness of 1/16 of an inch is expected to lose its sacrificial steel thickness in the time frames shown in Table 52.

Table 52. Expected time to consumption of 1/16 of an inch of uncoated weathering steel in general exposure environments found in Iowa where weathering steel will form an effective patina.

General Exposure Environment	Time to Consumption of 1/16 in. of Weathering Steel	Justification
C2 (Low)	>200 years	C2 (Low) exposure is expected to correspond to the lower bound of corrosion loss for uncoated weathering steel with an effective patina, described in Equation 8.
C3 (Moderate)	140 years	Corrosion loss in C3 (Moderate) exposure is assumed to be represented by the midpoint between the lower bound described by Equation 8 and the upper bound described by Equation 9. The assumed first-year corrosion loss is subsequently 2.05 mils and the assumed long-term corrosion rate is 0.21 mpy.
C4 (High)	95 years	C4 (High) exposure is expected to correspond to the upper bound of corrosion loss for uncoated weathering steel with an effective patina, described in Equation 9.

Therefore, uncoated weathering steel is expected to provide the Maximum target service life (125 years) in C2 (Low) and C3 (Moderate) environments, and the Normal target service life (75 years) in C4 (High) environments, provided there are no site features or conditions that make it unsuitable for weathering steel.

6.2.5.2.2 Capabilities of Coatings

The performance described in Section 6.2.5.2.1, Capabilities of Uncoated Weathering Steel with 1/16-Inch Sacrificial Steel Thickness, is limited to the area of the superstructure that experiences the general exposure environment, and a different strategy is needed at the locations of the superstructure with locally aggressive conditions. Table 51 relies on steel protective coatings, which are handled differently from the steel elements with sacrificial steel. While steel protective coatings can have long lives, particularly metallic coatings and duplex coating systems, such long lives generally require regular coating maintenance, which conflicts with the goal of this project to design for minimum maintenance. Further, while long life is predicted for metallic coatings by zinc corrosion rates and for duplex coating systems by the model proposed by van Eijsbergen (1994), experience indicates that the time-to-maintenance of metallic and duplex coatings can be relatively short compared to these predictions. Therefore, the coatings recommended in Table 51 are selected based on their likelihood of avoiding maintenance for 50 years in the expected service environment. The recommended coatings themselves are not necessarily expected to be able to reach the target service lives, even with coating maintenance. Coating maintenance and removal and replacement will likely be needed within the target service life for the steel superstructure to avoid rapid deterioration of the steel elements themselves at locations with locally aggressive conditions and their premature replacement.

Table 53 identifies the exposure environments in which the four categories of coatings, i.e., paint systems, galvanizing, metallizing, and duplex systems, may be expected to have a time to first maintenance of at least 50 years. This is summarized as follows:

- No paint systems are expected to be able to provide 50 years of service without needing at least one spot repair and touch up maintenance.
- Galvanizing is expected to be able to provide at least 50 years of service before needing maintenance in C2 (Low) exposures based on the estimated time to consumption of galvanizing in Table 43 and time to first maintenance of galvanizing in Table 44. Galvanizing may also be expected to provide such performance in C3 (Moderate) exposures, although its performance with respect to the desired time to maintenance may be marginal. The galvanizing on structural shapes and plates is expected to meet the target time to maintenance, but based on the galvanizing thicknesses expected for fasteners, washers, and other hardware, the galvanizing at connections may begin to need maintenance at approximately 40 years. Identifying galvanizing as likely requiring maintenance within 50 years in C3 (Moderate) exposure was deemed too conservative because: (1) the galvanizing thicknesses assumed for hardware are based on specified minimums and greater thicknesses may be achieved, and (2) this applies to a relatively small area of the entire superstructure. But to reduce this risk, consideration of a topcoat over galvanized connections is recommended.
- Metallizing is expected to be able to provide at least 50 years of service life before needing maintenance in C2 (Low), C3 (Moderate), and C4 (High) exposures, primarily because it can be assumed to be built to a much greater thickness than galvanizing. Table 45 and Table 46 identified the minimum metallizing thicknesses needed for various times to consumption or maintenance, respectively, based on zinc corrosion rates, and the thicknesses were considered achievable. While metallizing can be expected to have greater corrosion rates because of its porosity, metallizing can also have lesser corrosion rates because it commonly is a zinc-aluminum alloy. Since the actual corrosion rate of metallizing compared to that of zinc coupons is not well understood, the maintenance and service life expectations are based on zinc corrosion rates, and Table 53 clarifies that the 85/15 Zn/Al alloy and a sealer are both assumed to be used.
- Duplex coatings are expected to be able to provide at least 50 years of service life before needing maintenance in C2 (Low), C3 (Moderate), C4 (High), and C5 (Very High) exposures, provided a suitable metallic coating and paint system combination is selected. Not all duplex systems are suitable; for example, the 4-mil coat of thermally sprayed zinc with 4 coats of alkyd-based paint used in Norway did not demonstrate a 50-year time to maintenance in C5 (Very High) exposure (Knudsen, Matre, Dorum, & Gagne, 2019). However, based on the expected lives of galvanizing and a 2-coat paint system consisting of a surface tolerant epoxy and a polyurethane topcoat in “Severe (heavy industrial)/C5” exposure, showed in Table 48, and the model proposed by van Eijsbergen (1994), a duplex coating system using these components could reasonably provide at least 50 years of service before needing maintenance.

Table 53. Types of coatings expected to be able to provide at least 50 years of service before needing maintenance in various exposure environments.[1]

Exposure Environment	Paint Systems	Galvanizing	Metallizing[2]	Duplex Coatings[3]
C2 (Low)	✘	✓	✓	✓
C3 (Moderate)	✘	✓[4]	✓	✓
C4 (High)	✘	✘	✓	✓
C5 (Very High)	✘	✘	✘	✓

Notes: [1] ✓ *indicates that coatings in the identified category can be expected to have a time to first maintenance of at least 50 years in the given exposure environment, provided a suitable zinc thickness is selected (in the case of metallizing) or a suitable combination of coating systems is selected (in the case of duplex coatings).* ✘ *indicates a time to first maintenance of at least 50 years is not considered reasonably achievable for coatings in the identified category for the given exposure environment.*

[2]*Metallizing is assumed to be 85/15 Zn/Al and have a sealer on top. Pure zinc metallizing will have a shortened life compared to 85/15 Zn/Al metallizing. Different minimum thicknesses may need to be selected for each environment to achieve the desired performance.*

[3]*Not all duplex coatings are expected to provide the level of performance indicated in the table. However, some combinations of metallic coatings and paint systems are predicted to have a time to first maintenance of greater than 50 years in each of the exposure environments identified.*

[4]*The galvanizing on structural shapes and plates is expected to have a time to first maintenance of at least 50 years in C3 (Moderate) exposure, but the galvanizing at connections may need maintenance at approximately 40 years if thicknesses greater than the minimum specified by ASTM A153 cannot be achieved. To reduce this risk, a topcoat over galvanized connections should be considered.*

Table 53 includes the exposure environment C5 (Very High), which has not been discussed previously in this section. C5 (Very High) is described qualitatively by ISO 9223 as "[t]emperate…atmospheric environment with very high pollution…and/or significant effect of chlorides, e.g. industrial areas, coastal areas, sheltered positions on coastline." It was considered too aggressive to represent general site exposure conditions in Iowa. However, as stated previously, protective coatings are expected to be applied at the locations on the superstructure that will experience relatively aggressive conditions locally. For the purposes of this analysis, the corrosivity of the locally aggressive conditions is assumed to be represented by the "general exposure environment" representing one greater level of severity than that expected at the bridge site. For example, for a bridge site with C2 (Low) general exposure conditions, corrosion rates and coating performance at locations with locally aggressive exposure conditions are assumed to align with those expected of C3 (Moderate) exposure. For a bridge site with C4 (High) general exposure conditions, corrosion rates and coating performance at locations with locally aggressive exposure conditions are assumed to align with those expected of C5 (Very High) exposure. This assumption is reflected in the protective coatings recommended in Table 51 for the various levels of general exposure severity.

6.2.5.2.3 Achieving Durability in C4 (High) Exposure and Inferior Deicing Environments

For "inferior" deicing chemical environments for uncoated weathering steel, McConnell et al. (2024) recommended increasing the sacrificial steel thickness of the bottom flanges to 1/8 of an inch. Before considering "inferior" environments and appropriate strategies for such environments, it is helpful to consider bridge sites in C4 (High) exposure with environments suitable for uncoated weathering steel. In suitable C4 (High) environments, 1/8 inch of sacrificial steel thickness is expected to be consumed in approximately 200 years. Therefore, uncoated weathering steel with a sacrificial steel thickness of 1/8 of an inch is expected to provide the Maximum target service life (125 years) in C4 (High) exposure environments, when combined with a suitable partial coating as discussed above.

Moving on to inferior deicing environments, such environments are assumed to be a relatively aggressive subcategory of C4 (High) exposure. Both the inferior environments identified by McConnell et al. (2024) and the set of criteria used by the Iowa DOT to identify bridge sites that are too aggressive for uncoated weathering steel because of chloride-contaminated splash from the underlying roadway are grouped under “Inferior Environments (Deicing)” in Table 51 because of their similarities. The corrosion rates of weathering steel coupons on the bottom flange of the Moore Drive Bridge in Rochester, New York experienced average corrosion rates of approximately 2 to 2.5 mpy. If these corrosion rates are assumed, then a 1/8-inch sacrificial steel thickness would be expected to be consumed in approximately 25 to 30 years in an inferior deicing environment.

Further increasing the sacrificial steel thickness of weathering steel to achieve Normal, Enhanced, or Maximum target service life in the C4 (High) exposure is assumed to be infeasible. Based on the corrosion rates described above, a 1/4-inch sacrificial steel thickness would still be too little to achieve even the Normal target service life (75 years).

One option that the Iowa DOT has begun to investigate is the use of ASTM A1010/ASTM A709 Grade 50CR steel. The average corrosion rate of steel coupons of this grade on the bottom flange of the Moore Drive Bridge was 0.58 mpy. Assuming this corrosion rate and two-sided corrosion, a sacrificial steel thickness of 1/16 of an inch would be consumed shortly after 50 years of service, a sacrificial steel thickness of 1/8 of an inch would be consumed shortly after 100 years of service, and a sacrificial steel thickness of 3/16 of an inch would be consumed after approximately 160 years of service. Uncoated Grade 50CR steel with a sacrificial steel thickness of 1/8 of an inch is, therefore, expected to provide the Normal target service life (75 years) and the Enhanced target service life (100 years) in inferior deicing environments, and uncoated Grade 50CR steel with a sacrificial steel thickness of 3/16 of an inch is expected to provide the Maximum target service life (125 years). A partial coating at locations of the superstructure expected to be subject to aggressive exposure conditions is assumed to remain necessary.

A potential alternative to using a more corrosion-resistant grade of steel, i.e., Grade 50CR, is to use a coating with no intent to maintain it. For example, if a 1/8-inch sacrificial steel thickness of weathering steel is expected to be consumed after approximately 25 to 30 years, then a full coating that will provide protection for 50 years before the weathering steel begins to see exposure would theoretically result in a design capable of meeting the Normal target service life (75 years) with minimal maintenance needs. Even in circumstances where the use of an unmaintained coating cannot replace the need for Grade 50CR steel, the use of a sacrificial metallic coating may still be economical because it could reduce the necessary sacrificial steel thickness, thereby resulting in a reduction in the weight of corrosion-resistant steel needed and the associated material cost.

Therefore, combinations of metallic coatings and corrosion-resistant steels are presented in Table 51, even though this combination is sufficiently unusual to be considered experimental at this time and requires further work before it can be implemented with confidence in its performance. The Virginia DOT has at least one galvanized weathering steel bridge, the Genito Road Bridge which was constructed in 2011

(Sharp, Provines, Moruza, Via, Jr., & Harrop, 2019), but similar material designs were otherwise not identified in the literature review. Preliminary concerns include the compatibility of the galvanizing process with the chemistry of corrosion-resistant steels and the ability of the corrosion-resistant steels to develop their protective patinas after the barrier and cathodic protection offered by the metallic coatings have both been exhausted.

6.2.5.2.4 Low-Level Water Crossings

A low-level water crossing where uncoated weathering steel would be an unsuitable option could potentially occur at a bridge site belonging to any of the atmospheric corrosivity categories. Low-level water crossings are assumed to have more aggressive conditions than average within each category because of their elevated humidities and assumed time-of-wetness. This increased moisture exposure is the reason that weathering steel does not form an effective patina in these environments, and could prevent zinc from developing a patina as well. The negative impact of increased moisture exposure on the formation and long-term integrity of the patina is further expected to increase with increasing chloride exposure.

The ability of Grade 50CR steel to form a patina in persistent moisture conditions has not been assessed. Given its classification as a “lean stainless steel” and the mechanism by which stainless steels passivate, persistent moisture conditions are not expected to compromise the ability of Grade 50CR steel to form an effective passivating layer, but this expectation of performance needs to be verified before Grade 50CR steel can be recommended for such environments. However, as with weathering steel and zinc, the presence of chlorides and increased moisture exposure will negatively impact the long-term performance of the patina.

Avoiding aggressive conditions is one of the best strategies for achieving long service lives, and this strategy is acknowledged in Table 51 for both low-level water crossings and inferior deicing environments. Because uncoated weathering steel performs similarly to carbon steel in low-level water crossings, the duration of the protection offered by metallic, zinc-based coatings may be greatly shortened in low-level water crossings, and the performance of Grade 50CR steel is too unknown in low-level water crossings, Table 51 does not provide pre-determined options for bridges in these types of exposure sites and recommends engaging a corrosion consultant. Future work that could be conducted to develop options for low-level water crossings in Table 51 is also presented in Section 6.2.5.3, Additional Recommendations.

6.2.5.2.5 Bridge Sites Exposed to Industrial Emissions

For bridges located near enough to industry that industrial emissions are expected to reach the bridge, the aggressiveness of the atmospheric exposure conditions and suitable corrosion protection strategy are expected to be case-specific. Therefore, Table 51 recommends engaging a corrosion consultant when bridge sites expected to be affected by industry are encountered. Such cases are expected to be rare.

6.2.5.3 Additional Recommendations

Additional recommendations to the Iowa DOT pertaining to improving the durability of steel superstructures are:

1. Consider alternative details for flange deflectors to avoid or mitigate the risk of crevice corrosion, e.g., “gluing” the plate in place instead of bolting.

2. Expand sacrificial steel thickness requirements to secondary members of the superstructure as much as feasible. Implementation of this recommendation may change little in practice because the available section sizes relative to the thicknesses necessary usually result in a large corrosion allowance that is expected to be greater than the design sacrificial steel thicknesses discussed; however, the knowledge that a design sacrificial steel thickness is included in the sizing provides a greater level of confidence that the risk of maintenance needs due to corrosion of these elements is mitigated. This recommendation should particularly be considered for bridge sites where the replacement of secondary members will be undesirable because of associated traffic control costs on the underlying roadway or other reasons.
3. Consider the use of paint systems with longer lives than the two-coat system consisting of an inorganic zinc primer and waterborne acrylic topcoat that is currently used by the Iowa DOT as standard practice.
4. Provide clarification for when galvanizing should be used for steel superstructures instead of a paint system according to the current practices of the Iowa DOT.

The following studies are also recommended to the Iowa DOT to address knowledge gaps identified by this review and improve confidence in the service life predictions for steel superstructures and their protective coatings:

1. Conduct field exposure testing and characterize the corrosivity of atmospheres at representative locations across Iowa to better understand the corrosion rates that should be expected of the grades of structural steel used or desired to be used by the Iowa DOT, as well as of galvanizing and metallized coatings.
2. Periodically conduct detailed inspection of Bridge No. 053651 including thickness measurements of the ASTM A709 Grade 50CR steel girders and the weathering steel girders to compare their performance in an environment that is aggressive towards weathering steel and quantify the corrosion rate of Grade 50CR steel in this environment.
3. Assess the need for a partial coating on ASTM A709 Grade 50CR steel under joints and at other locations where relatively aggressive exposure conditions may develop by comparing the corrosion rates of uncoated Grade 50CR steel at vulnerable locations of the superstructure with those across the general area of the superstructure.
4. Investigate the feasibility of galvanizing corrosion-resistant steels, specifically weathering steel and potentially ASTM A709 Grade 50CR steel, and the impact of galvanizing on the corrosion resistance of corrosion-resistant steels and their ability to form a patina when exposed.
5. Verify the minimum clearance over water features in Iowa, e.g., ranging from small creeks to wetlands, needed to avoid elevated humidities and times-of-wetness that will compromise the development of the patina on weathering steel, particularly if water crossings that classify as “low-level” based on Technical Advisory 5140.22 (FHWA, 1989) are difficult to avoid. Investigate the potential need for greater clearances due to flooding.
6. Investigate the feasibility of using metallizing with greater amounts of aluminum, or aluminum-based metallizing and identify the performance of thermally-sprayed aluminum in environments where a long-lived protective coating is desirable, such as at low-level water crossings or inferior deicing environments.

7. Verify the expected performance of duplex coatings through laboratory testing and field performance studies, and study the feasibility of construction of duplex coatings to identify the resources needed for their successful implementation.
8. If duplex stainless steel is of interest, conduct field exposure testing to assess the ability of potential grades of duplex stainless steel to resist pitting corrosion in bridge environments in Iowa, and identify the resources needed for welding of duplex stainless steel to be executed successfully.

6.3 Bridge Expansion Joints

Expansion joints are key elements in bridges that accommodate the superstructure expansion and contraction imparted by thermal loads, creep, and shrinkage. Properly designed and installed expansion joints serve to not only accommodate superstructure movement but also protect bridge superstructure and substructure elements from deterioration and corrosion drivers such as debris intrusion and water and deicer infiltration. The current state-of-practice provides a large variety of joints to choose from, each having their own advantages and disadvantages. A common concern when considering joints is water and debris leaking through the joint onto the superstructure and substructure, initiating and feeding the premature deterioration of its components. Joint types are described largely by their movement capacity. In practice, damage to bridge decks and approach headers in the vicinity of expansion joints is commonly reported and attributed to the use of poor materials or improper construction. It has been reported (Arora and Associates, P.C., 2019) that installation of expansion joints is generally the responsibility of the general contractor. While current practices leave joint selection to the bridge designer, it has been widely established that a simple more prescriptive selection and design procedure or design guide for bridge deck joints would be beneficial to bridge owners specifically governmental departments of transportation. An expansion joint that is easy to install, can accommodate prescribed movements, prevents leakage of surface runoff with deicing salts and chemicals, requires minimal maintenance, is cost efficient, and has a long service life is the ideal scenario for a bridge joint that would be selected and specified by a bridge designer.

This section will focus on the following bridge expansion joints, which encompass most common types of expansion joints in the United States that are currently in service or have been specified in the past:

- Pourable Seal Joints
- Compression Seal Joints
- Asphaltic Plug Joints
- Sliding Plate Joints
- Strip Seal Joints
- Finger Joints
- Modular Joints
- Semi-Integral Abutments (Jointless)
- Integral Abutments (Jointless)

Many factors affect the performance of bridge expansion joints during its service life and thus, a designer must assess the requirements of the structure including, but not limited to, the type of the structure, the projected vehicular traffic volumes, the magnitude and direction of movement, environmental conditions, skew of the bridge, and financial costs.

The highlighted expansion joints are classified and grouped into three main categories based on the maximum longitudinal movement capacity for each expansion joint; small, medium and large longitudinal movement. In addition, a fourth category is included representing a jointless bridge given the ongoing joint elimination trend that aims to minimize maintenance activities that are typically associated with a bridge joint. However, a jointless bridge can only be prescribed by a designer in certain situations. Several governmental agencies permit the design of bridges without the presence of bridge deck joints. Depending on the skew angle of the bridge, bridges with a steel or concrete superstructure can be designed with no joint up to 300 feet or 400 feet, respectively (Hassiotis, Khodair, Roman, & Dehne, 2006). In addition, each joint can be classified as either open or closed depending on if the joint by design allows water to flow through the joint opening. The following properties are discussed for each expansion joint:

- Description and Classification
  - Movement Limit and Range
  - Expansion Span Length
  - Skew Limit and Range
- Service Life and Performance
  - Life Expectancy
  - Common Issues and Failure Modes
  - Advantages and Disadvantages
- Maintenance Requirements
- Practical Considerations
- Construction and Maintenance Costs

The presented relative installation costs of the different expansion joints were estimated based on an average of available unit price data from recent year(s) online bid tabulations obtained from various states including Washington State, Illinois, California, Iowa, Missouri, and Indiana.

6.3.1 Small Longitudinal Movement Joints

Small longitudinal joints are joints that can be characterized as being capable of accommodating less than 4 inches of longitudinal movement (Shenton III & Mertz, 2016). Defining small longitudinal movements as 4 inches or less is a threshold that the authors selected based on average values present in literature. The most typical and/or commonly utilized joints in this category are pourable seals, compression seals, asphaltic plugs, and sliding plate joints as highlighted in the following sections.

6.3.1.1 Pourable Seal Joints

Description and Classification. Pourable seal joints, a type of field-formed joint, are closed expansion joints that typically consist of a viscous adhesive and pourable waterproof silicone installed with backer rods to prevent the sealant from flowing down into the joint. The most common seal material utilized is either silicone and/or polyurethane (Baker Engineering & Energy, 2006). Pourable seal joints are traditionally used on shorter spans where the joint movement is 3/16 inch or less (National Cooperative Highway Research Program, 2003). However, newer systems are suggested by manufacturers for larger movements depending on the sealant material (National Cooperative Highway Research Program, 2003). Manufacturers allow installment for skews up to 45 degrees; however, some states limit installment at a max skew of 30 degrees or less.

Figure 77. Pourable joint on US 62 over Mississippi River that has become an open joint

Service Life and Performance. The service life expectancy of a pourable seal joint ranges between 5 to 10 years (D.S. Brown, 2016) depending on various parameters including initial construction workmanship, selected manufacturer sealant product, joint nosing material selection, environmental factors, truck traffic volumes, and maintenance protocols. According to a national survey of bridge owners and agencies (Baker Engineering & Energy, 2006), the estimated average service life of pourable seal joints is 11.50 years. This average is based on survey responses from 25 state agencies and two Canadian provinces. It was reported (Baker Engineering & Energy, 2006) that the estimated service life of pourable joints and compression seals are comparable while the cost of poured seals is only a fraction of compression seals. However, in another survey (Chang & Lee, 2001) in the state of Indiana and neighboring states, the estimated average service life of pourable joints is 5.56 years. The state of Arizona Department of Transportation has found favorable results with the practice of replacing compression seal joints that have failed in service with pourable seal joints (Baker Engineering & Energy, 2006). On the other hand, Montana Department of Transportation has reported that pourable silicone joints have performed very poorly in Montana and should be avoided, when possible, in rehabs and never used in new construction. Common failure modes of pourable seal joints include bond failures, sealant rupture, and traffic impact if the joint is poured too high. A summarized list of advantages and disadvantages for pourable seal joints is presented below.

Advantages:

- Repels water
- Easy to use (self-leveling)
- Can be installed using standard tools at all temperatures Good weatherability
- Fast curing (Baker Engineering & Energy, 2006)
- Easy and straightforward installation and replacement (Chang & Lee, 2001)
- Good candidate for rehabilitation and replacement in situations where extended traffic lane closure is not acceptable (VTrans, 2009)

Disadvantages:

- Prone to debonding of the sealant
- Can only be installed during dry conditions
- Not recommended for environmental conditions where continuous water moisture is expected (Baker Engineering & Energy, 2006)
- Multiple state departments of transportation reported nosing material frequently damaged causing leaking problems (Chang & Lee, 2001)
- Only applicable for small longitudinal movements

Maintenance Requirements. To ensure the longevity of a pourable seal joint, it is essential to perform annual cleaning and flushing. This preventative maintenance helps remove accumulated debris, thereby preventing premature joint failure. Depending on the truck traffic volumes, the joint nosing material would need to be replaced when damaged from cyclic impact. Nonetheless, respondents to a survey conducted by Chang and Lee (2001) indicated preferring this type of joint because of its easy installation and repairs; this type of joint can be repaired by bridge crews within one working day. Patch repairs can also be performed without removing the entire joint.

Practical Considerations. Pouring of joint sealant yields the best results when poured during an ambient temperature that is in the middle of the expected temperature range. It is recommended that steel armor be specified for new designs in conjunction with poured joints to protect the concrete deck headers from edge spalling, prolonging the service life of the joint. Additionally, the use of elastomeric concrete or other shock-absorbing embedment materials around the anchorages is recommended (Guthrie, 2005). The substrate should be prepared and inspected during initial installation and during repairs. The success of the joint depends on the workmanship of the installer, particularly in cleaning and preparing the faces of the concrete and ensuring that the seal is recessed below the top of the roadway to avoid impact from vehicular traffic (Arora and Associates, P.C., 2019).

Construction and Maintenance Costs. The relative installation cost for pourable joints with units of measurement of linear feet (LF) is estimated at a minimum cost of $45 per LF, maximum cost of $157 per LF, and a mean cost of $91 per LF. This unit price only includes the cost for installation of the silicone seal. Installation of the associated polymer concrete nosing for the joint is estimated at a minimum cost of $62 per LF, maximum cost of $625 per LF, and a mean cost of $280 per LF. The total arithmetic sum becomes a unit price at a minimum cost of $107 per LF, maximum cost of $782 per LF, and a mean cost of $371 per LF. If damage to the seal of a pourable seal joint is evident then replacement cost would be equivalent to initial construction costs, plus existing seal removal and mobilization costs.

6.3.1.2 Compression Seal Joints

Description and Classification. Compression seal joints are closed expansion joints consisting of cellular foam impregnated seals, most often neoprene, that are compressed when inserted into the joint gap opening and remain in a state of compression during all movement phases of the joint (Baker Engineering & Energy, 2006). Compression seals can be installed against the smooth vertical concrete faces within the joint gap opening; however, steel armor could be added to protect the concrete edge from spalling and provides enhanced impact resistance. In addition, special nosing material may be utilized to further enhance impact resistance within the joint blockout. The seals are held in place primarily by friction against the adjacent vertical joint faces. Some manufacturers formerly detailed their compression seals for a skew of up to 45 degrees (D.S. Brown, 2016); however, more recent guidance stipulates that compression seals are not recommended on skewed angles over 15 degrees (D.S. Brown, 2021). The 2023 Iowa DOT Bridge Design Manual mentions compression seals are rated for 2 to 3 inches of longitudinal movement; however, for new bridges, the state has more recently eliminated the use of compression seals in favor of strip seals (Iowa DOT Bridges and Structures Bureau, 2024).

Figure 78. Compression seal joint in SR 46 Eastbound over East Fork White River

Service Life and Performance. The service life expectancy of a compression seal joint ranges between 10 to 15 years (D.S. Brown, 2016) depending on various factors including initial construction workmanship, ambient environment conditions and maintenance protocols. According to a national survey of bridge owners and agencies (Baker Engineering & Energy, 2006), the estimated average service life of compression seal joints is 12.65 years. This average is based on survey responses from 25 state agencies and two Canadian provinces. In another survey conducted (Chang & Lee, 2001) in the state of Indiana and neighboring states, the estimated average service life of compression seal joints is 10.3 years. Compression seal joints are prone to fall out or pop out from the joint due to incorrect sizing or due to failure of the adhesive, which results in an open joint. In addition, compression seals that pop out from the joint above the elevation of the driving surface pose a hazard to vehicular traffic and the public. If the compression seal is oversized for the bridge joint opening, the seal bulges above the surface of the deck resulting in impact damage and a leaky joint. If the compression seal is undersized for the opening, the seal will lose compression and fall out (Baker Engineering & Energy, 2006; National Cooperative Highway Research Program, 2003). The survey conducted by Baker Engineering & Energy found that 11 out of 27 reported governmental agencies discontinued the use of compression seals. In addition, 18 of the 27 agencies reported a problem with compression seal joint failure related to loss of compression during service due to a variety of reasons. In general, service life issues that have arisen with compression seal joints include leakage after installation, falling out/loss of compression or bulging out onto the driving surface over time. Leakage after installation can most typically be attributed to inappropriate sizing and poor workmanship during installation. A summarized list of advantages and disadvantages for compression seal joints is presented below.

Advantages:

- Relatively easy to install and replace (Chang & Lee, 2001; Baker Engineering & Energy, 2006)
- Minimal maintenance is required during the joint service life
- Joint armor is not required making it less labor intensive replacement option than strip seals

Disadvantages:

- Prone to leakage (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014)
- Prone to bulge out or fall out from joint gap opening
- Not recommended for extreme temperature swings (Baker Engineering & Energy, 2006; National Cooperative Highway Research Program, 2003)
- Only applicable for small longitudinal movements

Maintenance Requirements. To ensure the longevity of a compression seal joint, it is essential to perform annual cleaning and flushing. This preventative maintenance helps remove accumulated debris, thereby preventing premature joint failure.

Practical Considerations. It is recommended that steel armor be installed in conjunction with the joint to protect headers from edge spalling. Many failures in compression seals over the years have led to a decrease in their usage. It has been reported that Minnesota and Louisiana have both stopped the use of compressional seal joints due to their poor performance (Arora and Associates, P.C., 2019). Moreover, both Louisiana and Colorado report unreliable performance of compressional seal joints (National Cooperative Highway Research Program, 2003). Meanwhile, other states, such as New York and Illinois, report compression seals being effective and requiring minimal maintenance (National Cooperative Highway Research Program, 2003). Discrepancies are likely due, in part, to the wide range of products that can be categorized as compression seals, and their varying effectiveness.

Construction and Maintenance Costs. The relative installation cost for compression seal joints with units of measurement of linear feet is estimated at a minimum cost of $67 per LF, maximum cost of $321 per LF, and a mean cost of $156 per LF. The unit price only includes the cost for the installation of a compression (preformed joint) seal. Installation of any associated polymer concrete nosing for the joint is estimated at a minimum cost of $62 per LF, maximum cost of $625 per LF, and a mean cost of $280 per LF. The total arithmetic sum becomes a unit price at a minimum cost of $128 per LF, maximum cost of $946 per LF, and a mean cost of $435 per LF. If damage to compression seal joints is evident then replacement costs of the seal would be equivalent to initial construction costs, plus existing seal removal and mobilization cost, in the case of joints with no steel armor. Where steel armor is present, there is a reduced probability of replacement as the joint headers are protected and no impact resistant concrete nosing would be required either.

6.3.1.3 Asphaltic Plug Joints

Description and Classification. Asphaltic plug joints are closed bridge expansion joints that consist of a hot applied, often polymer modified, asphalt placed in the expansion joint over a steel bridging plate. The system is often used during the construction of deck overlays (National Cooperative Highway Research Program, 2003). Asphaltic plug joints cannot accommodate long spans, with typically a maximum span limit of 100 ft, and with reduced functionality in skewed bridges. A system manufacturer for asphaltic plug joints reports that the joint can be installed up to a 30 degree skew (D.S. Brown, 2020). The longitudinal movement range of asphaltic plug joints is up to 1.5 inches.

Service Life and Performance. The service life expectancy of an asphaltic plug joint ranges between 2 to 5 years (D.S. Brown, 2016) depending on various parameters including initial construction workmanship, environmental factors, truck traffic volumes, and maintenance protocols. In a survey conducted (Chang & Lee, 2001) in the state of Indiana and neighboring states, the estimated average service life of polymer modified asphalt joints is between 5.74 to 5.82 years. It is naturally characteristic of asphaltic plug joints to soften in hot weather, crack in very cold weather and rut if placed over a high traffic volume bridge which could seriously degrade or limit the serviceability of the joint resulting in the need for replacement (Dornsife, 2000). A summarized list of advantages and disadvantages for asphaltic plug joints is presented below.

Advantages:

- Minimal maintenance is required during the joint service life
- Easy to install, repair and replace
- Low susceptibility to snowplow damage (National Cooperative Highway Research Program, 2003)
- Joint can be removed quickly by a cold planer
- Smooth seamless roadway surface (Iowa DOT Bridges and Structures Bureau, 2024)

Disadvantages:

- Prone to rutting over time and not recommended for heavy truck volumes
- Prone to potholing
- Matrix softens in hot weather and cracks in very cold weather (Arora and Associates, P.C., 2019; Iowa DOT Bridges and Structures Bureau, 2024)
- Low service life expectancy (Dornsife, 2000)
- Only applicable for small longitudinal movements

Maintenance Requirements. No maintenance over the service life of the joint is required other than a hot applied asphalt top-up in rutted areas where the asphalt has softened or failed.

Practical Considerations. When implementing asphaltic plug joints, a blockout needs to be installed where the surrounding concrete substrate needs to be sound. Otherwise, the substrate must be repaired prior to installation of the joint. For an ideal bond, the substrate should be prepared prior to its application to the matrix and aggregate (WSDOT, 2023). Washington State Department of Transportation no longer permits the use of asphaltic plug joints due to their inability to withstand high traffic counts, heavy trucks or acceleration/deceleration traction (WSDOT, 2023). For small movement range joints, the state agency has opted for the utilization of rapid-cure silicone sealant pourable seal joints almost exclusively.

Construction and Maintenance Costs. The relative installation cost for asphaltic plug joints with units of measurement of linear feet is estimated at a minimum cost of $219 per LF, maximum cost of $845 per LF, and a mean cost of $603 per LF. If damage to the asphaltic plug joint matrix is evident then replacement costs would be only the costs of the asphalt patching material and labor (assuming mobilization with a pothole patching project). Repair of more widespread asphalt plug damage requiring replacement of the bridging plate, matrix binder and aggregate would be equivalent in cost to the initial construction plus mobilization and any header repair costs.

6.3.1.4 Sliding Plate Joints

Description and Classification. Sliding plate joints are open expansion joints where two sides of the steel sliding joint are armored with steel angles. A steel field plate is attached to the angle on one side and slides on the other angle. The top surfaces of the plate are typically flush with the top of the bridge deck and the plates are either bolted down to timber deck panels or embedded with steel anchorages into the concrete deck (Arora and Associates, P.C., 2019). The longitudinal movement range for sliding plate joints is between 1 to 3 inches with bridge lengths between joints up to about 350 feet (Malla, Shaw, Shrestha, & Boob, 2006). Span length limitations are highly dependent on a combination of bridge skew, superstructure type, and maximum thermal movement.

Figure 79. Sliding plate joint in sidewalk on Iowa 926 over the Des Moines River

Service Life and Performance. The service life expectancy of sliding plate joints ranges between 15 to 25 years (Bolluyt, Kau, & Greiman, 2001) depending on various parameters including initial construction workmanship and if preventative maintenance or replacement of the sliding plate has occurred.

Sliding plates generally do not provide a seal against intrusion of water and deicing chemicals (Dornsife, 2000). As such, service life issues that have arisen include corrosion of the steel components, bending, misalignment, and fatigue of the sliding plate and clogging of the drainage troughs. In addition, debris typically accumulates in the slot where the sliding plate moves which has caused the joint to fail. Thus, sliding plate joints are not recommended for new deck installation or rehabilitation and replacement. A summarized list of advantages and disadvantages for sliding plate joints is presented below.

Advantages:

- Relatively long service life with proper maintenance

Disadvantages:

- Impossible to control leakage
- Prone to rust and corrosion
- Prone to anchorage problems and the anchors corroding over time
- Prone to fatigue damage from traffic loads

- Prone to snowplow damage by the snowplow blade striking the sliding plate distorting it resulting in a hazard for the public
- Not recommended for high truck volumes (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014; National Cooperative Highway Research Program, 2003)
- Only practical for small longitudinal movements

Maintenance Requirements. The use of sliding plate joints is not recommended for new deck installation or rehabilitation and replacement. Sliding plate joints require not only joint maintenance like debris removal and sliding plate replacement/anchorage repair, but also preventative maintenance to the substructure and superstructure below, because water can pass through the joint opening.

Practical Considerations. It has been reported that sliding plate joints are not typically used or specified and have been phased out by many state departments of transportation due to the connection of the sliding plate proving to be unreliable (Arora and Associates, P.C., 2019). There have been many issues with bolts fracturing, leakage through the joint and consequent corrosion of the steel parts (National Cooperative Highway Research Program, 2003). Moreover, since the sliding plates do not provide a watertight seal against infiltration of water and deicing salts and chemicals, sliding plate joints are considered an open joint (WSDOT, 2023). Typical defects include broken welds between the underlying steel angle and the sliding plate, and vertical misalignment of the sliding plate. Additionally, the sliding plates tend to bind up when debris and pack rust from steel corrosion get in between the steel plate and angles hindering movement of the joint and inability of the bridge to freely move within the span location.

Construction and Maintenance Costs. No major manufacturers currently supply sliding plate joints, and the joint type has been phased out by many departments of transportation. As such, no cost data is available for specifying sliding plate joints.

6.3.1.5 Small Joint Movement Category Summary

Table 54 provides a brief overview of the movement limits, joint classification, service life, skew limit, maintenance requirements, and relative cost for all the expansion joints categorized within the small joint movement category.

Table 54. Small Longitudinal Movement Expansion Joints Summary

Joint Type	Movement Limit (inches)	Open or Closed	Service Life (Years)	Skew Limit (Degrees)	Maintenance Level	Mean Installation Cost (LF)
Pourable Seal	Up to 4	Closed	5-10	45	Medium	$371
Compression Seal	Up to 2.5	Closed	10-15	45†	Medium	$435
Asphaltic Plug	Up to 1.5	Closed	2-5	30	Low	$603
Sliding Plate	Up to 4	Open	15-25	45	High	N.A ‡

† Upper bound value, however current manufacturer recommends installation at no more than 15 degrees.

‡ Not Available

In the small expansion joint movement category, pourable seal joints are recommended when possible. This joint type can be used on bridge joints with 2 to 4 inches of movement range, is quick and easy to install and maintain, and can last up to 10 years with yearly cleaning and flushing. To prolong the service life of a pourable seal joint, it is prudent to install steel armor angles using shock-absorbing materials as the joint nosing. The unreliability of compression seals and sliding plates due to water penetration and material damage has pushed multiple departments of transportation to discontinue their use. Asphaltic plug joints should only be used when necessary for bridge maintenance as the low service life and high susceptibility to rutting and temperature-induced failures proves them to be uneconomical. Yearly maintenance in the form of clearing debris should be conducted with any of the above mentioned joints to help them reach the prescribed service life. In addition, reporting of the condition state of joints during biennial routine bridge inspections aids in tracking the performance of various bridge joints.

6.3.2 Medium Longitudinal Movement Joints

Medium longitudinal joints are joints that can be characterized as being capable of accommodating up to 5 inches of longitudinal movement. Defining medium longitudinal movements as up to 5 inches is a threshold the authors have set based on values presented in literature. The main expansion joint that fits within this category, which is frequently used in many states, is the strip seal joint.

6.3.2.1 Strip Seal Joints

Description and Classification. Strip seal joints are a closed expansion joint consisting of steel retainer rails, i.e. steel extrusions embedded in the header concrete, a neoprene gland in-between the steel retainer rails, and a lubricant/adhesive to facilitate the installation of the gland and to seal the gland in the extrusions (Bolluyt, Kau, & Greiman, 2001). The neoprene gland serves as a water barrier protecting the structural elements below the joint from surface runoff, de-icing salts, chemicals, debris, and deleterious materials (Bolluyt, Kau, & Greiman, 2001). The neoprene gland can be anchored mechanically to the extrusion rails or by adhesives. The steel extrusion rails are typically fabricated in 20 to 40 feet lengths and are field spliced as required in-situ (Baker Engineering & Energy, 2006). The maximum skew angle limit reported by manufacturers is 45 degrees; however, movement capacity reduces with skews past 30 degrees. The longitudinal movement range of strip seal joints varies slightly between manufacturers but can go up to 5 inches (Bolluyt, Kau, & Greiman, 2001). Span length limitations are directly dependent on the skew, superstructure type, and maximum thermal movements.

Figure 80. Strip seal joint in US 61 over Mississippi River

Figure 81. Underside of strip seal joint in US 61 over Mississippi River

Service Life and Performance. The service life expectancy of a strip seal joint ranges between 20 to 25 years (D.S. Brown, 2013) depending on various parameters including initial construction workmanship, environmental factors, and maintenance protocols. One manufacturer mentions the joint system is expected to last the life of the bridge deck. In an Iowa DOT study, a service life of 15-20 years was reported (Bolluyt, Kau, & Greiman, 2001). In a national survey of bridge owners and agencies (Baker Engineering & Energy, 2006), the estimated average service life of strip seal joints is reported as 18.01 years. Strip seals typically exhibit long service life, good anchorage, and high degree of water tightness (Chang & Lee, 2001). However, the joints are susceptible to leakage at field splice locations and at any defects within the neoprene gland. Therefore, a gland repair could be warranted every 5 to 10 years, the replacement of the entire gland every 10 to 20 years and the replacement of the entire joint every 15 to 25 years. Common failure modes for strip seals include damage to the neoprene gland from debris such as anti-skid materials and sharp objects, snowplows damaging the steel extrusion rails, and loss of adhesive between the neoprene gland and the steel extrusion rails. A summarized list of advantages and disadvantages for strip seal joints is presented below.

Advantages:

- Steel extrusion rails armor the edges of the joint and the concrete wearing surface preventing the need for special impact resistant concrete
- Watertight joint if properly installed
- Gland can be easily replaced if damaged
- Good performance record (Chang & Lee, 2001; Bolluyt, Kau, & Greiman, 2001)

Disadvantages:

- Premature failure when neoprene gland debonds, tears or pulls out from the steel extrusion rails (Baker Engineering & Energy, 2006)
- Lack of routine deck drainage maintenance such as not clearing out the debris trapped within the neoprene gland could result in less than satisfactory performance and poor ratings
- Prone to leakage and water tightness degenerates with age
- Neoprene seals do not readily conform to out of skew turns at the gutters

- Lengthy traffic lane closures and detours are required for complete replacement of joint

Maintenance Requirements. To maintain and prolong the service life of a strip seal joint, yearly cleaning and flushing is necessary as a preventative maintenance action to remove accumulated debris in the joint, which could cause premature failure of the joint. In the event of gland tearing or pull out, complete replacement of the gland is required. In the event of damage to the extrusion rails, complete replacement of the joint is required.

Practical Considerations. It is recommended that the steel extrusion rails are recessed below the final elevation of the wearing surface of the bridge deck by 1/8 inch to prevent damage and hazards to vehicular traffic. In the survey conducted by Baker Engineering & Energy, 1 out of the 27 reported government agencies did not permit the use of strip seals due to bad experiences; however, it is very important to note that various states, such as Illinois, heavily utilize strip seals joint and primarily rely on their utilization for any opening up to 4 inches and have had favorable results (Baker Engineering & Energy, 2006).

Construction and Maintenance Costs. The relative installation cost for strip seal joints with units of measurement linear feet is estimated at a minimum cost of $174 per LF, maximum cost of $555 per LF, and a mean cost of $294 per LF. This unit price only includes the furnishing of the steel extrusions. Installation and testing of the associated neoprene gland is estimated at a minimum cost of $52 per LF, maximum cost of $109 per LF, and a mean cost of $75. Installation of any associated structural concrete material for the joint header is estimated at a minimum cost of $275 per LF, maximum cost of $1,291 per LF, and a mean cost of $767 per LF. The total arithmetic sum is a unit price at a minimum cost of $503 per LF, maximum cost of $1,954 per LF, and a mean cost of $1,136 per LF. Maintenance costs on strip seals vary as the whole joint does not necessarily need to be replaced. A patch repair on the neoprene gland could be warranted, the whole gland might need to be replaced, or the entire joint. Maintenance costs on strip seal joints in the form of gland repairs would cost on average approximately $75 per LF plus mobilization costs. If the strip seal joint exhibits significant damage, then replacement costs of the joint would be equivalent to initial construction costs, plus existing concrete demolition and mobilization costs.

6.3.2.2 Medium Joint Movement Category Summary

Table 55 provides a technical summary of medium movement joints listing the movement limit, classification, expected service life, skew limit, maintenance requirements, and relative cost.

Table 55. Medium Joint Movement Expansion Joints Summary

Joint Type	Movement Limit (in)	Open or Closed	Service Life (Years)	Skew Limit (Degrees)	Maintenance Level	Mean Installation Cost (LF)
Strip Seal	Up to 5	Closed	15-25	30-45	Medium	$1,136

Strip seals are primarily the only expansion joint that fit within the 3 to 5 inch longitudinal movement range. They have been found to be resilient to heavy traffic loads and display good anchorage and water tightness when maintained through routine annual debris clearing and joint flushing. However, they are prone to degradation of water tightness with age and generally do not perform well at larger skew angles (Bolluyt, Kau, & Greiman, 2001).

6.3.3 Large Longitudinal Movement Joints

Large longitudinal joints are characterized as being capable of accommodating more than 5 inches of longitudinal movement. Defining large longitudinal movements as greater than 5 inches is a threshold the authors have set that is consistent with existing literature. The two most common joints that fit within this category are modular and finger joints.

6.3.3.1 Finger Joints

Description and Classification. Finger joints are open expansion joints that are fabricated from steel plate and are installed in cantilever or propped cantilever configurations. The steel fingers are designed to support traffic loads and slide past each other accommodating longitudinal movement of the bridge. Typically, the steel fingers are designed with a slight taper downward toward the end of the fingers to avoid snowplow blade impact. Steel finger joints do not provide a seal, and a drainage trough is most often installed below the joint to collect any surface runoff (Chang & Lee, 2001). Finger joints can be designed for longitudinal movements greater than 4 inches (National Cooperative Highway Research Program, 2003) and skews up to 45 degrees, though they are not recommended for skewed bridges as they tend to lock up (Arora and Associates, P.C., 2019). Span length limitations are dependent on the skew, superstructure type, and maximum thermal movements of the bridge.

Figure 82. Finger joint on I-80 Westbound over Missouri River

Figure 83. Neoprene trough beneath finger joint on I-80 Westbound over Missouri River

Service Life and Performance. The service life expectancy of a finger joint ranges between 20 to 25 years (D.S. Brown, 2013) depending on various parameters including initial construction workmanship, environmental factors, and maintenance protocols. A system manufacturer mentions the joint system is expected to last the life of the bridge deck. In a national survey of bridge owners and agencies (Baker Engineering & Energy, 2006) it is reported that the estimated average service life of finger joints is 28.10 years. Service life issues that have arisen with finger joints include bent and misaligned fingers, corrosion of the steel, and clogging of the drainage troughs (Baker Engineering & Energy, 2006; WSDOT, 2023). A summarized list of advantages and disadvantages for finger joints is presented below.

Advantages:

- Can cover and accommodate large joint openings and movements
- Less susceptible to fatigue damage compared with modular joints
- Good performance record (Arora and Associates, P.C., 2019)

Disadvantages:

- Leakage control requires additional drainage trough design (WSDOT, 2023)
- Prone to rust and corrosion (Baker Engineering & Energy, 2006)
- Fingers/teeth can get caught in snowplows distorting the fingers, hindering movement and resulting in a hazard to vehicular traffic (Dornsife, 2000)
- Require large amounts of space and room longitudinally and laterally to be installed (Arora and Associates, P.C., 2019)
- Tend to lock up in skewed bridges due to the occurrence of transverse movement causing the joint to bind (Arora and Associates, P.C., 2019)

Maintenance Requirements. The drainage channels/troughs located below the finger joints that serve to catch all deck surface runoff require frequent flushing to prevent debris accumulation and trough overflow onto superstructure and substructure elements. Lack of routine maintenance on steel finger joint troughs could yield them ineffective. The fingers can jam, bend, or break during service due to horizontal and/or vertical misalignment during construction or damage caused by snowplows.

Practical Considerations. To prolong the service life of finger joints, bridge skew angles between 28 and 35 degrees should be avoided to avoid coinciding with the angle of snowplow blades (Baker Engineering & Energy, 2006). In addition, it is recommended that the finger joint be fabricated with a slight downward taper towards the ends of the fingers to minimize the striking of the fingers by snowplows, predominantly, but also by any vehicular traffic (Dornsife, 2000). Finger joints require proper and precise design and installation to avoid bent/broken fingers and debris buildup. The state of California no longer allows finger joints; however, it continues to be used in many states in situations where their vulnerabilities are low, i.e. straight bridges with low to zero skew (Arora and Associates, P.C., 2019). Iowa DOT permits the specifying of new finger joints up to a maximum movement of 10 inches. Finger joints with movements past 10 inches require approval from a unit team leader.

Construction and Maintenance Costs. The relative installation cost for finger joints with units of measurement of linear feet is estimated at a minimum cost of $2,811 per LF, maximum cost of $5,827 per LF, and a mean cost of $3,855 per LF. Maintenance costs on finger joints vary as the whole joint does not necessarily need to be replaced. Possible maintenance requirements include concrete joint header patch repairs, localized replacement of individual steel fingers, replacement of the elastomeric drainage trough, or the replacement of the whole joint if damage is severe or the joint is at the end of service life. Elastomeric drainage troughs are estimated at a minimum replacement cost of $523 per LF, maximum replacement cost of $1,298 per LF, and a mean replacement cost of $762 per LF plus mobilization costs.

6.3.3.2 Modular Joints

Description and Classification. Modular joints are closed expansion joints consisting of two or more preformed compression seals or neoprene glands fixed between transverse load distribution members. Modular joints are complex mechanical devices that are deeper than most other joint systems. The depth required to accommodate a modular joint essentially prevents the joint from being an option for a retrofit. The joints are generally shipped to the construction site for installation in a completely assembled configuration (Dornsife, 2000). The longitudinal movement range of modular joints range between 4 to 30 inches (Baker Engineering & Energy, 2006).

Figure 84. Modular joint on US 61 over Mississippi River

Figure 85. Underside of modular joint

Service Life and Performance. The service life expectancy of a modular joint ranges between 20 to 25 years (D.S. Brown, 2013) depending on various parameters including initial construction workmanship, environmental factors, and maintenance protocols. A system manufacturer mentions the joint system is expected to last the life of the bridge deck. In a national survey of bridge owners and agencies (Baker Engineering & Energy, 2006), the estimated average service life of modular joints is 19.21 years. Modular joints are susceptible to fatigue damage and leakage between compression seals and steel supports. Common defects with modular joints include fatigue damage to the welded connections and welded splices of the center beams, poorly consolidated concrete headers, and improper installation of the joint too high or low relative to the bridge deck wearing surface. In addition, accumulation of debris in the seals, and water leakage at seal splices. A summarized list of advantages and disadvantages for modular joints is presented below.

Advantages:

- Can accommodate very large bridge movements (Arora and Associates, P.C., 2019)

Disadvantages:

- Complex joint to install and replace
- Multiple joint steel components prone to fatigue
- Expensive construction and maintenance cost
- Requires frequent maintenance
- Neoprene seal prone to leakage and water tightness degenerates with age
- Prone to damage from snowplows if not installed in a recessed manner below the elevation of the wearing surface (Baker Engineering & Energy, 2006)

Maintenance Requirements. To prolong the service life of a modular joint, yearly cleaning and flushing is necessary to remove accumulated debris in the joint glands, which could cause premature failure of the joint.

Practical Considerations. Results from a survey conducted by Baker Engineering & Energy show that the Arizona Department of Transportation avoids modular joints whenever possible due to the complexity and high costs associated with the joint. Modular joints suffered from fatigue problems early in their development, but those have been largely resolved (Arora and Associates, P.C., 2019). However, modular joints remain complicated compound devices where skilled labor is needed for fabrication and installation.

Construction and Maintenance Costs. The relative installation cost for finger joints with units of measurement of linear feet is estimated at a minimum cost of $2,834 per LF, maximum cost of $5,311 per LF, and a mean cost of $3,881 per LF. Maintenance costs on modular joints vary as the whole joint does not necessarily need to be replaced. Possible maintenance requirements include concrete joint header patch repairs, neoprene gland patch repair, replacement of the glands, or the entire joint may need replaced. Maintenance costs on modular joints in the form of gland repairs are at an approximate average price of $75 per LF for each gland in the compound assembly system plus mobilization cost.

6.3.3.3 Large Joint Movement Category Summary

Table 56 provides a technical summary of large movement joints listing the movement limit, classification, expected service life, skew limit, maintenance requirements, and relative cost.

Table 56. Medium Joint Movement Expansion Joints Summary

Joint Type	Movement Limit	Open or Closed	Service Life (Years)	Skew Limit (Degrees)	Maintenance Level	Mean Installation Cost (LF)
Finger	Up to 24 in.	Open	20-30	45	High	$3,855
Modular	Up to 30 in.	Closed	18-25	45	High	$3,881

Modular expansion joints can accommodate large movements, reportedly up to 30 inches. It is stated that the service life of the joint is expected to last the life of the bridge deck, however the complexity of their fabrication, installation, and maintenance makes them an expensive investment. The complex geometry of the system accommodates skews as large as 45 degrees, but the joint is susceptible to fatigue damage, leakage, and snowplow damage. When implemented, these joints require regular routine maintenance to remain effective. Finger joints similarly can accommodate large movement, reportedly, but require regular routine maintenance to remain effective specifically with clearing the drainage trough as it is an open joint. Historically, modular joints were a more expensive option over finger joints but improvements to fatigue prone details have made the price comparable to finger joints. Based on a national survey of bridge owners and agencies (Baker Engineering & Energy, 2006), it was concluded that finger joints exhibited a longer service life than modular joints, 28.10 years vs 19.21 years, respectively.

6.3.4 Jointless Bridge Joints

The implementation of jointless bridges has grown in popularity with government transportation agencies in the hopes that the absence of a joint over bearings and/or superstructure elements will eliminate the maintenance problems associated with expansion joints. Jointless bridges can be characterized by continuous spans built integrally with their abutments. Approach slabs are tied to the superstructure slab or to the abutments. Bridges with semi-integral and integral abutments can be characterized within the jointless bridge joint category. Nonetheless, semi-integral and integral abutments have limitations where the maximum practical expansion span length is approximately 400 feet for concrete superstructures and 300 feet for steel superstructures (Hassiotis, Khodair, Roman, & Dehne, 2006).

6.3.4.1 Semi-Integral Abutments

Description and Classification. In a semi-integral abutment, the superstructure and back wall are monolithic, but isolated from the abutment via bearings (girders) and foam (backwall) or other isolation methods. Often, the back wall lies behind the abutment stem and extends below the top of it. The relative movement between the superstructure and foundation is accommodated by the bearings and foam-filled gaps. The bearings are needed both during construction and in service but are protected from precipitation and deicing agents due to the relocation of the joint(s) to the approach(es) (Hassiotis, Khodair, Roman, & Dehne, 2006). The use of a semi-integral abutment can be implemented for bridges with larger expansion demand where an integral abutment could not be selected (Arora and Associates, P.C., 2019). Iowa DOT does not currently prescribe an expansion length limit for semi-integral abutments in the bridge design manual, rather stating that the length limits for no-skew integral abutments are appropriate for semi-integral abutments with skews up to 45 degrees. Indiana DOT, who has extensively adopted semi-integral abutments for new designs, similarly does not prescribe an expansion limit due to the versatility and adaptability of semi-integral abutment construction, but indicates that they should be explored when the limits of integral abutments are exceeded. Indiana DOT requires integral abutments for most bridges with lengths less than 1000 feet, depending on skew. The precise expansion limit will depend on the capacities of the bearings and isolation materials and can vary widely based on design.

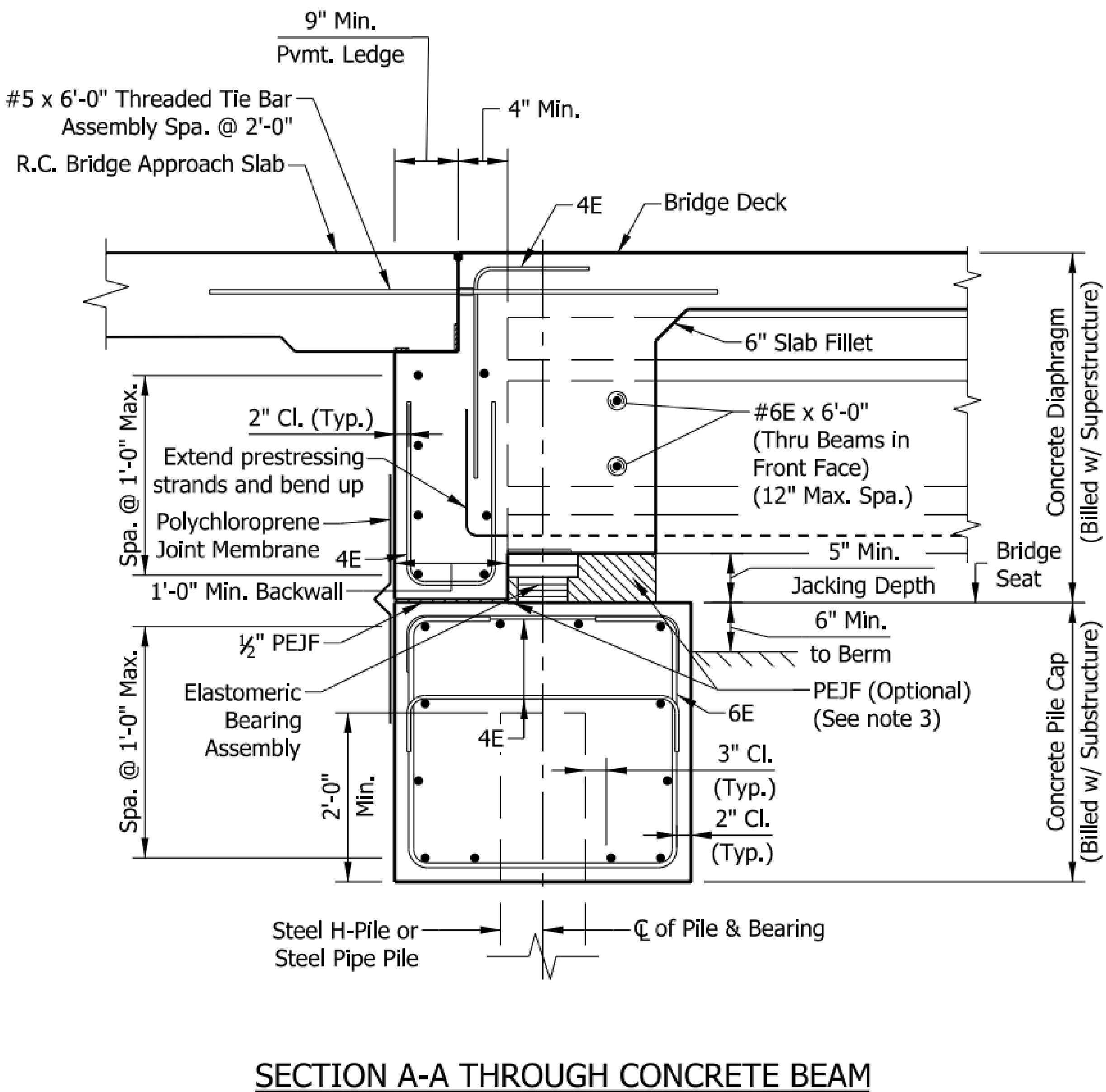

Figure 86. Indiana DOT standard semi-integral abutment detail (Indiana Department of Transportation, 2024)

Service Life and Performance. When designed and constructed properly, the service life of semi-integral abutments should match that of the superstructure (Hassiotis, Khodair, Roman, & Dehne, 2006). They do not require regular maintenance efforts. Common failures of this design include approach cracking and settlement, which results in poor performance of approach slabs that could be reported by public users as having poor rideability (Baker Engineering & Energy, 2006). A summarized list of advantages and disadvantages for semi-integral abutments is presented below.

Advantages:

- Offers larger expansion demands than integral abutments

- Ease of constructability
- Can utilize existing stub abutments to eliminate joints on deck replacement and superstructure replacement projects
- Allow for greater lateral expansion than integral bearings
- The absence of a joint above the bearings eliminates active joint-related issues (i.e. leakage, debris build up, maintenance demands, etc.)

Disadvantages:

- Cracking and settlement of the approach slab panels causing poor in-service rideability
- Not favorable with high bridge skews due to movements parallel to the support (Arora and Associates, P.C., 2019)

Maintenance Requirements. No direct maintenance is required, though relief joints in the pavement beyond the approach slab should be installed and sealed with an approved sealant product. Replacement of the relief joint sealant should be scheduled based on manufacturer recommendations.

Practical Considerations. Semi-integral abutments should be designed in tandem with pavement relief joints and the utilization of approach slabs. Some conditions do not favor semi-integral abutments such as bridges that are skewed or curved which cause movement parallel to the support and thus some shearing motion at the joint (Arora and Associates, P.C., 2019).

Construction and Maintenance Costs. Semi-integral abutments form a jointless bridge as the superstructure and back wall are monolithic. Cost of installation of a semi-integral abutment is not equivalent in the sense of an expansion joint device or assembly as the cost for the semi-integral abutment is embedded within the cost of concrete per cubic yard and pounds of steel used in other components of the bridge within a contract. The cost viability of a semi-integral abutment will vary greatly depending on span and skew, and each bridge for which a semi-integral abutment is considered will require a cost-benefit analysis. In general, however, the shorter and less skewed a bridge is, the greater the life-cycle cost benefits of semi-integral abutments will likely be. Maintenance costs associated with semi-integral abutments would be realized in the costs of concrete partial depth repairs, crack sealing, and addressing erosion beneath approach slabs to improve rideability.

6.3.4.2 Integral Abutments

Description and Classification. Integral abutments can be designed in single or multi-span structures with a continuous concrete deck and approach slabs. They are similar to semi-integral abutments except without bearings separating the girders from the abutment stem. Instead, the girders are continuous with the abutment or support. This allows for the entire end diaphragm or back-wall to be cast as one unit. The rotational movements caused by having fixed ends requires flexible foundations to be included in the design (Hassiotis, Khodair, Roman, & Dehne, 2006). Integral abutments accommodate smaller expansion movements than semi-integral abutments as they are limited by bending of the foundation piles. Integral abutments could easily accommodate up to two inches of total movement, if properly designed (Wasserman & Walker, 1996). Tellingly, Indiana DOT requires specification of integral abutments, with various degrees of skew, for structures with expansion lengths up to 1000 feet (Indiana Department of Transportation, 2024), with a design exception required for other substructure types. This implies an anticipated unconstrained expansion of up to 6 inches (between both abutments), which would be reduced by the structural response of the substructure by an amount that depends on the substructure stiffness. Iowa currently limits integral abutment to bridges with a bridge length of 575 feet at zero skew, with allowable lengths decreasing with increasing skew (Iowa DOT Bridges and Structures Bureau, 2024).

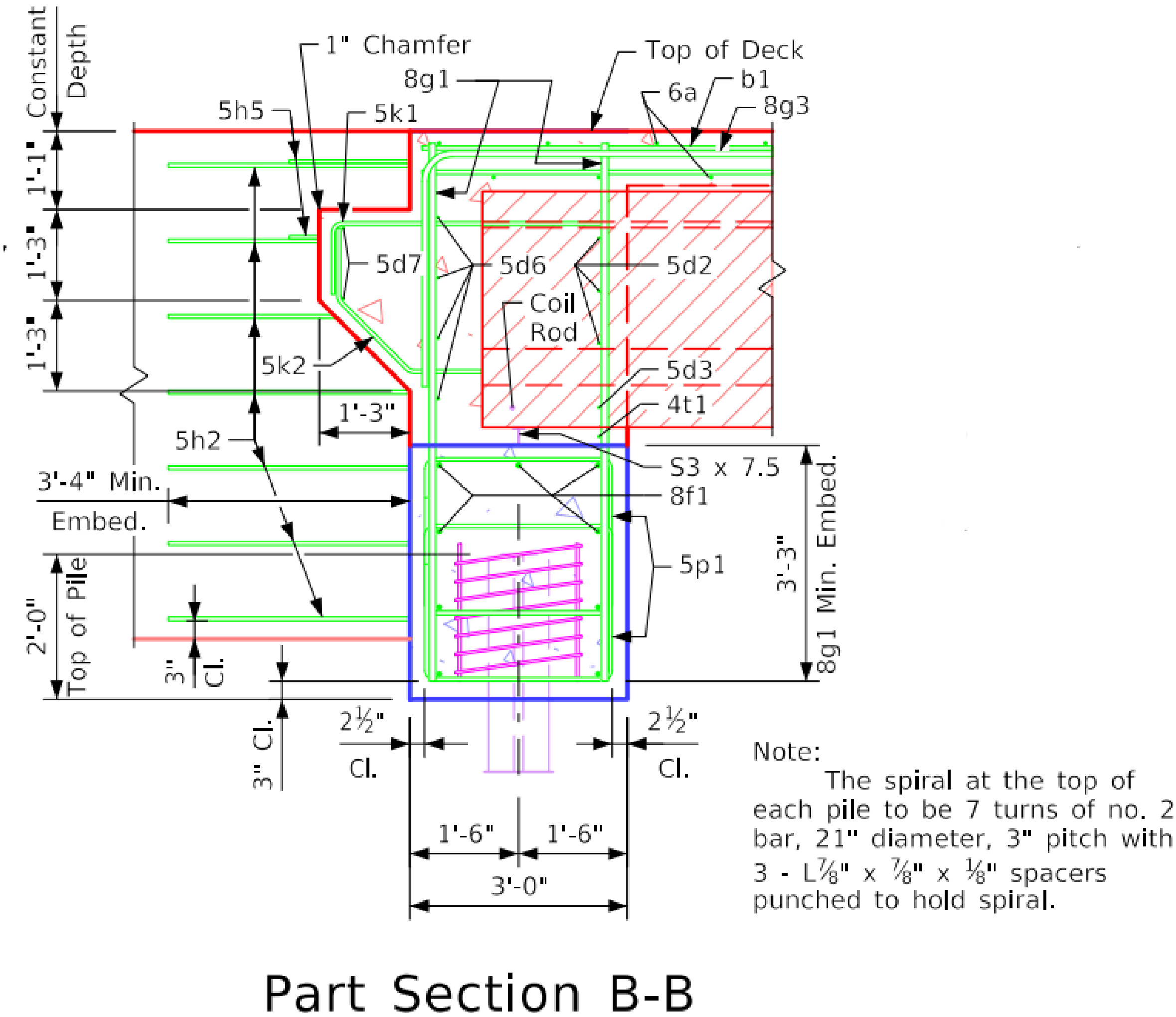

Figure 87. Excerpt from Iowa standard integral abutment detail (Iowa DOT, 2023)

Service Life and Performance. When designed and constructed properly, the service life of integral abutments should match that of the superstructure. They should not experience leakage or other issues commonly associated with expansion joints and, therefore, do not require regular maintenance efforts. Common failures of this design include approach slab cracking and settlement, which have been reported to cause poor rideability (Baker Engineering & Energy, 2006). A summarized list of advantages and disadvantages for integral abutments is presented below.

Advantages:

- Lack of required maintenance leads to low life cycle costs
- Ease of constructability

- The absence of a joint eliminates joint-related issues (i.e. leakage, debris build up, maintenance demands, etc.)

Disadvantages:

- Settlement of the approach slab causes poor in-service rideability
- Design is limited to small expansion movements tolerated by bending of the foundation piles
- Imposed limits on the length and skew of bridges (Hassiotis, Khodair, Roman, & Dehne, 2006)

Maintenance Requirements. Maintenance is not required at the abutment, but approach slabs should be inspected for potential seal failures and 45 degree cracks across the corners.

Practical Considerations. In the survey conducted by Baker Engineering and Energy, one governmental agency reported using integral abutments opting for a jointless bridge design on more than 90% of new construction where applicable. Other states put limitations on integral abutment implications to ensure that support conditions are properly met. The state of Minnesota for example, does not use integral abutments at stream crossings because of the possibility of scour (Arora and Associates, P.C., 2019). Similarly, Michigan avoids integral abutments at stream crossings because of the possibility of scour. It was reported (Hassiotis, Khodair, Roman, & Dehne, 2006) that many states have limited the skew angle on integral abutment bridges to 30 degrees. Connecticut DOT and Oklahoma DOT do not allow integral abutments on skewed bridges while Colorado DOT has no limit of skew on integral abutments (Hassiotis, Khodair, Roman, & Dehne, 2006).

Construction and Maintenance Costs. Integral abutments form a jointless bridge where the concrete end diaphragm and backwall are cast as a monolithic unit. Cost of installation of an integral abutment is not equivalent in sense to an expansion joint device or assembly as the cost for the integral abutment is equivalent and embedded within the cost of concrete per cubic yard and pounds of steel used in other components of the bridge within a contract. The cost viability of an integral abutment will vary greatly depending on span and soil conditions, and each bridge for which an integral abutment is considered will require a cost-benefit analysis. In general, however, the shorter a bridge is, the greater the life-cycle cost benefits of integral abutments will likely be. Maintenance costs associated with integral abutments would be realized in the costs of concrete partial depth repairs, crack sealing, and addressing erosion beneath approach slabs to improve rideability.

6.3.4.3 Jointless Bridge Category Summary

Table 57 provides a technical summary of jointless bridge options, listing the movement limit, classification, expected service life, skew limit, maintenance requirements, and relative cost.

Table 57. Jointless Bridge Systems Category Summary

Joint Type	Bridge Length (feet)	Open or Closed	Service Life (Years)	Skew Limit (Degrees)	Maintenance Level	Mean Installation Cost (LF)
Semi-Integral Abutment	575+	Closed	Life of Superstructure	30	Low	N/A †
Integral Abutment	575	Closed	Life of Superstructure	30	Low	N/A †

† *Not Applicable*

Integral and semi-integral abutments provide the longest service life of the discussed joints by matching that of the bridge superstructure. Both evade many issues commonly encountered by joints as they are cast monolithically; however, there is a limitation on when they can be implemented. Semi-Integral abutments tolerate larger expansion movements than integral abutments in both transverse and longitudinal directions. Nonetheless, integral abutments boast the advantage of simple and quick constructability.

6.3.5 Current Iowa DOT Practice for Bridge Joints

The Iowa DOT prefers jointless bridge construction because it avoids the risks of joint leakage and consequent deterioration and maintenance needs of the underlying elements (Iowa DOT Bridges and Structures Bureau, 2024). While these bridge designs are called "jointless," joints accommodating the expansion and contraction of the bridge relative to the adjacent pavement are usually still present; the joints have simply been pushed into the bridge approach system, outside of the limits of the bridge. The bridge approach systems used by the Iowa DOT often have multiple joints, and the joint designs vary depending on the type of bridge, type of route, and types of elements present. Joints in approach systems are addressed in Section 6.6.

When a deck expansion joint within the limits of the bridge cannot be avoided, the Iowa DOT generally uses a strip seal joint, or a finger joint if a strip seal joint cannot provide the necessary performance. Strip seal joints allow for movements up to 5 inches, but lose the ability to accommodate movement if the bridge skew exceeds 30 degrees and, therefore, may not be practical in such scenarios. Finger joints are rated for movements up to 10 inches, and may be used for even greater movements with special approval. The Iowa DOT previously used compression seal joints as well, which were rated for movements to 2 or 3 inches. However, there is less need for joints limited to relatively small movements because the Iowa DOT has expanded the conditions under which integral abutments are considered permissible. As a result, the Iowa DOT uses strip seal joints instead of compression seal joints on new bridges (Iowa DOT Bridges and Structures Bureau, 2024).

The strip seal joints used by the Iowa DOT consist of galvanized carbon steel armor and anchors and a neoprene gland. The neoprene gland is required to conform to ASTM D2628, *Standard Specification for Preformed Polychloroprene Elastomeric Joint Seals for Concrete Pavements*, excluding the low-temperature recover testing (Iowa DOT, 2023).

The Iowa DOT requires that finger joints be constructed with galvanized steel and be recessed 0.25 inches to avoid damage from traffic and snowplows. The finger joint armor is also required to have regularly spaced, 0.75-inch diameter vent holes to allow trapped air to escape during concrete placement. To protect underlying bridge elements, the Iowa DOT requires the use of an elastomeric drainage trough underneath finger joints. The drainage trough is required to have a slope of at least 8% to channel the chloride-contaminated water and debris away from the protected elements and decrease the risk of debris buildup and resulting blockage. The hardware used to attach the trough to the finger joint is required to be stainless steel (Iowa DOT Bridges and Structures Bureau, 2024).

6.3.6 Joint Design Recommendations

The current Iowa DOT practice of joint elimination, with a strong preference toward integral abutments, is currently being practiced by several other DOTs, and though data on the time scale of a bridge's lifetime is not yet available, studies and performance to date support continuing this practice. Iowa DOT's current integral abutment details a very similar to other early adopters who have likewise seen good results. Semi-integral abutments should continue to be used only in situations that do not lend themselves to integral abutments because of the added construction expense of elastomeric bearings, but also because of added maintenance possibly required to replace sealant materials and repairs moisture affected concrete that is more likely in these abutment types. Iowa DOT is already among the leaders in span length for integral abutment bridges, and is operating on very recent research, but as more information and design guidance becomes available, Iowa would benefit from continuing to investigate the limits of span length and skew for integral abutment bridges. In long bridges, where medium or small expansion joints are required, the strip seal joint is an appropriate choice. Note that the steel extrusion rail is expensive to replace if overlay projects change the top of deck elevations at the joints. With the mean service live of the extrusion rail significantly longer than all viable small movement joints and the replacement costs of the neoprene gland sufficiently low, Iowa's current practice of installing strip joints preferentially for small and medium expansion situations minimizes maintenance. The small movement joints considered should be reserved for rehabilitation situations, and small bridges with low truck traffic. Here, the small longitudinal joints may fare better and the higher initial cost (up to 3 times the cost of most small movement joints) of the strip seal joint, which will be more expensive per foot at lower quantities, may not overcome replacement costs of a pourable seal or compression seal every five to ten years for the bridge's life. Even in these low ADTT situations, a strip seal is likely still preferred to minimize trips for a maintenance crew to the bridge if it's sufficiently remote that mobilization becomes an excessively large portion of the maintenance costs. Finger joints should continue to be standard for large expansion joints in the state due to the longer expected service life. Both modular and finger joints have service issues, but the finger joints concentrate the maintenance needs on the replaceable elastomeric trough and have less tendency for components to corrode and bind. Iowa DOT's trough detail for finger joints is one of the more accessible for routine cleaning with access from the substructure. Trough materials that are more puncture resistant, have lower friction values on the inside faces, remain flexible when exposed to salts and sun could be a focus of future research to limit substructure deterioration at these joints.

6.4 Bridge Bearings

Bearings are integral components in the design and service life of a bridge because they transmit the forces from the bridge superstructure to abutments and piers and allow for expansion. There are various types of bearings available to a bridge designer; however, each bearing has specific advantages and limitations. This report chapter highlights and focuses on the following bridge bearings, which encompass the vast majority of bearings utilized in the United States currently in service and prescribed by bridge designers in the past:

- Elastomeric Bearings
  - Plain Elastomeric Pad
  - Steel-Reinforced Elastomeric Pad
  - Cotton Duck Pad (CDP)
- High-Load Multirotational (HLMR) Bearings
  - HLMR Disc
  - HLMR Pot
  - HLMR Spherical
- Steel Mechanical Bearings
  - Rocker bearings
  - Low profile curved sole plate bearings

Bearings will be classified and grouped into four main categories based on the type of bridge bearing. Within each category, each of the following properties is discussed:

- Description and Classification
  - Load Range
  - Movement and Rotation Capacity
- Service Life and Performance
  - Life Expectancy
  - Common Issues and Failure Modes
  - Advantages and Disadvantages
- Maintenance Requirements
- Practical Considerations
- Construction and Maintenance Costs

Additionally, the use of sliding surfaces will be discussed first as they can be and are commonly implemented in tandem with many of the highlighted bearings. The presented relative installation costs of the different bridge bearing types were estimated based on an average of available unit price data from recent year(s) online bid tabulations obtained from various states including Washington State, Illinois, California, Iowa, Missouri, and Indiana.

6.4.1 Sliding Surfaces

Description and Classification. When the lateral movement requirements of a design surpass the shear capacity of a bearing, designers may implement a sliding surface with low frictional properties to account for the difference. The addition of a sliding surface is commonly implemented to accommodate large thermal movements (AASHTO/NSBA Steel Bridge Collaboration, 2023). Moreover, they provide additional movement capability (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Sliding surfaces are commonly achieved by adding a bronze plate or a layer of polytetrafluorethylene (PTFE) between the bearing and the bridge superstructure girder. PTFE layers are combined with a stainless steel plate embedded into the sole plate to decrease friction between the components. The stainless steel surface is larger than the PTFE surface so full movement is achieved without exposure of the PTFE (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The stainless steel surface serves to also protect the PTFE from dirt and debris contamination. PTFE has desirable material properties such as having low frictional characteristics, chemical inertness, and resistance to weathering and water absorption, which make it ideal for use with bridge bearings (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Bronze plates are finished and lubricated and typically paired with stainless steel or galvanized top plate connected to the beam.

Sliding surfaces are attractive as movement is only initiated once a displacement is large enough for the shear stress of the bearing to overcome the static friction at the interface (Arora and Associates, P.C., 2019). Fiber glass, carbon fibers, or other chemically inert reinforcements can be added to PTFE to increase wear and creep resistance. Adding fibers, however, will increase the friction coefficient by as much as 30% (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). A sliding surface can be added to any fixed, elastomeric, or HLMR bearing. For example, one of Iowa DOT's standard expansion bearings adds a bronze sliding plate to the standard Iowa DOT fixed bearing comprised of a masonry plate, pintle plate, and a curved sole plate to allow expansion and rotation. When equipped with a sliding surface, an elastomeric bearing can achieve movement ranges comparable to HLMR bearings (Arora and Associates, P.C., 2019).

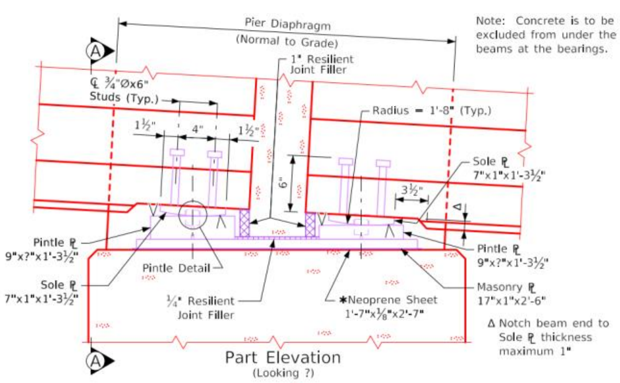

Figure 88. Iowa DOT standard fixed pier bearing (Iowa DOT, 2023)

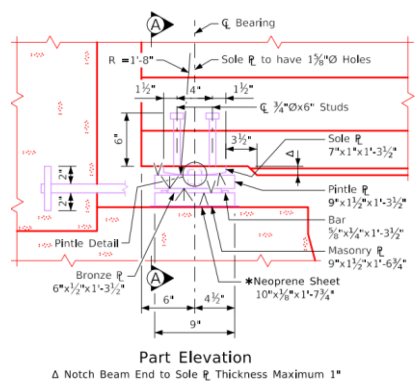

Figure 89. Iowa DOT standard bronze plate expansion bearing

Service Life and Performance. Sliding surfaces have a service life between 20 to 25 years before replacement is required. Factors affecting wear life include sliding speeds, contact pressure, mating surface roughness, and contamination of the sliding interface. Lubrication of the PTFE is a method to significantly reduce the coefficient of friction (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Research (Nejad & McGormley, 2024) showed that even small amounts of dust contamination can result in a factor of 10 increase in the friction coefficient of PTFE bearing. A summarized list of advantages and disadvantages for bearing sliding surfaces is presented below.

Figure 44. Pre-test photographs of PF3 specimen sprinkled with a light amount of dust prior to testing.

Figure 90. Reproduction of Figure 44 from Nejad and McGormley, 2024 demonstrating amounts of dust required to drastically change coefficients of friction on PTFE sliding surfaces

Advantages:

- Compensate for demand-capacity gaps between bridge loading and bearings
- Accommodate a wider range of expansion movements

Disadvantages:

- Frictional coefficient of PTFE is susceptible to changes in weather, having an inverse relationship with temperature
- Routine maintenance inclusive of surface lubrication and debris removal is necessary to maintain low coefficients of friction
- Additional maintenance/inspection required beyond typical bearing needs

Maintenance Requirements. To maintain effectiveness of the sliding surface, the surface requires routine maintenance beyond that of the bearing on its own. The contact surface should be checked for dirt and debris buildup, steel surface corrosion, and scratching, paint or other contamination. Dimpled and lubricated PTFE surfaces require routine maintenance because lack of lubrication or depletion of the lubricant will cause the coefficient of friction to rise. PTFE fragments are an indication of wear (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014) where consideration should be given to replacing the PTFE. A common indicator of overloading and failure of the PTFE surface is flaking and PTFE debris (AASHTO/NSBA Steel Bridge Collaboration, 2023). If the sliding surface is found to be worn or damaged, then it must be replaced.

Practical Considerations. Bridges designed with sliding surfaces on their bearings should be detailed such that the bridge can be jacked because maintenance during the service life of the bridge may be required (AASHTO/NSBA Steel Bridge Collaboration, 2023). Dimpling of the PTFE surface can be implemented to facilitate lubrication. Dimples are spherical indentations that are machined into the PTFE surface to act as reservoirs for storing lubrication (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Proper alignment and positioning of the steel to PTFE surfaces must be ensured for effective use. Plain PTFE should not be used in cases where the bearing is subject to relatively high sliding speeds and low temperatures (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Sliding surfaces should be installed away from joints. Jointless decks offer the least exposure to corrosion (AASHTO/NSBA Steel Bridge Collaboration, 2023).

Construction and Maintenance Costs. The cost for the addition of a PTFE or any sliding surface is typically embedded within the cost of purchase and installation of a bearing. Although the cost for the addition of a sliding surface is dependent on various factors, the addition of a 1/8 inch thick stainless plate with a top PTFE sheet to a bearing is estimated at a mean cost of $600. This estimated mean cost is based on a recent WJE research project conducted in 2023 where a PTFE configuration was added to variously sized steel reinforced elastomeric bearing pads (Nejad & McGormley, 2024).

6.4.2 Elastomeric Bearings

Elastomeric bearings are a type of bearing best characterized as neoprene or natural rubber units that have no movable parts. They facilitate lateral and longitudinal bridge movements through shear deformations. Variations in geometry and materials are used to adapt the bearing to specified needs. Circular pads can be used for example when larger rotational limits are desired. Higher loads can be accommodated by layers of steel or cotton being interspersed within the polymer. Larger lateral movements are often accounted for by adding a sliding surface (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Plain, steel reinforced, and cotton duck elastomeric bearings pads are the three predominant elastomeric bearing styles designed and supplied in the United States (AISC, 2022).

6.4.2.1 Plain Elastomeric Bearing Pads

Description and Classification. Plain elastomeric pads are a type of elastomeric bearing consisting of an unreinforced plain neoprene or natural rubber pad. Neoprene has greater resistance to ozone and a wide range of chemicals than natural rubber, making it more suitable for harsh chemical environments (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The bearing has no movable parts and can accommodate movement and rotation through the deformation of the elastomeric pad (Arora and Associates, P.C., 2019). The ability of the elastomeric pad to deform in shear allows for the accommodation of both lateral and longitudinal movements. Maximum expansion lengths for utilization of this bearing go up to 150 feet (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Plain elastomeric pads are typically used for short spans where loads are not too high, and the loads and movements can be accommodated by the plain single layer of elastomer (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The permissible compressive stress of plain elastomeric pads is a function of the shape factor, which is the bearing plan area divided by the area of perimeter free to bulge. Therefore plain elastomeric bearing pads can only accommodate small horizontal translations and rotations (AISC 2022). Plain elastomeric bearing pads are rated to accommodate up to 100 kips of load with up to 1/2 inch of movement and 0.01 radians of rotation.

Figure 91. Plain elastomeric bearing pads on the Carr-Thompson Bridge in Lake Forest, IL

Service Life and Performance. Plain elastomeric bearing pads have a service life of 15 to 25 years depending on loading. They are susceptible to load effects and are known to experience crushing or tearing when overloaded (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Plain elastomeric bearing pads rely on contact friction to resist bulging (AISC, 2022). In addition, the properties of elastomers could degrade over time (Fu & Angelilli, 2007). The constant presence of oxygen on the bearing coupled with heat and UV radiation can cause the bearings to crack on their exposed edges (Fu & Angelilli, 2007). A summarized list of advantages and disadvantages for plain elastomeric pads is presented below.

Advantages:

- Accommodates multidirectional rotation
- Durable depending on the imparted loads
- Low maintenance (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014)
- Corrosion resistant

- Ease of production and construction

Disadvantages:

- Low load and displacement capacity
- Prone to pad "walk out"
- Exposure to UV radiation can contribute to aging of bearing (Fu & Angelilli, 2007)

Maintenance Requirements. Minimal maintenance, such as clearing debris from bearing seats, is required for plain elastomeric bearing pads. The pads require replacement when crushing or tearing is present. Elastomeric pads should be checked for evidence of the bearing pad "walking" out from under the supported load (AASHTO/NSBA Steel Bridge Collaboration, 2023).

Practical Considerations. Plain elastomeric bearing pads have proven to be economical and require minimal maintenance. They are best suited for bridges with short spans, small loads, and small movements. Adhesion to contact surfaces can be considered to reduce walking of the bearings, but adhesives and other methods of increasing contact forces vary in their effectiveness and service life. Although aging of the bearing pad can occur from exposure to UV radiation, some states, such as California, view the resistance to ozone or ultra-violet light as a cosmetic, surface problem that has essentially no influence on the performance of the bearing as a whole (Arora and Associates, P.C., 2019).

Construction and Maintenance Costs. The relative installation cost for plain elastomeric bearing pads with units of measurement of "each" is estimated at a minimum cost $175 each, maximum cost of $908 each, and a mean cost of $306 each. If damage to the bearing pad is evident from walking out or crushing, then the bearing should be replaced and would be equivalent to the initial installation cost with additional costs for jacking and accessing the bearing.

6.4.2.2 Steel-Reinforced Elastomeric Bearing Pads

Description and Classification. Steel-reinforced elastomeric bearing pads are a type of elastomeric bearing consisting of a neoprene or natural rubber pad combined with reinforcing internal steel shim plates. As vertical load and movement requirements increase, reinforcing steel plates are incorporated within the multiple layers of elastomer to form a laminated reinforced elastomeric assembly (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The addition of internal steel reinforcing plates allows for higher compressive design stresses and higher translation and rotation capacities as the internal bonded steel shims resist bulging of the unit (AISC, 2022). Steel-reinforced bearing pads are rated to accommodate 50 to 750 kips of load with up to 4 inches of movement and 0.02 radians of rotation. On their own, steel-reinforced elastomeric bearing pads can accommodate high truck movements.

Figure 92. Steel-reinforced elastomeric bearing on Deerpath Road over East Skokie Ditch in Lake Forest, IL

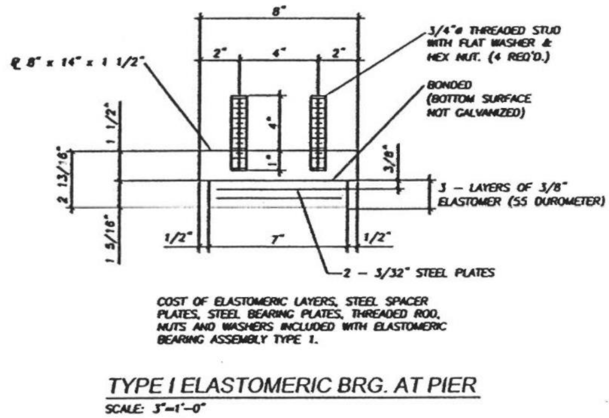

Figure 93. Plan detail of steel reinforced elastomeric bearings (2002)

Service Life and Performance. Anecdotal evidence suggests steel-reinforced elastomeric bearing pads have a service life that exceeds 50 years, with the chance of lasting over 100 years (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The state of Texas has reported an in-service steel reinforced elastomeric bearings over 70 years old (Arora and Associates, P.C., 2019). Pads that are improperly designed or installed may experience splitting or delamination. Other issues encountered are instability when the bearing becomes thick enough when trying to accommodate large lateral movement with large end rotations (Stanton, et al., 2008). In addition, the properties of elastomers could degrade over time (Fu & Angelilli, 2007). The constant presence of oxygen on the bearing coupled with heat and UV radiation can cause the bearings to crack on their exposed edges (Fu & Angelilli, 2007). A summarized list of advantages and disadvantages for steel reinforced elastomeric pads is presented below.

Advantages:

- High ozone and chemical resistance
- Higher load capacity than unreinforced elastomeric bearing pads
- Higher rotational capacity
- Low long term maintenance requirements.

Disadvantages:

- Prone to bulging, tearing or splitting if inadequately designed or improperly installed (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014)
- Prone to pad "walk out"
- Exposure to UV radiation can contribute to aging of bearing (Fu & Angelilli, 2007)

Maintenance Requirements. Steel-reinforced elastomeric bearings require a low level of maintenance. If the steel shims become exposed or the pad experiences deterioration due to excessive bulging or tearing, the entire pad requires replacement.

Practical Considerations. Circular bearing pads should be considered for skewed bridges to allow greater rotation at the bearing. Large lateral movement combined with large end rotations result in thick bearing pad designs. If bearings become thick enough, stability concerns can affect its performance (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The state of New York considers the combination of steel-reinforced bearings and sliding surfaces to accommodate the same large displacement capacity as HLMR bearings (Arora and Associates, P.C., 2019). Multiple states have set height limitations on steel-reinforced elastomeric bearings, which implicitly limits the displacement capacity (Arora and Associates, P.C., 2019). In addition, some states specify that steel-reinforced elastomeric bearings be made only from neoprene (Arora and Associates, P.C., 2019).

Iowa DOT currently prefers steel reinforced elastomeric bearings pads for most bridge superstructures inclusive of precast prestressed concrete beams, continuous welded plate girders, and rolled steel beams. Though the capacity of the bearing is capable of transmitting up to 750 kips of load and service translations up to 4 inches (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014), Iowa Bridges and Structures Bureau stipulates a maximum service load of 450 kips and a max service translation in one direction of 2-1/2 inches. The Iowa DOT Bridge Design Manual provides design detailing requirements for steel reinforced elastomeric bearings pads. The pads shall have a minimum side cover of 1/8 inch with a preferred layer thickness of 1/4 inch for cover elastomeric layers, 1/8 inch for steel plates, and 3/8 to 3/4 inch for internal elastomeric layers. In addition, tapered elastomeric layers are not permitted and the limiting thickness of the bearing is set at 5 inches (IOWA DOT Bridges and Structures Bureau 2023).

Construction and Maintenance Costs. The relative installation cost for steel reinforced elastomeric bearing pads with units of measurement of “each” is estimated at a minimum cost $1,079 each, maximum cost of $6,112 each, and a mean cost of $2,531 each. If damage to the bearing pad is evident through splitting, bulging, or exposure of the internal steel shims, then the bearing should be replaced and would be equivalent cost to the initial installation cost with additional costs for jacking and access.

6.4.2.3 Cotton Duck Elastomeric Bearing Pads

Description and Classification. Cotton duck elastomeric pads are a type of elastomeric bearing that consists of very thin layers of elastomer interlaid with cotton or polyester fabric. Cotton duck pads (CDPs) are typically specified for precast concrete I-girder bridges within the range of 150 feet to 180 feet (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). They are known to be stiff and strong in compression resulting in better accommodation of higher loads than plain elastomeric pads. Because of their limited resistance to translation, they are commonly used with a PTFE sliding surface or as fixed bearings and do not require a metallic substrate between the PTFE and CDP (Arora and Associates, P.C., 2019) (AISC, 2022).

CDPs can support loads up to 300 kips. The movement range is approximately 0.25 inches laterally and 0.005 radians in rotation. Limits in shear deflection capacity can be compensated by the addition of a sliding surface (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014).

Service Life and Performance. The service life of CDPs is currently uncertain due to a lack of data. Common issues are associated with production and operation defects pertaining to design and manufacturing (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The controlling limit state is often delamination of elastomer layers or secretion of oil and wax (Lehman, Roeder, Larson, & Curtin, 2003). Preventative measures to mitigate service life issues with CDPs include stress limits of 3,000 psi for total dead load plus live load and 2,000 psi for live load (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). A summarized list of advantages and disadvantages for CDPs is presented below.

Advantages:

- High resistance to corrosive environments
- Higher compression capacity than plain elastomeric bearing pads
- Aids in limiting rotational instability of heavy girders during construction.

Disadvantages:

- Do not support multidirectional rotation
- Limited movement range if used without PTFE sliding surface
- Delamination of elastomer layers or secretion of oil and wax are common limit states (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014)

Maintenance Requirements. CDPs require a low level of maintenance but are susceptible to splitting or cracking, which can cause the girder to slip when designed with inadequate capacity. If splitting or cracking of the pad is evident, then the bearing pad requires replacement. If a PTFE sliding surface is added for increased lateral movement, increased inspection and maintenance is required for the sliding surface.

Practical Considerations. The limited shear capacity of CDPs is frequently overcome by the addition of a PTFE sliding surface. CDPs have performed well, but they have had limited use (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The state of Pennsylvania's design manual for example explicitly excludes cotton duck bearing pads from use (Arora and Associates, P.C., 2019). On the other hand, Minnesota until recently used plain elastomeric pads at fixed bearings but those tended to squeeze out, and they have switched to cotton duck pads to resolve this issue (Arora and Associates, P.C., 2019).

Construction and Maintenance Costs. The relative installation cost for cotton duck bearing with units of measurement of "each" is estimated at a minimum cost $1,990 each, maximum cost of $4,000 each, and a mean cost of $2,820 each. If damage to the bearing pad is evident through splitting, bulging or exposure of the internal reinforcement, then the bearing should be replaced and would be equivalent cost to the initial installation cost with additional costs for jacking and access.

6.4.2.4 Elastomeric Bearing Summary

Table 58 presents a technical summary of all elastomeric bearings discussed in this section by listing their load capacity, rotation capacity, movement capacity, fatigue performance, corrosion resistance, and required maintenance levels.

Table 58. Elastomeric Bearing Category Summary

Bearing Type	Load Capacity Range (kips)	Rotation Capacity (Rad.)	Movement Capacity (in.)	Required Maintenance Level	Mean Installation Cost (Each)
Plain Elastomeric Pad	Up to 100	Up to 0.01	p to 0.5 †	Low	$306
Steel Reinforced Elastomeric	50-750	Up to 0.02	p to 4 †	Low	$2,531
Cotton Duck Pad	Up to 300	Up to 0.005	p to 0.25 †	Low	$2,820

† Value increases with addition of a sliding surface

Elastomeric pads are an inexpensive means of load distribution. Plain elastomeric pads are good for low-movement and low load designs. Steel-reinforced elastomeric bearings can support a larger load while also allowing for a movement range as large as 4 inches. They have the highest rotational capacity of the group, with a limit of 0.02 radians. Cotton duck pads have a higher stiffness, offering stability especially during girder erection. This stiffness, however, decreases the rotational and movement capacities as compared to plain and steel-reinforced elastomeric pads. All three elastomeric pads exhibit high corrosion resistance and a low level of maintenance. When paired with a sliding surface, all three experience an increased range of motion and a decreased performance under cyclic truck loading.

6.4.3 High Load Multirotational Bearings

High load multirotational (HLMR) bearings are generally used when design loads surpass elastomeric bearing capacities. Common implementations can be found in modern steel bridges where span lengths are maximized, and the number of longitudinal members is minimized (AASHTO/NSBA Steel Bridge Collaboration, 2023). Other application conditions include large skews, curved bridges, and complex framing. Three types of bearings currently make up the readily available varieties of HLMR bearings: disc bearings, pot bearings, and spherical bearings (AISC, 2022). All HLMR bearings risk damage from over-rotation or steel on steel contact; however, it is rarely experienced in practice (Arora and Associates, P.C., 2019).

6.4.3.1 Disc Bearings

Description and Classification. HLMR disc bearings consist of an upper and lower steel plate that compresses a hard polyether urethane disc, with a center shear pin device to resist horizontal loads. The discs have a high stiffness such that they can sustain high compressive loading. In turn, this increases rotational stiffness. A PTFE sliding surface is typically utilized as part of the disc bearing assembly. Disc bearings have a load bearing range of 100 to 5,000 kips and a rotational capacity of 0.02 to 0.04 radians. Lateral movement ranges can be set by designers to meet bridge needs by specifying the sliding surface size (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014).

Figure 94. Disc Bearing on I-80 Westbound Bridge over Missouri River

Service Life and Performance. HLMR disc bearings have an estimated service life of 25 years or more. Few service life problems have been reported over the years. Reported problems typically have been due to defects in design and manufacturing. Disc bearings are composed of multiple exposed steel surfaces that are susceptible to corrosion depending on environmental exposure levels. The high rotational stiffness is partly accommodated by uplift of the steel plates from the urethane disk during light loading. This can potentially cause edge loading on the PTFE sliding surface. Typically, the bearing lasts longer than 25 years. However, the sliding surface is the main component needing replacement from wear or damage from debris due to typically minimal maintenance activities being undertaken. A summarized list of advantages and disadvantages for HLMR disc bearings is presented below.

Advantages:

- High load capacity
- Accommodates multidirectional rotation
- Easy to inspect (Arora and Associates, P.C., 2019)

Disadvantages:

- High rotational stiffness limits rotation
- Bearing assembly components susceptible to corrosion
- Routine maintenance inclusive of sliding surface lubrication and debris removal is necessary to maintain low coefficients of friction
- Sliding surfaces subject to wear

Maintenance Requirements. To improve the longevity of the bearing, individual components should be replaced when damaged or worn. Those include the PTFE materials of the sliding surface and the elastomer disc which can split, crack, or bulge. Adequate corrosion protection systems are recommended on non-contact surfaces to protect the disc bearing assembly components.

Practical Considerations. Iowa Bridges and Structures Bureau has shown a strong preference towards the utilization of disc bearings for large loading conditions. However, Iowa Bureau specifies a maximum service load of 2,500 kips and a max service translation in one direction of 5 inches (Iowa DOT Bridges and Structures Bureau, 2024). Because disc bearings use less steel than pot bearings, they can be more economical for the same load capacity (Arora and Associates, P.C., 2019). AASHTO requires all HLMR bearings to be designed for future removal via vertical jacking to allow for maintenance (AASHTO/NSBA Steel Bridge Collaboration, 2023).

Construction and Maintenance Costs. The relative installation cost for HLMR disc bearings with units of measurement of "each" is estimated at a minimum cost of $6,516 each, maximum cost of $22,424 each, and a mean cost of $14,145 each. Maintenance costs on disc bearings are in the form of replacing individual components if they have been damaged but primarily the cost of replacing the sliding surface. The estimated mean cost for replacing a PTFE sliding surface would be $596 each plus mobilization and bearing access costs. Costs for jacking and removing bearing components are estimated at a mean cost of $4,443 each.

6.4.3.2 Pot Bearings

Description and Classification. HLMR pot bearings feature a shallow cylinder, or pot, that holds a tight-fitting elastomeric disc thinner than the depth of the pot. Bearing directly onto the elastomeric disc is a machined steel piston. A singular solid or multiple stacked brass rings seal the elastomer between the piston and pot components. The steel piston is topped with a PTFE surface and a stainless steel sliding plate above. Pot bearings transfer vertical loads through the piston to the confined elastomeric pad. They can accommodate rotation about any axis. Horizontal loads are resisted by direct contact between the pot wall and the piston. Pot bearings have a vertical load capacity range of 100 to 5,000 kips and a rotation capacity of 0.02 to 0.04 radians, though rotation capacities above 0.02 radians typically involve material substitutions or design changes from producers' standard bearings, and therefore increased costs.

Figure 95. Exploded view of pot bearing (D.S. Brown, 2022)

Service Life and Performance. The service life of pot bearings is estimated at 25 years or more. To ensure satisfactory performance during its service life, a high degree of quality control during fabrication and field installation is required. Improper production and installation can cause leakage or extrusion of the elastomer, broken seal rings, abraded elastomeric pads, and internal metal-metal contact (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). A summarized list of advantages and disadvantages for HLMR pot bearings is presented below.

Advantages:

- High load capacity
- Allows for rotation about any axis

Disadvantages:

- Routine maintenance inclusive of sliding surface lubrication and debris removal is necessary to maintain low coefficients of friction
- Sliding surfaces subject to wear
- Exposed steel surfaces susceptible to corrosion (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014)
- Difficult to inspect (Arora and Associates, P.C., 2019)

Maintenance Requirements. Pot bearings should be checked for "leaking" of elastomer which may be indicative of the sealing rings failing (AISC, 2022). As with all bearings using sliding surfaces, PTFE materials should be checked for wear and failure and replaced where needed. Adequate corrosion protection systems are recommended to protect the pot bearing assembly components.

Practical Considerations. The Iowa Bridges and Structures Bureau specifies a maximum service load of 2,500 kips and a max service translation in one direction of 5 inches for HLMR Pot Bearings. AASHTO LRFD requires that pot bearings be designed with a secondary rotational tolerance for installation and fabrication as there is a high potential of hard contact between metal surfaces during construction (AISC, 2022). The state of Minnesota has begun actively avoiding bridge designs where new pot bearings are specified due to the complexity of components, which leads to higher than desired initial maintenance costs. Other states do not permit the use of pot bearings or have a stronger preference toward disc or spherical bearings (Arora and Associates, P.C., 2019).

Construction and Maintenance Costs. The relative installation cost for HLMR disc bearings with units of measurement of "each" is estimated at a minimum cost of $10,734 each, maximum cost of $19,620 each, and a mean cost of $15,197 each. Maintenance costs on pot bearings are in the form of replacing individual components if they have been damaged but primarily the cost of replacing the sliding surface. The estimated mean cost for replacing a PTFE sliding surface would be $596 each plus mobilization and bearing access costs. Costs for jacking and removing bearing components are estimated at a mean cost of $4,443 each.

6.4.3.3 Spherical Bearings

Description and Classification. Spherical bearings are considered the most expensive bearing type, as they require a large amount of material and are complex to fabricate (AISC, 2022). Spherical bearings consist of four main parts: a masonry plate, a convex steel plate that is welded on top, a concave plate with PTFE surface between the spherical interface, and a sole plate. Unlike its HLMR counterparts (Pot and Disc), spherical bearings rely on sliding concave and convex metal surfaces rather than deformation of elastomeric components (Arora and Associates, P.C., 2019; Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). An additional sliding surface can be installed between the concave plate and the sole plate to accommodate translational movement (WSDOT, 2023). If provided adequate clearance, spherical bearings can be designed to accommodate almost any rotational capacity. The bearing's lateral movement is made possible by the sliding surface of the sole plate. The primary utilization for spherical bearings has become bridges with large rotations or rotation about an unknown axis (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). The use of high-grade steel allows for the bearing to be designed to handle loads upwards of 10,000 kips (AASHTO/NSBA Steel Bridge Collaboration, 2023).

Service Life and Performance. Spherical bearings have a service life upwards of 25 years or more (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). To ensure satisfactory performance during its service life, a high degree of quality control during fabrication and field installation is required. Major service life issues remain to be uncertain as no significant problems have been reported other than corrosion on the steel components and wear on the sliding surfaces. A summarized list of advantages and disadvantages for HLMR spherical bearings is presented below.

Advantages:

- Unlimited lateral and rotational movement capacity
- Unlimited load capacity
- Lowest resisting moment of all HLMR bearings

Disadvantages:

- Routine maintenance inclusive of sliding surface lubrication and debris removal is necessary to maintain low coefficients of friction
- Sliding surfaces subject to wear
- Moderate susceptibility to corrosion for exposed steel components
- High initial and maintenance costs
- Most expensive HLMR bearing

Maintenance Requirements.

As with all bearings using sliding surfaces, PTFE materials should be checked for wear and failure and replaced where needed. Adequate corrosion protection systems are recommended to protect the spherical bearing assembly components.

Practical Considerations. The state of California almost exclusively uses spherical bearings for high load applications and is satisfied with their performance (Arora and Associates, P.C., 2019). The PTFE sliding material is the most likely to wear but cannot be seen without disassembling the bearing (Arora and Associates, P.C., 2019). In addition, although either orientation provides the same movement capabilities, most spherical bearings are fabricated such that the concave surface is oriented downward to minimize dirt intrusions (WSDOT, 2023).

Construction and Maintenance Costs. The relative installation cost for HLMR spherical bearings with units of measurement of “each” is estimated at a minimum cost of $17,896 each, maximum cost of $50,000 each, and a mean cost of $30,381 each. Due to limited history of usage, maintenance costs on spherical bearings are not well documented and no performance issues have specifically been reported by the state of California (Arora and Associates, P.C., 2019); however, replacement of steel components that have corroded significantly and replacement of work sliding surfaces could be warranted.

6.4.3.4 High-Load Multirotational Bearing Summary

Table 59 presents a technical summary of all high-load rotational bearings discussed in this section by listing their load capacity, rotation capacity, movement capacity, fatigue performance, and required maintenance levels.

Table 59. High-Load Multirotational Bearing Category Summary

Bearing Type	Load Capacity Range (kips)	Rotation Capacity (rad.)	Movement Capacity	Required Maintenance Level	Mean Installation Cost (Each)
HLMR Disc	100-5,000	0.02 to 0.04	High †	Moderate	$14,145
HLMR Pot	100-5,000	0.02 to 0.04	High †	Moderate	$15,197
HLMR Spherical	No limit	No limit	High †	Moderate	$30,381

† No limit when sliding surface included

High-load multirotational bearings boast the ability to allow rotation and sustain heavy loads, limited only by space and materials. From a technical standpoint, disc and pot bearings perform similarly in capacity ranges for load, movement, and rotation, though most U.S Departments of Transportation favor disc bearings due to the less complex design that facilitates easier maintenance and inspection. Disc bearings relative to other HLMR bearings are more corrosion resistant as there are less exposed steel components. When load capacity needs are high and large rotations are expected, spherical bearings can fit almost any need, though at a high price. All HLMR bearings require careful fabrication and meticulous design to ensure they perform as designed throughout their service life.

6.4.4 Steel Mechanical Bearings

Steel mechanical bearings, or fabricated steel bearings, are the oldest type of bearing discussed. The most common in-service steel mechanical bearings include rockers, rollers, and sliding steel mechanical bearings. Fixed bearings are also utilized and are made from steel. Fixed bearings can take many forms including steel bearing plates anchored to the substructure that are fixed in place on the girder via welds or bolts and pin-type fixed bearings with top and bottom brackets that each bear on a pin that allows rotation. Lateral forces are restrained using either keeper angles, concrete keeper blocks, or bolts. (AASHTO/NSBA Steel Bridge Collaboration, 2023). Steel mechanical bearings typically provide only unidirectional movement. Movements and forces at mechanical bearings most closely match typical idealized boundary conditions in structural design, when compared to other bearing types. They transfer loads through direct metal-to-metal contact. They boast the advantage of being able to extend service life when properly protected and maintained (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). While many existing bridges still utilize rocker bearings, many states have banned their utilization for future designs (Arora and Associates, P.C., 2019). This trend to replace existing steel mechanical bearings with elastomeric ones or HLMRs and eliminating them in new design is largely driven by steel bearings' general tendency to corrode in environmental exposure or topple over when movement limits are exceeded (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). States that experience high seismic activity, like Oregon, Washington, and California, are replacing steel mechanical bearings quickly due to their poor performance and unsatisfactory track record in seismic events (Arora and Associates, P.C., 2019). In addition, the state of New York is also replacing steel rocker bearings and is trying to phase them out.

6.4.4.1 Rocker Bearings

Description and Classification. Steel rocker bearings can be designed using many different configurations. Conventionally bearings that utilize two distinct steel components pinned together with the upper component fixed to the superstructure while the bottom component has a curved lower section such that it can 'rock' across a bearing plate on the pier or abutment are referred to as rocker bearings. Rocker bearings have a load capacity range of 50-750 kips and a high capacity for rotation. In bridges that use rocker bearings for expansion, similarly shaped bearings referred to as fixed shoes or fixed shoe bearings that are pinned together in the same manner, but lack a curved rocking surface and are instead anchored to the substructure, are often used at fixed bearings.

Figure 96. Rocker bearing on US 77 over Missouri River

Figure 97. Fixed shoe on US 61 over Mississippi River

Service Life and Performance. Rocker and fixed shoe bearings, when properly installed and maintained, can have a service life of over 100 years (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). However, the rocker bearings feature steel exposed to the elements and have historically displayed corrosion that causes pins to lock up, inducing large forces and possible damage to abutments. Galvanizing, metalizing, or high-performance paint systems can be utilized to mitigate corrosion threats (Arora and Associates, P.C., 2019). These expansion bearings risk toppling over when facing excessive translations. A summarized list of advantages and disadvantages for steel rocker bearings is presented below.

Advantages:

- Long service life with routine maintenance regiments
- Relatively high load capacity

Disadvantages:

- Dirt, moisture, and debris can build up between surfaces causing the joint to freeze
- Highly susceptible to corrosion
- Potential to topple over when overloaded
- Expensive to fabricate and install
- Routine maintenance requirements of lubrication, cleaning and removing corrosion product and debris

Maintenance Requirements. The surfaces of rocker bearings must be protected against corrosion. Additionally, bearings should be checked for debris and corrosion material built up on the masonry plate as they can limit the rocker movement, increasing the tilt and causing freeze or even tipping (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). If there is only limited corrosion, periodic cleaning and lubrication is recommended (Arora and Associates, P.C., 2019).

Practical Considerations. The Iowa Bureau specifies a maximum service load of 650 kips and a max service translation in one direction of 4-1/2 inches. Iowa DOT no longer permits rocker bearings for new or replacement bridges and is trying to phase them out due to difficulty engaging foundries and fabricators to produce small quantities of bearings, poor performance, and the high maintenance costs. The utilization of rocker bearings has been implemented only as needed to match existing bearings for bridge widening or rehabilitation projects. Similarly, the states of Louisiana, Pennsylvania, and Utah no longer allow steel rocker bearings in any new bridges (Arora and Associates, P.C., 2019). Studies show that steel rocker bearings are the most susceptible to freezing in place (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Rocker bearings are being replaced in many retrofit instances as they have been shown to have poor performance in seismic events (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). States with a high risk for seismic activity are replacing rocker bearings as quickly as feasible. Except for Mississippi and Missouri River crossings and bridge sites with soil properties classified as site class F, the state can be classified in AASHTO Seismic Zone 1, so Iowa is exposed to significantly less rocker bearing tipping risks than high seismic states (Iowa DOT Bridges and Structures Bureau, 2024). If rocker bearings show little corrosion, periodic cleaning and lubrication could be both a satisfactory and cost-effective option (Arora and Associates, P.C., 2019).

Construction and Maintenance Costs. Steel rocker bearings are no longer permitted for new bridges in most states; thus no recent cost data was able to be obtained from online bid tabulations and pay item reports. However, steel rocker bearings remain in service on numerous bridges and the primary maintenance costs are resetting rocker bearings when pack rust and corrosion from lack of maintenance has impeded the tilt of the bearings resulting in reduced movement. Units of measurement for resetting rocker bearings are an "each" quantity with an average minimum cost of $1,500 each, average maximum cost of $7,250 each, and a mean cost of $4,823 each.

6.4.4.2 Other Mechanical Bearings

Description and Classification. While steel rockers and fixed-shoe bearings are the most common mechanical bearings observed in Iowa's inventory, other types exist in the Iowa DOT inventory and are typically custom designed to meet a specific intent. Roller bearings (Figure 98), for example, are frequently found on long span bridges where it is necessary to accommodate both larger expansion and larger vertical loads that an HLMR or elastomeric bearing is capable of. Another example is roller hinge bearings, such as those seen in Figure 99. These allow for a hinge in the superstructure, transferring vertical load but not longitudinal load or bending. The load and expansion capacities of these other types of bearings are typically constrained mostly by the area and height available to place them and the cost. Sliding bronze plate bearings are another example of mechanical bearings still in service and that can be implemented in rehab or widening projects to match the expansions characteristics of the bridge's other bearings.

Figure 98. Roller bearing beneath tied-arch on US 61 over the Mississippi River

Figure 99. Roller hinge bearing on I-80 Eastbound over the Missouri River

Service Life and Performance. Other mechanical bearings reach service lives similar to those of rocker bearings if properly maintained. Roller bearings feature fixed steel tracks open to the elements that typically corrode and collect debris causing rollers or gear teeth to freeze inducing large forces and possible damage to substructures or superstructures. Galvanizing, metalizing, or high-performance paint systems can be utilized to mitigate corrosion threats (Arora and Associates, P.C., 2019). Roller hinge bearings can experience pack rust and failure of retainer device connection, leading to the potential of roll-out of the roller. Bronze sliding plates are also susceptible to freezing or changes in friction coefficient if allowed to collect debris. A summarized list of advantages and disadvantages for steel rocker bearings is presented below.

Advantages:

- Long service life with routine maintenance regiments
- Higher load and expansion capacities that achievable with other bearing types
- Match expansion properties of similar existing bearings

Disadvantages:

- Dirt, moisture, and debris can build up between surfaces causing the joint to freeze
- Highly susceptible to corrosion
- Certain bearings can become risks for support instability if not properly designed or maintained
- Expensive to fabricate and install
- Routine maintenance requirements of lubrication, cleaning and removing corrosion product and debris

Maintenance Requirements. The surfaces of all mechanical bearings require regular cleaning to avoid debris collection and surface corrosion. Surfaces in contact require special consideration during maintenance, and maintenance activities may need to be performed at ambient temperatures to access and clean or coat all parts of the contact surface. Proper detailing in design and routine maintenance are critical to mitigate the risk of larger rehab projects including jacking and resetting bearings or replacement of individual bearing components. For example, weld details at other locations of the roller hinge bearing shown in Figure 99 led to pack rust and cracks on the welds between the roller and the pintle bar, which required retrofit later in the bridges life (Figure 100).

Figure 100. Pack rust and weld cracking at pintle bars in roller hinge bearings on I-80 Eastbound over the Missouri River

Practical Considerations. The Iowa Bureau specifies a maximum service load of 650 kips and a max service translation in one direction of 4-1/2 inches. Iowa DOT no longer permits rocker bearings for new or replacement bridges and is trying to phase them out due to the poor performance issues that have arisen, and the high maintenance costs associated with their preservation. The utilization of rocker bearings has been implemented only as needed to match existing bearings for bridge widening or rehabilitation projects. Similarly, the states of Louisiana, Pennsylvania, and Utah no longer allow steel rocker bearings in any new bridges (Arora and Associates, P.C., 2019). Studies show that steel rocker bearings are the most susceptible to freeze in place (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). Rocker bearings are being replaced in many retrofit instances as they have been shown to have poor performance in seismic events (Azizinamini, et al., Design Guide for Bridges for Service Life, 2014). States with a high risk for seismic activity are replacing rocker bearings as quickly as feasible. However, if the bearings show little corrosion, periodic cleaning and lubrication could be both a satisfactory and cost-effective option (Arora and Associates, P.C., 2019).

Construction and Maintenance Costs. The cost of other mechanical bearings is difficult to tease out for each type, as the bearing sizes and installation effort can vary widely even within the same bearing type. Fabrication costs for steel mechanical bearings are typically higher on a per pound basis than the average structural steel cost due to complex and uncommon geometry and tight tolerances. It is appropriate to consider at least twice the cost per pound for mechanical bearings.

Regular cleaning of these bearing is required, the cost of which will vary greatly depending on access. For long span bridges, access is often provided by ladder and can make routine maintenance less than $1000 per pier of bearings. Spot coating of these bearings in between full bridge repainting projects may also be required. Spot painting prices vary but can be estimated as $150 per square foot for small quantities.

6.4.4.3 Steel Bearing Summary

Steel mechanical bearings are largely being phased out and other bearing types are replacing existing steel mechanical bearings. Steel rocker bearings still remain in service; however, multiple states are actively taking steps to replace them with elastomeric bearings as rocker bearings are highly prone to corrosion and capable of toppling, if not properly maintained or during seismic events. Other mechanical bearing types can provide large load and expansion capacities. These bearings require regular cleaning and recoating, but if this is provided service life longer than either elastomeric or HLMR bearings is achievable.

6.4.5 Current Iowa DOT Practice for Bearings

The Iowa DOT prefers jointless bridge construction when possible, as described in the current Iowa DOT practice for bridge joints section. This preference applies to the most typical bridge types, including concrete slabs, prestressed concrete beam, and steel multi-beam and multi-girder bridges (Iowa DOT Bridges and Structures Bureau, 2024). The Iowa DOT Bridge Standards for integral bridges show either an S3 X 7.5 section or 3-inch by 3-inch bars (or 3-inch square HSS tubes filled with concrete as an alternate) at abutments and 9-inch long by 1-inch thick (parallel to the longitudinal axis of the bridge) neoprene bearing pads at piers supporting the ends of the precast beams prior to diaphragm placement (Iowa DOT, 2023). Neither one of these bearings is expected or required to allow expansion, and both ease landing of the girders during erection. Slab bridge superstructures are built integrally with the abutments and piers, without bearings in all practical cases. Semi-integral standard drawings are not currently available for Iowa DOT, but standard practice in other states is to use steel reinforced elastomeric expansion bearings at the abutments that are protected by a backwall integral with the superstructure and independent of the substructure.

When integral diaphragms at interior piers or abutments are not appropriate and fixed bearings are required for beam or girder bridges, Iowa DOT prefers the curved sole plate bearings seen in standard drawings and discussed briefly in the sliding surfaces section (Iowa DOT Bridges and Structures Bureau, 2024). These bearings are compact, durable, and inexpensive. Similarly, when expansion bearings are required, Iowa DOT prefers steel reinforced elastomeric bearing pads, with or without curved sole plates for prestressed concrete beams and with curved sole plates for steel beams for service translations up to 2 ½ inches from neutral. For larger expansion capacities, up to 5 inches or as specified by manufacturer, Iowa DOT recommends disc or pot bearings, with a preference to disc bearings all else equal.

When attempting to match existing bearing stiffnesses in a widening or bearing replacement Iowa DOT permits the use of rocker or bronze sliding bearings (Iowa DOT Bridges and Structures Bureau, 2024). Bronze sliding plate bearings can also be used if a low profile bearing that exceeds the expansion capacity of similar height elastomeric bearings is required.

6.4.6 Bridge Bearing Design Recommendations

The current Iowa DOT practice of joint elimination whenever practical is also a practice in bearing elimination, and results in significantly less maintenance for these bridges. Semi-integral abutments should continue to be used only in situations that do not lend themselves to integral abutments because of the added construction expense of elastomeric bearings, but also because of added maintenance possibly required to replace sealant materials and repair moisture affected concrete that is more likely in these abutment types.

The fixed bearings used by Iowa are similar to those used in other states and provide a very low life cycle cost. Iowa DOT's policy of favoring steel reinforced elastomeric bearings when expansion is needed, and HLMR disc (preferred) or pot bearings when the expansion demand exceed to capacity available from elastomeric bearings minimizes maintenance requirements over of other bearing options.

Iowa DOT LRFD Bridge design manual does not currently list preferences for expansions exceeding the capacities of HLMR disc and pot bearings. For these cases, the DOT should consider steel mechanical bearings, such as roller bearings, over spherical bearings. Both types of bearings are expensive, both types involve steel components that can result in corrosion, and both types of bearings require cleaning. In addition to these shortcomings, however, spherical bearings involve a PTFE sliding sheet that will require replacement every 15 to 20 years (which can be challenging for a long, heavy span). These PTFE sheets also have a high variability in performance when exposed to minimal amounts of dust and dirt, requiring a designer who is carefully considering demands from the spherical bearings on the substructure to assume wide bounds. Additionally, spherical bearings are not as readily repaired or modified in the field as a roller bearing- full replacement is likely in a situation where a spherical bearing is damaged. Steel on steel contact surfaces provided by roller bearings and other steel mechanical bearings provide the most predictable expansion surface at joints with exceedingly high loads and displacements.

6.5 Foundations

6.5.1 Current State Policy for Bridge Foundations

This section focuses on the types of foundation that are considered under the bridge substructure design by Iowa DOT, Load and Resistance factor Design (LRFD) Bridge Design Manual 2024 (BDM). The soils design unit prepares the soils design package and provides recommendation on the foundation types based on several factors. The parameters that are considered important for the recommendations are also discussed. The literature is then presented to highlight the potential durability issues for commonly used foundation by Iowa DOT. The recommendations based on the current guidelines and the findings from the literature review are used to provide the recommendations at the end of this section.

The Iowa DOT chooses the bridge foundation from three common types: (1) spread footings; (2) drilled shafts; and (3) piles. Spread footings are commonly recognized as shallow foundations while piles and drilled shafts are referred to as deep foundations because the elements are embedded at much greater depths. Deep and shallow foundations transfer structural loads to the soil in different ways. For example, deep foundations transfer the vertical compressive load along the vertical surface area (along the length of pile) in contact with soil (known as skin friction) and at end bearing whereas the lateral load is transferred and redistributed by converting the load into axial compression and uplift (dead weight and skin friction). Spread footings transfer the compressive load by end bearing and the lateral load are balanced by redistribution of bearing pressure.

In general, spread footings can be simply supported on soil or sound rock. When the spread footing is directly placed over the soil or sound rock, the nominal rock bearing resistance (at service and at strength limit state) and the footing elevations are considered critical for the design. A pile or drilled shaft foundation is necessary when the soil beneath the surface is poor in load bearing capacity and greater depth is required to reach a stable soil stratum. Therefore, the geotechnical resistance and target driving resistance become critical for consideration. The current geotechnical resistance charts in revised LRFD 2007 are based on pile load tests and experience from previous charts (1989, 1994 and accepted charts update for trial use in 2006).

Typically, the IowaDOT soils design unit provides bridge soils package that include boring logs, soil profile and supplementals. The supplemental item provides information related to N-value for standard penetration tests and recommendation related to three design factors of slope stability, settlement and foundation type. In addition to the design factors for achieving the service life, the IowaDOT bridge inspection manual requires periodic checking for scour and inspecting substructure elements. Underwater inspection once in 4 years is performed where the water depth is never below 2 feet. And the foundation elements condition can be recorded as unknown when they are not visible. Inspection for visible distress, cracking, section loss, settlement, misalignment, scour, collision damage and corrosion are necessary. The waterway characteristics inspection includes the water levels, scour hole, pile tip, plan streambed, reference point of the elevation used and length of pile. The focus of inspection in the case of steel bents with H piles is on damage from flood debris, fatigue cracking, pack rust, and section loss. Inspection for spalling, scaling, cracking, hollow area of concrete, cracking in high stress area, and flood debris are documented for concrete piers.

The bridge design manual (BDM) recommends different foundation types based on the abutment type and the pier type. A detailed summary of the foundation types recommended based on the type of abutment and pier is provided in Table 60 below. The three types of abutments that are discussed in the IowaDOT BDM are integral, semi-integral and stub. The integral and semi-integral types of abutments can accommodate lateral flexibility for expansion and contraction through foundation. Whereas the stub abutment requires expansion joints in the bridge deck due to the rigidity of foundation. The Bridges and Structures Bureau states that integral and semi-integral abutments are preferred because they decrease maintenance needs by allowing the use of jointless bridge decks. The IowaDOT BDM also recommends changing stub abutments to semi-integral abutments when a stub abutment bridge is undergoing a major repair/preservation project and the change is feasible in terms of cost. Overall, because the IowaDOT prefers to use integral abutments, steel H piles are typically considered for the vast majority of conditions due to their flexibility, capacity, cost and efficiency. If the bed rock is too shallow for steel H piles to be feasible, spread footings or drilled shafts and semi-integral or stub abutment design may be required. Drilled shafts are also permitted for sites where noise and vibration levels are of concern.

Table 60. Iowa DOT recommended foundation types

Substructure	Type	Recommended Foundation
Abutment	Integral abutment	▪ Steel H pile (due to its ability to develop fixity); sections are typically HP 10, HP 12, or HP 14. ▪ Timber piles for bridge lengths up to 200 feet.
	Semi-integral abutment	▪ Two rows of piles: steel H piles, prestressed concrete piles, or timber piles. The piles are typically battered (i.e., pile driven at an anlgle) to 1:4. Deadman anchors to resist lateral forces are required in case battered piles are not possible due to downdrag. ▪ Drilled shaft foundation socketed into rock in case battered piles cannot be used. ▪ Spread footings on sound rock.
	Stub abutment	▪ Two rows of piles (piles type not specified) (back row vertical and front row battered to 1:4). Deadman anchors are required in case battered piles are not possible due to downdrag. ▪ Spread footings on sound rock. ▪ Drilled shafts, when also used for piers.
Piers	Pile bent (used in low-level, short-span bridges in small stream applications)	▪ Driven piles, steel H piles (HP 10, HP 12, HP 14). Galvanized steel H piles can be used under special circumstances under section 8.3.1 of LRFD BDM 2023. ▪ Prestressed concrete piles (14 or 16 inch). ▪ Concrete filled steel pipe piles (14 or 16 inch).
	T-pier (used in grade separation superstructure and bridges under waterways)	▪ Steel H piles, prestressed concrete piles (12 inch), or concrete-filled steel pipe piles. ▪ Spread footings on sound rock. ▪ Drilled shaft foundation socketed into rock that is too shallow for driven piles. Single line or small array of drilled shafts connected by common footings.

Substructure	Type	Recommended Foundation
	Frame piers (used in grade separation structures)	▪ Steel H piles, prestressed concrete piles (12 inch), or concrete-filled steel pipe piles. ▪ Drilled shaft foundation with one drilled shaft per column. ▪ Spread footing on sound rock.
	Diaphragm piers (used in low-level, short-span slab bridges or pretension prestressed concrete beam bridges where bedrock is near the surface)	▪ Steel H piles, prestressed concrete piles (12 inch), or concrete-filled steel pipe piles. ▪ Drilled shaft foundation socketed into rock when rock is too deep for a spread footing. ▪ Line of drilled shafts with a common footing. ▪ Spread footings on sound rock.

6.5.2 Piles

Because jointless bridges with integral abutments are preferred by IowaDOT, piles are used more commonly than the other options (Iowa DOT, 2023). The following sections describing additional selection criteria and material details of foundation types consequently focus on the four different types of piles used by IowaDOT, including steel H piles, concrete-filled pipe piles, timber piles, and prestressed concrete piles.

6.5.2.1 Steel H Piles

Steel H piles are considered feasible for typical Iowa site conditions and desirable because of their flexibility, ability to provide adequate capacity, and relatively low cost. Steel H piles of 10 X 42, 10 X 57, 12 X 53, 14 X 73, and 14 X 117 are considered and required to be Grade 50 steel per ASTM A572/A572M, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*, unless an exception is approved. Four different structural resistance levels (SRL-1, SRL-2, SRL-3, and SRL-4) are defined based on allowable stress levels (6, 9, 12, and 15 ksi, respectively) (Iowa DOT, July 2024). SRL - 4 can be considered only for sites with no known environmental problems, and the deterioration considerations that are detailed in AASHTO LRFD section 10.7.5 (i.e., corrosion of steel pile foundations in fill soil, low pH soil, marine environment; sulfate, chloride and acid attack of concrete pile; decay of timber pile from wetting and drying or from insects or marine borers) have been addressed. The AASHTO LRFD section 10.7.5 also provides a baseline for consideration of indicative of potential pile deterioration based on resistivity, pH (for normal soil and for soil with high organic content), sulfate concentrations, landfills and cinder fills, and soils subjected to mine and industrial damage (AASHTO, 2017).

6.5.2.2 Concrete-Filled Steel Pipe Piles

The Bridges and Structures Bureau generally considers concrete-filled steel piles to be uneconomical for bridges in Iowa, but considers them on a project-specific basis at the request of the contractor. The material for the steel pipes must meet the physical and chemical requirements of Grade 2 or Grade 3 per ASTM A252/A252M, *Standard Specification for Welded and Seamless Steel Pipe Piles*. The maximum limit for standard penetration test number of blows per foot corrected to a hammer efficiency of 60% (N_{60}-values) is 40 for driving pipe piles.

6.5.2.3 Timber Piles

Timber piles in Iowa are allowed for bridges with spans up to 200 feet and are encouraged to be considered for design where site conditions are feasible for their use (Iowa DOT Bridges and Structures Bureau, 2024). Treated or untreated timber piles (depending on wood type as specified in section 1007.08 of Standard Specifications for Road and Bridge Construction) made from sound and solid trees cut within 12 months prior to use. The pile should not consist of any unsound knots or knots in groups, twist of grain exceeding half of the circumference, shake appearing on both sides or more than 1/3 of the diameter of pile, rot, incipient, or advanced decay, season checks (penetrate more than 1/4 of the diameter or the more than 1/4 inch wide). In addition, all the outer bark shall be removed for untreated piles whereas in case of treated pile, all the outer bark and minimum of 80% of inner bark (no stipe remaining on pile over 3/4 inch wide) shall be peeled (Iowa DOT, 2023).

Timber piles are suggested for soils with an $N_{60} \leq 25$. In addition, timber piles cannot be considered for soils having boulders and are not to be used if bearing on rock. The piles are supplied in lengths between 20 and 55 feet, with 5-foot increments. A metal driving shoe, as per the recommendation of the Soils Design Unit, must be fitted for pile driving. Timber piles should not be considered for sites having significant downward drag forces (Iowa DOT Bridges and Structures Bureau, 2024).

6.5.2.4 Prestressed Concrete Piles

Prestressed concrete piles can be considered in case where soil permitting displacement and adequate geotechnical resistances (either through friction or combination of friction and end bearing). Due to difficulties in driving prestressed concrete piles in glacial clay and very firm sandy glacial clays; piles should not be driven where N_{60} values are consistently greater than 30 to 35. The N_{60} value at the tip of the pile should be in the range of 25 to 35 (Iowa DOT, July 2024).

Square-shaped piles with a side length of 12 inches are commonly considered, and 14- or 16-inch square-shaped piles can be considered for pile bents. The maximum length of an individual pile should not exceed 55 feet and pile splices are required to be used as fasteners for site locations where piles longer than 55 feet is necessary to reach sound soil stratum. When the pile is embedded in a footing or stub abutment, the top of the pile is sandblasted to improve the bond between the pile and the other element.

The requirements related to concrete mixtures are minimum of 610 pounds of total cementitious content and maximum water to cementitious ratio of 0.45.The use of cement with equivalent alkali content of 0.61 % to 0.75% is allowed only if aggregates shown to be non-reactive by testing per ASTM C1260, ASTM C1567, or ASTM C1293 are used. The specified concrete compressive strength is 5000 psi at an age of 28 days, and the concrete is required to have a minimum compressive strength of 4000 psi before the piles can be moved. Use of 7-wire strand with ½ inch diameter and 270 ksi grade for prestressing steel (uncoated) and 5 gage wire spiral conforming to ASTM A1064 grade A is allowed. Steel wires with minimum yield strength of 40 ksi are required for spirals and wire ties. The clear cover of 2.5 inches is required and the concrete cover within ± 0.25 inch range of specified cover is allowed. All other structural steel can be grade 36 as per ASTM A709 (Iowa DOT, 2023).

6.5.3 Drilled Shafts

Drilled shafts are often considered for urban sites where noise due to pile driving could be of concern or in the areas where bridge site is close to a building of sensitive nature. In addition, the construction speed for drilled shafts is also faster than other options. The use of drilled shafts over spread footing is feasible on sites where the shales at shallower depth may degrade due to exposure to air.

Drilled shafts can also act as extension of piers for bridge support without footing. And the drilled shafts for bridge support are typically socketed into rock. The thickness of the drilled shafts is increased instead of casting bettered shafts to achieve the lateral load capacity. The shafts are generally designed as tied columns. The 28 days concrete strength can be in range of 4 to 8 ksi and a minimum of 4 ksi is used for structural capacity of shaft design unless a higher strength is specified. The concrete mix design shall imply the requirements for maximum water to cementitious ratio of 0.45 and slump of 8 inches ±1.5 inches. The shafts are reinforced to their full height and the reinforcement details like the cage for round reinforced concrete column with equally spaced vertical bars and spirals. Longitudinal bars of #8 (minimum of 8 bars or bundle in case when larger number of bars are required) or larger can be used. Spirals and ties of #4 or higher shall be used. A minimum clear cover for spiral and tie bars of 1.5 and 1.875 inches are required for bar sizes of #4 and #5, respectively. The use of uncoated and epoxy-coated bars is allowed for bars (Iowa DOT, 2023).

6.5.4 Spread Footings

Footings can be directly supported over stable rock or soil and they can also be supported over piles and drilled shafts. In addition to the load applications, they differ majorly due to the concrete and reinforcement characteristics. In general, spread footings are bigger in size and require additional care related to mass concrete. It is recommended that shrinkage, temperature or skin reinforcement must be provided in case the thickness of the footing is more than 5 feet.

Due to the similarity between bridge substructural elements made with reinforcement concrete and exposed to similar exposure conditions, the recommendations provided in Section 6.1 on Reinforced Concrete Members may be applicable to spread footings.

6.5.5 Deterioration of Piles

This section presents durability-related issues that may be experienced by piles according to existing literature. Because steel H piles are commonly used in Iowa and both steel H piles and prestressed concrete piles can be designed for extended service life, this section focuses on the deterioration and practices for durable design of these two types of piles. While timber piles are also common in Iowa, their limitation to the bridges span (up to 200 feet) and N_{60} requirement of 25 and less may limit their applicability to the provide extended service life with little to no maintenance compared to steel and prestressed concrete piles, and as such timber piles are excluded from the following discussion. Concrete-filled pipe piles are also excluded because they are rarely used in Iowa.

Steel piles corrode over time and can lose their capacity to transfer load. Corrosion of steel occurs under different exposure conditions and at different rates depending on the environmental exposure and the steel composition. In general, the rate of corrosion under sea water or in the areas where chloride salts are used for deicing purposes could be much higher than the rate of corrosion under other soil conditions.

The presence of high concentration of chlorides in sea water is responsible for chloride induced corrosion and the different layers of soils having different concentrations of oxygen can result in corrosion due to differential aeration. The steel surface area in the soil layer with the relatively low concentration of oxygen can act as anode while the cathode is located in the soil with the relatively high oxygen concentration. Literature suggests that the amount of corrosion in undistributed soils could be small due to deficiency of oxygen (Liu, Dahlberg, Phares, & Mousavi, 2022). In contrast, disturbed soils will have greater oxygen availability and result in a more corrosive environment toward steel piles.

The failure of pier number 22 of the Leo Frigo Memorial Bridge on September 25, 2013 in Green Bay, Wisconsin highlights the potential severity of the consequences of steel H pile corrosion (Michael Baker, 2015). The bridge was constructed between 1978 to 1980, and steel H-piles (with varying depths at different locations) were used. Based on the field investigations it was found that the allowable design stress of 14 ksi and 9 ksi for piers on land site and river site, respectively were considered for the design. Based on the details provided in the report published in 2015, it can be summarized that the factors that can impact the rate of corrosion and cause durability related issues were not considered. For example, reduction in daily pool levels and presence of a large stockpile for salt (at least for past several years) was found. The impact of such factors on the durability of steel H were unaccounted for the design consideration. A corrosion rate calculated based on the average section loss at the age of failure was found to be 7 mils per year. The potential corrosion mechanisms are a combination of several factors that may have contributed to section loss and ultimate failure of the piles due to the occurrence of highly unusual environment were:

- Aerobic corrosion occurring at air gaps of porous soils adjacent to the piles. The presence of fly ash in the surroundings was also confirmed
- Galvanic cell soil corrosion occurring between soil layers due to variability in the soil pH, moisture, oxygen levels, and chloride and sulfate concentrations along the length of the piles; and
- Microbiologically influenced corrosion.

Another study sponsored by the Wisconsin DOT showed the effect of different soils on corrosion of steel piles (Poursaee, Rangaraju, & Ding, 2016). A total of 9 soil samples collected from three different locations in Wisconsin were tested for pH, resistivity, chlorides, sulfates, sulfides, and mean total organic carbon (MTOC). Corroded samples as received from Wisconsin DOT and new steel coupons with a composition similar to that of steel H piles were used for the corrosion study. The steel samples were embedded vertically in cementitious mortar with 1 inch depth. The steel and mortar specimens were cured for up to 7 days and then embedded into the soil samples. The tests were conducted in two groups of soils, wherein the first group included as-received soils and the second group included chloride-boosted soils, to which 3% chloride by weight of the soils was targeted. Out of the 9 soil samples, one soil sample was also used with sandblasted specimens. The corrosion potential measurements were collected for up to one year. The researchers found that the physiochemical parameters of the soil did not show a strong correlation with the rate of corrosion of the steel, and highlighted the shortcomings of corrosion potential testing to assess corrosion of steel piles in soil. The researchers also hypothesized that chlorides and other factors have a synergy, resulting in greater corrosion rates than would otherwise be expected. Both the Wisconsin DOT study and the field investigations of the failure of the Leo Frigo Memorial Bridge emphasized the need for inspection techniques for steel H piles that can provide a detailed characterization of corrosion potential.

Several studies have focused on protecting or repairing steel piles using high performance concrete, ultra-high-performance concrete, or coatings such as zinc and paint. As mentioned earlier, the Iowa DOT LRFD Bridge Design Manual allows the use of steel pile bents without encasement if the steel piles are galvanized. However, the use of galvanized steel H pile bents is allowed only for shorter bridges built over streams with minimum ice flow or debris. The bridge located on Buchanan County Road (CR) D-22 over Buffalo Creek was built in 2018 and 2019 and is one of example from field where Iowa DOT considered the galvanized-painted steel H pile bents without encasement. All the piles were galvanized per ASTM A123 and then painted (Liu, Dahlberg, Phares, & Mousavi, 2022).

The Iowa DOT sponsored project report IHRB Project TR-766, *Evaluation of Galvanized and Painted Galvanized Steel Piling*, showed that the use of galvanizing and then painting can significantly lower the rate of corrosion. The laboratory scale study performed on steel coupons and 1-foot sections taken from an HP 10 x 57 pile had a weight loss of approximately 0.4% and 0.3%, respectively, after 600 cycles of corrosion testing, intended to be equivalent to 100 years of exposure in service. The cyclic corrosion testing followed the GMW 14872 procedure. The weight variation (before and after testing) in galvanized-painted steel coupons with coating defects was also found to be approximately 5 times lower than the weight loss of the uncoated coupon sample. A detailed inspection of the galvanized and painted steel H pile bents considered for the bridge located on Buchanan County Road (CR) D-22 over Buffalo Creek was carried out until 2021. The study concludes there were no signs of damage or corrosion at least during the first three years of service.

Many studies have also shown that the chloride threshold for stainless steel could be 7-10 times higher than carbon steel and the use of stainless steel can be a viable option for increasing service life. But high chloride threshold of stainless steel may not directly result in an increase in service life because of other factors such as stress induced corrosion in stainless steel. Further protective measures would therefore be necessary. The use of stainless steel in case of steel H piles may not be practical due to several limitations including the design protocol. Still, as a protective measure, the use of stainless steel in case of steel H piles may not be practical due to several limitations including the design protocol, cost, mechanical properties of stainless steel and limited knowledge of stainless-steel behavior under the exposure conditions like that of steel-H piles in Iowa. Therefore, it would be unpractical to consider the use of stainless-steel piles at this time because of the limitations mentioned earlier. More detailed research work would, therefore, be necessary to consider any such effort.

The use of stainless steel in prestressed concrete piles could be a possible approach that is already being considered by several DOTs and other organizations in the US. The use of stainless steel in bridge decks has been implemented by many DOTs in the US, but a limited number of studies on using stainless steel in prestressed concrete piles are available. While uncommon, prestressed concrete piles using stainless steel prestressing strands have been used in field applications. For example, multiple infrastructure projects in Pearl Harbor, Hawaii have used octagonal prestressed piles with 0.5-inch stainless steel strands. The pile lengths have ranged from 70 to 195 feet, and some have been designed for an ultimate load capacity of 400 to 800 kips. The stainless-steel strands met the requirements of ASTM A276, *Standard Specification for Stainless Steel Bars and Shapes*, for Type XM-29 steel (Mullins, Rajan, & Sagues, 2014).

A study sponsored by the Georgia DOT explored the possibility of using stainless steel in concrete piles. A detailed field investigation conducted on 11 bridges in Georgia showed two main durability problems: (1) surface erosion of concrete due to wave action; and (2) corrosion of reinforcement, which caused cracking in the cover (Moser, Halland, Kahn, Singh, & Kurtis, 2011). Pile samples collected from the field were tested for carbonation to understand the influence of deterioration mechanisms other than chloride-induced corrosion on the deterioration of the piles. The measured chloride profile data indicated that the piles were unlikely to achieve the service life of 100 years, based on the rate of chloride diffusion, and additional measures for more durable concrete mixture design are therefore necessary. The Georgia DOT sponsored study also included a laboratory-scale investigation of the corrosion of A416 prestressing strands and six other stainless steel alloys (Austenitic grade 304 and 316, duplex grade 2101, 2205 and 2304, pearlitic 1080 and Martensitic 17-7). Testing was conducted to analyze the corrosion rates of the steel materials in simulated pore solutions with different chloride concentrations of 0.25, 0.5 and 1.0 M Cl^-. The results showed that the presence of crevices in the strands affects the corrosion resistance when compared with wire specimens. All of the stainless steels tested in the study exhibited good corrosion resistance in the alkaline environment of the simulated pore solution, but the level of corrosion resistance decreased with increasing chloride concentration of the solution used for testing. Grade 2205 consistently demonstrated the greatest corrosion resistance even at 1.0 M CL- while Grade 2304 had low corrosion susceptibility only up to 0.5 M CL- ions concentrations.

Another study sponsored by the Florida DOT compared three different grades of stainless steel under consideration for prestressed piles (Mullins, Rajan, & Sagues, 2014). The grades were 316L, for which seven-wire strands were tested; ASTM A276 Type XM-29, for which seven-wire strands were tested; and 2205, for which single wires were tested. The Corrosion rate of stress corrosion cracking was analyzed by testing single wire from each grade of steel strands under different temperatures, and different solutions intended to represent exposure to ions in seawater or embedment in concrete. The results confirmed the surface cracking of the steel wire samples with high temperature. Grade 2205 when tested at 135 °C for 1 hour also showed surface cracks. The study confirmed that the use of stainless steel in concrete members under high load and variable temperature conditions may behave differently from expected behavior at room temperature and can induce stress corrosion cracking. Inadequate performance of concrete piles reinforced with stainless steel due to stress corrosion cracking has also been documented in case studies from Europe .

Overall, the laboratory investigation to determine the long-term performance of different grades of stainless steels and duplex steels against chlorides shows that the use of stainless steel in prestressed concrete piles and drilled shafts could be another approach for extended design life with minimum maintenance.

6.5.6 Recommendations

- It is recommended to record information related to soil profile for deterioration considerations mentioned in AASHTO LRFD section 10.7.5 at the design stage and during the service life to observe any change in soil conditions due to any external factor.
- Similar to steel piles, steel pile bents are highly susceptible to corrosion at the waterline where steel piles have wetting and drying cycles and ample oxygen availability. Section loss due to the failure of the coating at the waterline is a common problem experienced by steel substructures in Iowa (Bridge

Maintenance Manual). The use of high-performance materials such as ultra-high-performance concrete (for fabrication and for repair) may be suitable to protect the steel H piles at waterline level. Additional research is necessary to develop materials and protocols.

- The laboratory investigation to determine the long-term performance of different grades of stainless steels and duplex steels against chlorides shows that the use of stainless steel in prestressed concrete piles and drilled shafts could be another approach for extended design life with minimum maintenance.
- The lessons learned from the Leo Frigo Memorial Bridge pile failure at Pier number 22 indicate limitations of bridge inspection protocols and the need of method development for monitoring the performance of piles under corrosive environment.

6.6 Approach Systems

6.6.1 Current Practice of the Iowa DOT

The Iowa DOT generally uses a bridge approach system consisting of multiple approach slabs, referred to as "panels," on top of a modified subbase and other geotechnical details designed to facilitate good subdrainage. The details of the bridge approach system used, particularly the panel reinforcement as well as the types of connections between adjacent elements, primarily depend on: (1) the type of route carried; (2) the type of adjacent roadway; and (3) the type of abutment and superstructure.

6.6.1.1 Interstate and Primary Road Bridges

For new bridges on interstates or "primary roads," i.e., roads and streets under the jurisdiction of the Iowa DOT, a three- or four-panel approach system is used when the adjacent roadway is a PCC pavement or an HMA pavement, respectively. Schematics of these systems are shown in Figure 101. The panels are 12 inches thick and the first and second panels, as the motorist moves away from the bridge, are double- and single-reinforced, respectively. The remaining panel(s) is unreinforced concrete. The clear cover for the top and bottom surfaces of the double-reinforced panel is 2.5 inches.

If the abutment is a fixed abutment, then an expansion joint is present between the abutment back wall and bridge deck. The expansion joint is typically a strip seal joint, but may also be a finger or modular joint. On the approach side of the abutment, the double-reinforced panel is tied to the abutment with reinforcing bars. If the abutment is a movable, i.e., integral or semi-integral, abutment, then the double-reinforced panel is isolated from the abutment through a 2-inch expansion joint and asphaltic felt paper is used on the paving notch underneath the panel (not identified in the schematic) to prevent bond. When a moveable abutment is used, the double-reinforced panel has a pavement lug as well.

If the bridge is a continuous concrete slab (CCS) bridge, the double-reinforced panel is tied to the abutment with stainless steel reinforcing bars regardless of whether the abutment is fixed or moveable. The switch to a tied approach for integral abutment, CCS bridges on interstates and the primary roadway system was implemented in the standard drawings maintained by the Iowa DOT in 2016. Schematics for the approach systems used for CCS bridges on interstate and primary roads are shown in Figure 102. They show that for CCS bridges, a sleeper slab is used between the double- and single-reinforced panels and a 2-inch expansion joint is present between the double-reinforced panel and the sleeper slab lug.

6.6.1.2 Secondary Road Bridges

For new bridges on secondary roads, approach systems using a double-reinforced approach slab are encouraged, but approach systems using only a single-reinforced panel followed by unreinforced panels or simply abutting the PCC pavement roadway are permissible. The single-reinforced panel is tied to the abutment if the abutment is fixed and not tied if the abutment is moveable. The Iowa DOT has also developed standard drawings for approach systems intended for bridges on gravel, secondary roads crossing over interstates and primary roads. These systems consist of one double-reinforced approach slab tied to the fixed or integral abutment with dowels or reinforcing bars.

6.6.1.3 Joints in the Approach System

As shown in the schematics, approach systems contain numerous types of joints. The key joints of the system are: (1) the expansion joints accommodating expansion and contraction of the bridge and consequent movement of the abutment and/or approach slab (in the case of moveable abutments); and (2) the pavement pressure relief joints (PPRJs) used to accommodate the expansion and contraction of the roadway pavement relative to the bridge approach. The contraction joints control cracking and can impact the performance of the bridge approach system, but are not usually given as much attention as the expansion joints.

Pertaining to the bridge expansion joints in the approach systems, the Iowa DOT currently uses the joint detail labeled as a 'BE' joint. This detail is reproduced in Figure 103. It consists of a backer rod and tire buffings, and is designed to be 2 inches wide at temperatures between 40 and 80 °F. The use of the 'BE' joint was implemented in October 2024, and prior to its use, the Iowa DOT used a 'CF' joint. The CF joint differed primarily in that it used tire buffings for the full depth of the joint instead of tire buffings and a backer rod, and it could be specified at widths between 2 and 3-1/2 inches, at 1/2-inch increments. The switch from the 'CF' joint to the 'BE' joint was implemented because the Iowa DOT expects the 'BE' joint design to provide better performance.

The type of abutting pavement influences the presence of a pavement pressure relief joint. As shown in Figure 101 and Figure 102, a 3.5-inch, doweled PPRJ is used between the unreinforced panels in the bridge approach system when the abutting pavement is PCC. When the abutting pavement is HMA, no PPRJ is used. The joint between the unreinforced approach slab and the HMA pavement is sawcut 1-1/4 inch deep and sealed.

6.6.1.4 Subbase, Backfill, and Subdrainage

The backfill detail from the standard bridge plans for CCS bridges on interstates and primary roads is shown in Figure 104. The detail shows that a 4-inch diameter subdrain embedded in porous backfill is used near the base of the abutment. The subdrain is required to have a slope of 2 percent to facilitate proper drainage. A geotextile fabric wraps around the porous backfill and subdrain to mitigate the risk of erosion.

Subdrains are also generally installed underneath PPRJs, regardless of the use of a double-reinforced approach slab or a single-reinforced approach slab system. Bridges on secondary roads do not always have additional subdrains underneath PPRJs.

The abutment backfill, which is placed by flooding and vibratory compaction, extends from the top of the porous backfill to the bottom of the modified subbase material, 2 feet below the subgrade elevation, i.e., the bottom of the approach slabs. Because achieving adequate compaction is challenging at locations adjacent to the abutment, the detail identifies the height to which the backfill must be placed before abutment construction begins.

The modified subbase identified in the detail may consist of crushed stone, recycled concrete pavement, recycled asphalt pavement, or a combination of these materials. Based on the detail shown in Figure 104 and the standard road plans, the modified subbase is 2 feet deep and extends to 2 feet beyond the limits of the approach slabs in all directions, except where the roadway begins. A plastic grid, not shown in the bridge detail, is placed underneath the modified subbase. For bridges on secondary roads, a modified subbase and the plastic grid may or may not be used.

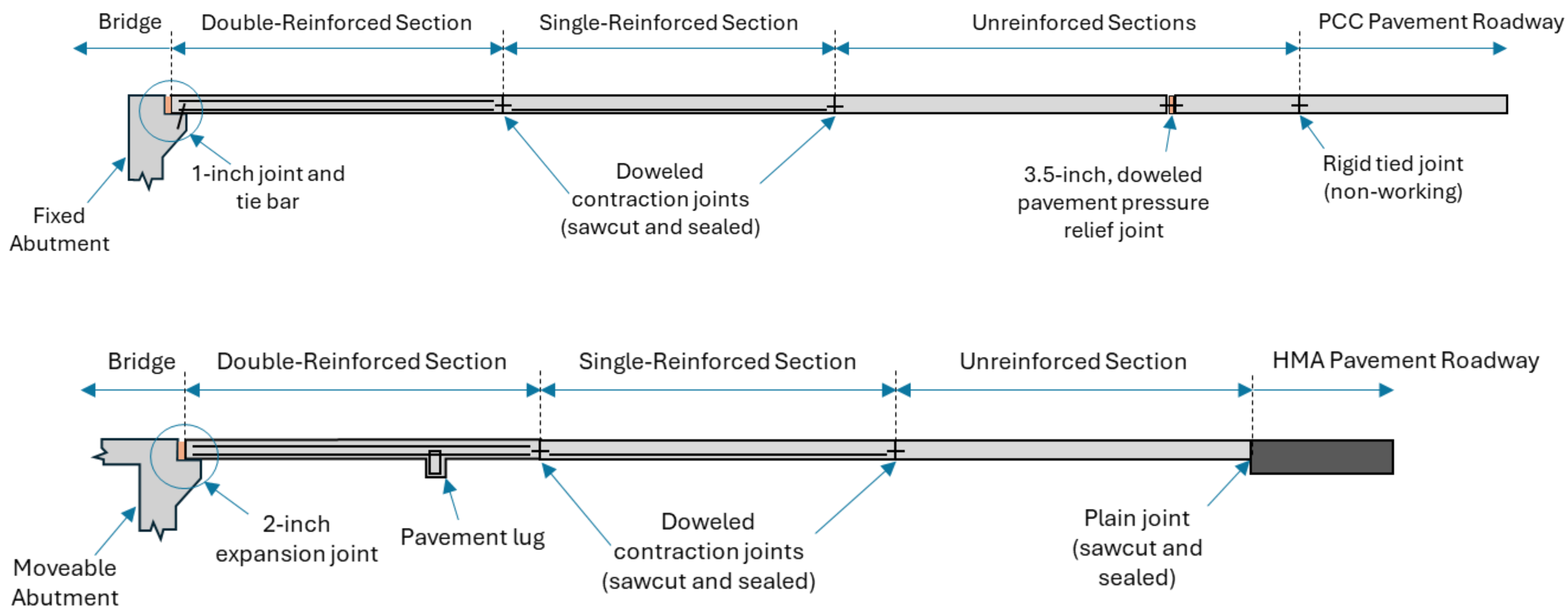

Figure 101. Schematic showing the reinforced and unreinforced concrete slabs and types of joints used in bridge approach systems for bridges carrying interstates or primary roads, excluding continuous concrete slab bridges. The top schematic represents the configuration used with a Portland cement concrete (PCC) pavement roadway and the bottom schematic represents the configuration used with a hot mix asphalt (HMA) pavement roadway.

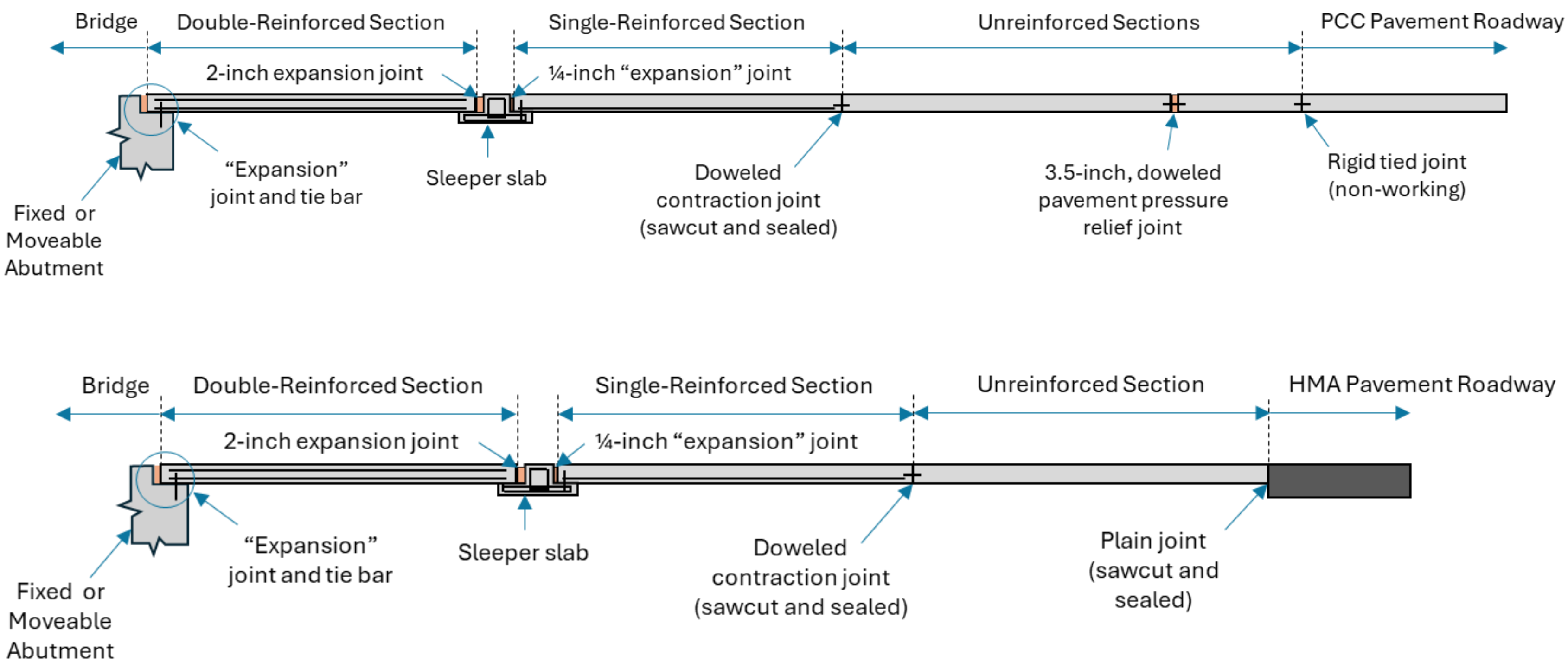

Figure 102. Schematic showing the reinforced and unreinforced concrete slabs and types of joints used in bridge approach systems for continuous concrete slab bridges carrying interstates or primary roads. The top schematic represents the configuration used with a Portland cement concrete (PCC) pavement roadway and the bottom schematic represents the configuration used with a hot mix asphalt (HMA) pavement roadway.

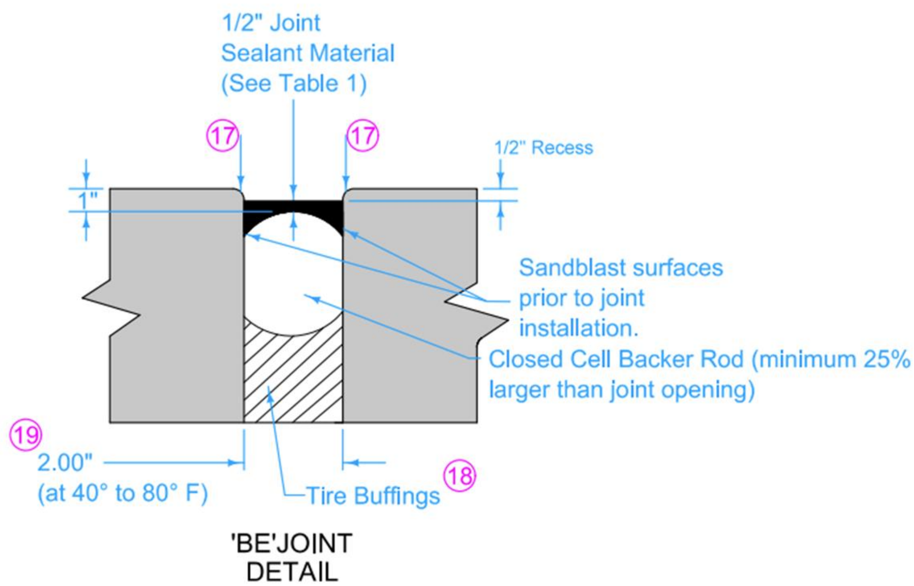

Figure 103. The 'BE' joint detail currently used by the Iowa DOT for expansion joints in bridge approach systems.

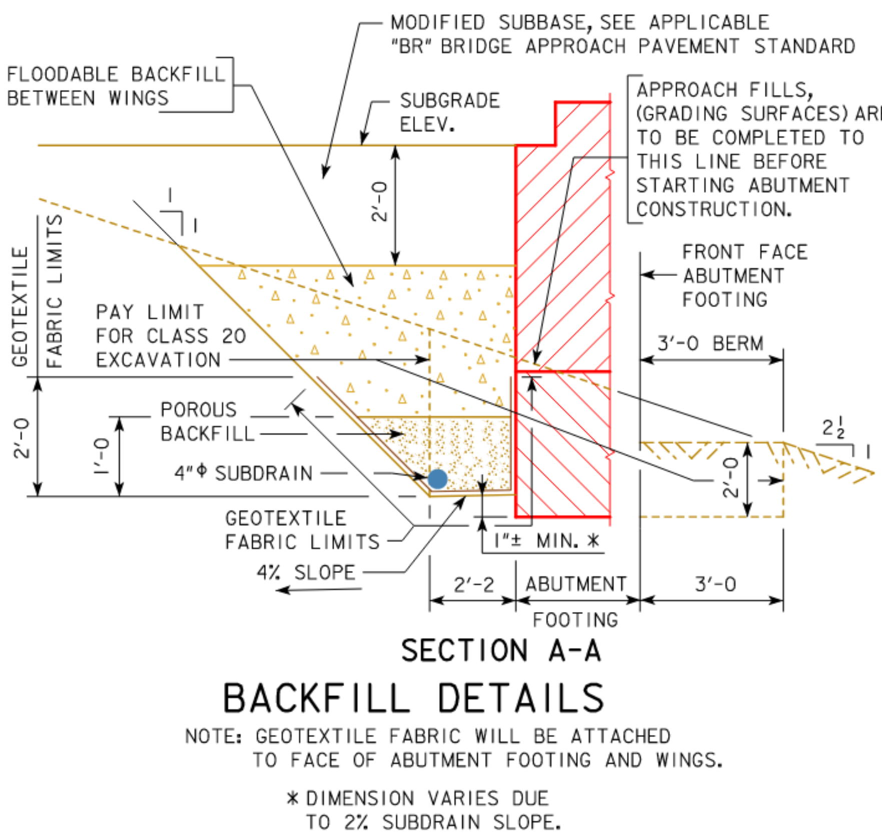

Figure 104. Abutment backfill detail from the standard drawings for CCS bridges on interstates and primary roads.

6.6.2 Deterioration of Bridge Approach Systems

The primary purpose of the bridge approach system is to provide a smooth ride between the roadway and the bridge. While ride quality can be measured and discussed in quantitative terms, poor ride quality is often qualitatively associated with a "bump at the end of the bridge," an issue that has persisted in the United States for decades (Wahls, 1990; ElBatanouny, Hawkins, & Krauss, 2021). Deterioration that causes the ride quality over the approach system to decrease, or deterioration that increases the risk that the ride quality over the approach system will decrease, is of greatest concern to bridge owners and commonly addressed through maintenance.

Poor ride quality over an approach system can develop with time for numerous reasons. While typically associated with differential settlement between the various elements that make up the approach and resulting discontinuities across joints within the approach, other factors associated with the pavement and the structure may play a role in the formation of a bump as well. Based on a synthesis of literature (Ardani, 1987; Wahls, 1990; Schaefer & Koch, 1992; Briaud, James, & Hoffman, 1997; Seo, Ha, & Briaud, 2002), potential contributing factors may generally be categorized into the following four groups (ElBatanouny M. K., Hawkins, Pham, Krauss, & Stauffer, 2023):

1. Deformation of foundation soils, which is commonly associated with the use of heavy approach embankments and/or the presence of soft, compressible, cohesive soils prone to consolidation or movement after construction.
2. Deformation of backfill and embankment soils, which may also experience post-construction consolidation or other movements, especially due to inadequate compaction of the materials, uncontrolled subdrainage, and/or traffic loading.
3. Erosion of soils, caused by inadequate drainage and water control.
4. Unaccommodated relative movement of the structure and pavement.

Poor ride quality caused by the presence of a bump and approach settlement has been an issue across Iowa in the past, and considerable resources were needed to address these problems, typically by void filling with grout and the application of asphalt concrete overlays (White, Sritharan, Suleiman, Mekkawy, & Chethur, 2005). As a result, the Iowa DOT initiated a field study to evaluate the causes of the poor performance of bridge approaches; identify design, construction, and maintenance practices that could ameliorate the issue of approach settlement and poor ride quality at Iowa bridge sites; and develop quantitative thresholds that should trigger maintenance (White, Sritharan, Suleiman, Mekkawy, & Chethur, 2005). The study was conducted between 2002 and 2005 and included field evaluations of 65 existing bridges as well as inspection of 8 new bridges under construction during the study. The researchers identified voiding and erosion underneath bridge approaches, attributed to insufficient backfill compaction and moisture control as well as poor subdrainage; ineffective sealing of expansion joints due to the use of flexible foam and recycled tire joint fillers; poor surface drainage; and poor quality construction of the paving notches supporting the approach slabs at the bridge end. As a result of the findings, White et al. (2005) recommended the following on a trial basis:

- Improve the subdrainage behind the abutment by using porous backfill, a vertical geocomposite drain, geotextile reinforcing, or a 1- to 2-foot thick layer of elastic tire chips, or a combination of the potential options.

- Improve the surface drainage details on the approach by providing inlets on the approach that are not prone to blockage by debris.
- Require less than 60% of granular backfill materials to pass the No. 8 sieve and apply moisture control limits of 8 to 12 percent when the granular backfill material is being placed.
- Eliminate the expansion joint at the bridge end of the approach by connecting the approach slab to the bridge deck or abutment.
- Support the roadway end of the approach slab on a sleeper beam and use a 2-inch construction joint.
- Improve the joint sealing system of the bridge expansion joints.

A later study was initiated in 2017 to evaluate the effectiveness of the recommendations implemented by the Iowa DOT as a result of the former study, and consisted of field evaluations of eight bridges between 3 and 12 years of age at the time of inspection (ElBatanouny M. K., Hawkins, Rende, & Krauss, 2018). The study concluded that the approaches of the inspected bridges were generally in good condition. While voiding underneath the approach slabs, particularly at integral abutment bridges, was still occurring, elevation survey data indicated that the bridge approaches were performing adequately. However, failed or missing sealant in the expansion joints and PPRJs continued to be observed as well as spalling and raveling around the joints. The inspected approaches of integral abutment bridges also typically had large gaps between the barrier walls and the approach slabs due to differential movement between the approach slabs and the barriers. These gaps were expected to be contributing to water leakage and consequent erosion. Based on the findings, ElBatanouny et al. (2018) recommended the following design modifications or alternatives:

- Compare the maintenance needs of stub and integral abutment bridges to determine which design should be preferred.
- Decrease the number of joints in the approach system, e.g., by considering a design that carries the approach slab over the abutment and therefore removes the joint between the abutment and the slab.
- Cast the barriers on top of the approach slab to eliminate the joint that results in a large gap between the barriers/wingwalls and the approach slab to mitigate water leakage and erosion.

Readers are referred to both studies for further details pertaining to the performance of and deterioration experienced by bridge approach systems specific to Iowa (White, Sritharan, Suleiman, Mekkawy, & Chethur, 2005; ElBatanouny M. K., Hawkins, Rende, & Krauss, 2018). The need for these studies is shown by the programmed work records within SIIMS for 2001 to 2021. Based on this list, 210 bridges have required bridge approach repair, resurfacing of their approaches with HMA, or replacement of the bridge approach pavement and/or paving notch, the latter three of which are commonly conducted in response to poor ride quality. The number of instances of these items, shown in Table 61, sums to 214 because some bridges were considered for two different scopes of programmed work, e.g., HMA resurfacing and replacement of the bridge approaches, although both items were not necessarily done.

Table 61 also shows the number of instances in which bridge approaches were recommended for various types of maintenance activities in the dataset for in-house forces. Of the recorded instances, 46% were pavement repair of one or both approaches, 19% were repair of the shoulders or shoulder panels at one or both approaches, 19% were miscellaneous recommendations, and 16% were re-cutting or installation of PPRJs at one or both approaches. Based on the associated comments, the need for pavement repair may be reported due to a variety of different forms of distress, including but not limited to:

- Settlement and observable undermining or erosion underneath or around the approach;
- Poor ride quality due to pavement drop-off;
- Cracking or spalling of the portland cement concrete approach slabs;
- Cracking or potholes in asphalt concrete overlays;
- Spalling of asphalt concrete overlays adjacent to joints;
- Cracking or deterioration of the adjoining roadway pavement;
- Gaps between the barrier rails and the approach slabs;
- Wheel rutting of portland cement concrete; and
- Distress associated with the sidewalk belonging to the approach.

In contrast, of the comments for shoulder or shoulder panel repair that described the type of distress present, 60% described erosion or undermining and 46% described the need for sealing, usually of the gaps between the shoulder pavement or panels and the barriers or wingwalls as described by ElBatanouny et al. (2018). Sealing of the pavement-barrier gap and erosion were commonly identified in the same item, and inspectors sometimes recommended gap sealing as a preventive measure to mitigate the risk of erosion in the future. Despite the predominant comments on erosion, settlement was only observed in 4% of the comments. The presence of spalls in the shoulders or broken or cracked shoulder panels was only identified in approximately 4% of the comments as well.

Table 61. Summary of Bridge Approach Maintenance Records in Iowa DOT SIIMS Database as of 2021

Maintenance Description	No. of Instances in Dataset
Maintenance Recommendations List (2011 to 2021)	
Re-cut or install PPRJs	242
Repair approach pavement	685
Repair shoulders or shoulder panels	287
Misc. recommendations for approaches	283
Programmed Work Records (2001 to 2021)	
Bridge approach repair	72
HMA resurfacing of bridge approach pavement	72
Replace bridge approach pavement	43
Replace pavement notch and bridge approach pavement	27

Comments pertaining to recommendations coded as re-cutting or installing PPRJs are also relatively focused. The PPRJ is generally required to have a width of at least 2 inches, except when the bridge is an integral abutment bridge, in which case the sum of the expansion joint for the deck and the PPRJ is required to be at least 2 inches (HDR, 2014). Inspectors' comments commonly identify inadequate widths. In 14 instances, the inspectors reported that the PPRJ(s) had been covered or filled with HMA.

There is significant overlap between the distress and maintenance needs described in the comments belonging to the category of miscellaneous recommendations and those described in the comments belonging to the other three categories. The miscellaneous recommendations include comments pertaining to pavement drop-off, the need for sealing, erosion, and insufficient PPRJs, and additionally include items relating to signage and vegetation.

6.6.3 Preventive Measures

Best design practices for mitigating bridge approach settlement and the formation of a bump were compiled by ElBatanouny et al. (2023) and include:

- Conducting site-specific subsurface investigations to validate foundation soil properties assumed in settlement and stability analyses;
- Applying a factor of safety of at least 1.5 in the stability analysis;
- Including temporary surcharges in the construction schedule, and/or placing the embankment as early in the construction sequence as possible;
- Minimizing the embankment height;
- Using lightweight embankment materials;
- Using increased compaction requirements, especially near the abutment;
- Using good quality control procedures during and after construction, e.g., to verify that the embankment soil properties assumed in settlement and stability analyses were achieved;
- Implementing reinforced soil embankments;
- Deferring the placement of a permanent approach slab by using temporary asphalt pavement until the majority of post-construction settlement or movement has occurred;
- Using a long, strong, full-width approach slab;
- Specifying maximum lift heights, density, and moisture control when placing backfill and embankment soils;
- Improving abutment geometry to facilitate easier access and better compaction of abutment backfill;
- Installing erosion-resistant surfacing on slopes;
- Implementing good surface and subsurface drainage measures;
- Using select, pervious backfill with a limited fines content;
- Designing joints to minimize water leakage and runoff from accessing the backfill and embankment;
- Including elastic materials, compressible inclusions, or vertical voids between integral abutments and the backfill to accommodate integral abutment movement; and
- Incorporating pavement growth joints when abutting concrete and HMA pavements to accommodate pavement expansion.

6.6.4 Recommendations

It is generally recommended that the Iowa DOT consider implementing the recommendations developed by White et al. (2005) and ElBatanouny et al. (2018) that have not yet been incorporated into their standard practice on a trial basis. The Iowa DOT should monitor performance to identify the modifications that result in improved performance and should be widely implemented. As an exception to this, the elimination of the barrier-approach slab joint is a relatively low-risk, high-reward modification and is therefore recommended to be implemented with a short trial period to validate the constructability of this design.

Other recommendations developed based on the maintenance data from SIIMS are:

- Safeguard against over-paving of PPRJs with hot mix asphalt by developing procedures requiring documentation of the presence of a PPRJ within the limits of bridge or roadway work and requiring the contractor to re-install any PPRJs that were paved over during the work.
- Develop improved surface drainage details that will mitigate the risk of erosion of the shoulders, embankment, and berm slope by controlling the locations where water from the bridge or approach is released. A study may be needed to identify the water management solutions considered cost-effective for routes of varying levels of importance.

As good practice, reinforced concrete elements within the approach system, including but not limited to the reinforced concrete approach slabs and the sleeper slabs, should be designed considering the recommendations in Section 6.1. While unreinforced concrete is not at risk of deterioration due to chloride-induced or carbonation-induced corrosion, the concrete materials used should still be selected considering freeze-thaw deterioration, alkali-aggregate reaction, sulfate attack, and other forms of concrete material degradation not associated with steel reinforcement as described in Section 6.1.

6.7 Berm Erosion

Erosion is a process wherein soil and rock particles are carried across the surface due to wind, water and/or ice. Though this movement is a natural process, human intervention can exacerbate the rate of erosion. Flow of water through channels and watershed (wet erosion) or heavy winds (dry erosion) causes movement of soil/rock. Removal of topsoil/vegetation, improper surficial shrubs or plant cover and/or lack of compaction of surficial soil are some of the main construction related activities which can increase the amount and speed of soil erosion (Iowa DOT, 2022). With excessive erosion, the stability of a slope reduces and can progressively deteriorate leading to sloughing and at times to complete failure. More specifically, erosion of the toe of a slope is major area of concern for slope stability.

6.7.1 Current Policy

The current Iowa DOT berm slope design policy under normal situations allows for certain slopes based on the height of the fill within the berm (Iowa DOT, 2023). For fills less than 30 feet, the allowable maximum slope of the berm is 2.5:1, horizontal to vertical. For fill heights between 30 to 40 feet, the slope of the berm can be 3:1, and for fills above 40 feet, the slope of the berm is estimated by the soils design unit. The toe of the berm is typically designed to be a minimum of 4 feet away from the edge of the adjacent shoulder to improve snow removal operations.

The design of berm protection is based on the location of a bridge. For bridges over a roadway, slope protection is typically done with the use of macadam stone or concrete. For bridges over waterways, riprap and erosion stone is recommended.

6.7.2 Deterioration Mechanisms

Erosion is largely influenced by the erodibility of the soil, shear stress of the soil, water velocity, and the geometry of the obstacle (Briaud, 2008). The erodibility of the substrate is proportional to the shear stress at the water-substrate interface which is dependent on the particle size of the substrate, density, and the cohesive properties of the substrate. Fine sand and non-plastic silt have a higher rate of erosion compared to coarser sand, high plasticity silt, and low plasticity clay. Coarse gravel and high plasticity clay have lower erodibility, while riprap and jointed rock have very low erodibility (Briaud, 2008). Smaller particles generally have lower shear stress in submerged conditions and are more susceptible to erosion. The velocity of water increases when it is obstructed and has to flow around an obstacle in order to maintain its flow rate. This acceleration and turbulence can increase erosion.

Poor surface drainage conditions can also lead to erosion. The water draining from the bridge deck, basecourse, and upper layers of an embankment can cause washout of the fines and lead to berm erosion, slope instability (Long, Olson, Stark, & Samara, 1998) Volume changes due to freeze-thaw and swelling/collapse of certain soils, and excessive pressure are also possible due to inefficient drainage. Poor surface and subsurface drainage can also cause excessive settlement or reduction in soil strength leading to erosion related concerns (Mekkawy, White, Suleiman, & Sritharan, 2005). Saturation of soil can also lead to instability of the slopes and increase rate of erosion.

Stability of slopes are also influenced by other factors such as removal of support, surcharge, external disturbances, and uplift (National Highway Institute, 2008). Excavations activities near the toe of a slope can lead to instability of the slope, additional dead load (waste dump or snow on top of slope), seismic activities (earthquake, pile driving, heavy vehicles), and changes to ground water table are all examples of such factors.

6.7.3 Preventive Measures

The flow of water plays an important role in maintenance of berms. The control of water drainage on the surface and within the subsurface is important for the service life of berm slopes. Regular inspection and routine maintenance of the drainage system such as ditches, pipes, detention basins, and edge drains must be completed to check for accumulation of soil, vegetation, or any other debris. Failure to clear blockages can lead to backing up of water, leading to overflowing conditions and thereby causing saturation of slopes. Particular attention should be given to low lying areas including abutment slopes with potential for water logging and ponding – e.g. drains to lead water away from the slopes could be installed (FHWA NHI 08-098).

Expansion joints at the ends of a bridge and in pavements should be sealed to prevent drainage of water into the berm or provisions can be made to divert water away from the joints and into the underlying soils (Mekkawy, White, Suleiman, & Sritharan, 2005).

Selection of appropriate backfill material is crucial to limit erosion of the berm. Material gradations with low frost susceptibility along with low scour/erosion potential can be used during construction and/or any subsequent repairs. An engineered fill with coarse granular material and high internal friction is recommended (Dupont & Allen, 2002).

In general, sufficient (surface and/or subsurface) drainage to account for seasonality, soil and groundwater conditions along with the geometry of the slopes should be designed and periodically improved, as necessary.

6.7.4 Recommendations

The following recommendations can be helpful in reducing the maintenance required to control berm erosion:

- Control of water flow by taking measures to prevent clogging of the drainage system. This can include installation of geotextile membranes, sedimentation basins, and filters to reduce the movement of soil and other debris. By preventing the clogging of existing drainage systems, their effectiveness can be increased.
- Regular inspection and maintenance of joint sealers including necessary sealing of joints to prevent water leakage into the soil.
- Good design practices during construction; restricting the height and slope of the berm. The taller a berm is, the greater its dead load, which can lead to slope instability. A steep slope is also not recommended. Mechanically stabilizing/reinforcing slopes and end-slope protection using concrete slabs, paving blocks, gravel, riprap, and heavy stones are other important design considerations (Wahls, 1990).
- Selection and the use of appropriate backfill material that has low frost susceptibility, high friction, and low soil erodibility.
- Vibrations (adjacent construction) should be limited.

- Other approaches include using vegetation to prevent erosion and wetting of the berm surface to reduce drying in summer months that could lead to wind related erosion concerns.

CHAPTER 7. SUMMARY AND RECOMMENDATIONS

7.1 Summary

The goal of this study is to implement durable bridge designs that require minimal maintenance within the exposures found across Iowa during the bridges' target service lives. A literature review, analysis of the bridge maintenance records within the SIIMS database used by the Iowa DOT, and survey of local experience across Iowa were conducted in support of this goal, and durable bridge designs or best design practices for addressing the types of deterioration experienced in the exposure conditions across Iowa were identified. The designs were validated through service life modeling or quantitative performance prediction when feasible.

The analysis of the bridge maintenance records was conducted in order to identify the most common or costly bridge maintenance activities conducted in Iowa, which aids in understanding the maintenance needs to be addressed by the implementation of more durable designs. The Iowa DOT maintains two maintenance lists in SIIMS, one identifying programmed work and a second identifying the maintenance recommendations from bridge inspectors. The list of programmed work spanned 2001 to 2021 and the list of maintenance recommendations spanned 2011 to 2021. The lists were combined to identify which bridge component, i.e., the deck, superstructure, bearings and substructure, pavement, or "miscellaneous," most commonly needs maintenance and which maintenance activities are most commonly performed for each component. The analysis also investigated the sensitivity of the results to the average daily truck traffic of the bridges.

To supplement the findings from the bridge maintenance records analysis, a survey of bridge deterioration and maintenance was distributed across the districts of the Iowa DOT. The survey inquired about the common types of deterioration encountered locally, the maintenance that typically needed to be done, and the maintenance activities that were desirable to avoid. The district engineers were encouraged to share the survey with local agencies to cross-check the findings of the analysis of bridge maintenance records with local experience.

Durable designs that minimize maintenance in Iowa exposure environments were identified by applying a service life design methodology. A literature review of current practice for bridge service life design in the United States was conducted to inform the approach. Based on the findings, three levels of "target service life" were selected: (1) Normal, which corresponds to 75 years; (2) Enhanced, which corresponds to 100 years; and (3) Maximum, which corresponds to 125 years. These target service lives were chosen in order to decrease the likelihood of maintenance within the first 50 years of service, the length of service for which the Iowa DOT communicated that minimizing maintenance needs is particularly desirable. This approach assumes that the risk of unintended maintenance within the first 50 years decreases with increasing target service life.

The maintenance record analysis and survey indicated that durable designs are desirable for the following bridge elements:

- Reinforced concrete elements, specifically decks, barriers, girders, pier caps, and columns;
- Steel superstructures;
- Joints;

- Bearings;
- Foundations;
- Approach systems; and
- Berms.

Durable designs for the target service lives corresponding to Normal, Enhanced, and Maximum life were developed and their performance validated by service life modeling using WJE CASLE™ or quantitative performance prediction where feasible, i.e., reinforced concrete elements and steel superstructures. In support of the service life modeling, the chloride exposure assumptions suitable for Iowa sites were verified through chloride profile testing of cores collected from ten bridges across Iowa.

The service life design methodology could not be applied for all of the elements, either because no technology currently exists that can reduce the maintenance needs of the elements (i.e., joints and bearings) and/or because not enough is known to model the deterioration and expected service life of the element (i.e., foundations, approach systems, and berms). In these cases, the current standard design of these elements and their performance were reviewed and best design practices for avoiding deterioration or minimizing the deterioration rates of the elements were identified for the Iowa DOT to consider.

7.2 Future Research Needs

The durable designs identified for each bridge element focus on the potential of the design to provide the desired service life and minimum maintenance requirements and do not necessarily consider the life-cycle cost, constructability of the design or the additional work needed to develop local expertise, suitable quality control procedures, or guidance for the successful installation of the design. Additionally, for some of the identified technologies, further research of their performance is recommended to improve the reliability of the performance predictions and identify more specific designs worth development. Key research needs are:

- This study does not include life cycle cost analysis and as such, it is possible that there exist scenarios where designing for the 'normal' service life option of 75 years and regular maintenance of the bridge components thereafter could lead to a more cost effective solution than designing for an 'enhanced' or 'maximum' service life.
- Alternative strategies for bridge deck protection including development of concrete mixes with limited potential to cracking and use of two-course decks, a deck with overlay placed at time of construction, using materials such as ultra high-performance concrete
- Use of alternative reinforcing bars including corrosion resistance steel, stainless steel, and fiber reinforced polymer (FRP) bars.
- Understanding the chloride threshold for Class C, D, and E reinforcement. Current thresholds in the literature are generic and more research is needed in this area.
- Characterization of the corrosion resistance of ASTM A709 Grade 50CR in deicing chemical environments unsuitable for uncoated weathering steel and the needs for partial coatings under joints and at other locations prone to experiencing relatively aggressive exposure compared to the general bridge superstructure.

- Investigation of the feasibility of galvanizing weathering steel and ASTM A709 Grade 50CR steel and the impact of the galvanizing process and its presence on the corrosion resistance of these steels and their ability to form a patina when the substrate is exposed.
- Definition of the underclearance, flooding frequency and duration, and other exposure conditions of "low-level water crossings" that make the site unsuitable for uncoated weathering steel because of elevated moisture levels, and investigation of the performance of alternative designs, such as galvanized or metallized steel, in these environments.
- Verification of the expected performance of duplex coating systems in exposure environments present across Iowa.
- Further investigation of best practices or development of guidance for the successful construction of ASTM A709 Grade 50CR superstructures, galvanized weathering steel, and duplex coating systems.

7.3 Implementation

The content of this report is intended to be used as general guidelines for Iowa DOT to choose different design strategies to minimize maintenance of bridge elements based on a targeted service life approach. This will aid in minimizing maintenance during early stages of bridges life depending on the target service life selected during design. Portions of this report can be used by Iowa DOT to create a guide for service life design for bridges, similar to the guidance provided by the Guide Specification for Service Life Design of Highway Bridges, HBSLD-1 (AASHTO, 2020) and MnDOT Service Life Design Guide for Bridges (MnDOT, 2022). Additional information is provided in this report to classify exposure categories in Iowa based on salting practices along different Iowa road types. This information can be used in conjunction with design recommendation of this report to optimize the selection of design requirements for Iowa bridges.

A summary of the sections of this report that can be used to develop a guide for service life design for Iowa bridges is provided below.

- Classification of exposure zones can be found in Section 6.1, Section 6.2, and Appendix A.
- Guidance for service life design of reinforced concrete elements can be found in Section 6.1.
- Guidance for service life design of steel superstructures can be found in Section 6.2.
- Guidance for selection of joints and bearings can be found in Section 6.3 and Section 6.4, respectively.
- General consideration for foundations can be found in Section 6.5.
- General recommendations for approach slabs and berm erosion can be found in Section 6.6 and Section 6.7, respectively.

CHAPTER 8. BIBLIOGRAPHY

AASHTO. (2017). *AASHTO LRFD Bridge Design Specifications* (8th ed.). Washington, DC: American Association of State Highway and Transportation Officials.

AASHTO. (2020). *Guide Specification for Service Life Design of Highway Bridges* (1st ed.). Washington, DC: American Association of State Highway and Transportation Officials.

AASHTO/NSBA Steel Bridge Collaboration. (2023). *Steel Bridge Bearing Guidelines*. Washington, D.C.: American Association of State Highway and Transportation Officials and National Steel Bridge Alliance.

ACI Committee 117. (2015). *ACI SPEC-117-10: Specification for Tolerances for Concrete Construction and Materials*. Farmington Hills, MI: American Concrete Institute.

ACI Committee 201. (2023). *ACI PRC-201.2-23: Durable Concrete - Guide*. Farmington Hills, MI: American Concrete Institute.

ACI Committee 207. (2021). *ACI PRC-207.1-21: Mass Concrete - Guide*. Farmington Hills, MI: American Concrete Institute.

ACI Committee 301. (2020). *ACI SPEC-301-20: Specifications for Concrete Construction*. Farmington Hills, MI: American Concrete Institute.

AGA. (2012). *Hot-Dip Galvanizing for Corrosion Protection A Specifier's Guide*. Centennial, CO: American Galvanizers Association.

AGA. (n.d.). *Time to First Maintenance*. Retrieved November 1, 2024, from Hot-Dip Galvanizing (HDG): https://galvanizeit.org/hot-dip-galvanizing/how-long-does-hdg-last/in-the-atmosphere/time-to-first-maintenance

AISC. (2022). *Steel Bridge Design Handbook*. American Institute of Steel Construction.

AISC Marketing, Inc. (1993). Uncoated Weathering Steel Bridges. In I. AISC Marketing, *Highway Structures Design Handbook* (Vol. Vol. I). Pittsburgh, PA.

Albrecht, P., Coburn, S. K., Wattar, F. M., Tinklenberg, G. L., & Gallagher, W. P. (1989). *NCHRP Report 314 Guidelines for the Use of Weathering Steel in Bridges*. Washington D.C.: Transportation Research Board.

Andrade, C., & Alonso, C. (2004). Chapter 5, Electrochemical Aspects of Galvanized Reinforcement Corrosion. In *Galvanized Steel Reinforcement in Concrete* (p. 122). Elsevier.

ArcelorMittal. (2013, September). Plate Duracorr/A1010: Bridge Applications and Fabrication Guidelines. ArcelorMittal.

Ardani, A. (1987). *Bridge Approach Settlement*. Denver, CO: Colorado Department of Highways.

Arora and Associates, P.C. (2019). *Experiences in the Performance of Bridge Bearings and Expansion Joints Used for Highway Bridges*. Lawrenceville.

ASTM Standard C1778, 2022, Standard Guide for Reducing the Risk of Deleterious Alkali-Aggregate Reaction in Concrete. (2022). West Conshohocken, PA: ASTM International. doi:10.1520/C1778-22

Ault, J. P. (2023). *NCHRP RR 1048 Corrosion Protection of Steel Bridges Using Duplex Coating Systems.* Washington, DC: National Academies Press.

Ault, J. P., & Dolph, J. D. (2018). *NCHRP Synthesis 517: Corrosion Prevention for Extending the Service Life of Steel Bridges.* Washington, D.C.: The National Academies Press.

Azizinamini, A., Power, E. H., Myers, G. F., & Ozyildirim, H. C. (2014). *Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems, and Components.* Washington, D.C.: The National Academies Press.

Azizinamini, A., Power, E. H., Myers, G. F., Ozyildirim, H. C., Kline, E. S., Whitmore, D. W., & Mertz, D. R. (2014). *Design Guide for Bridges for Service Life.* Washington, D.C.: The National Academies Press.

Baker Engineering & Energy. (2006). *Evaluation of Various Types of Bridge Deck Joints.* Pheonix: Arizona Department of Transportation.

Bamforth, P. (2004). *Enhancing reinforced concrete durability: Guidance on selecting measures for minimising risk of corrosion of reinforcement in concrete, Concrete Society Technical Report No. 61.* Surrey, UK: The Concrete Society.

Bektas, B. A., & Albughdadi, A. J. (2018). *Characteristics of Decommissioned Bridges.* Ames, IA.

Berndt, M. L., & Berndt, C. C. (2003). Thermal Spray Coatings. In *ASM Handbook Volume 13A, Corrosion: Fundamentals, Testing, and Protection* (p. 803 to 813). Materials Park, OH: ASM International.

Bolluyt, J., Kau, V., & Greiman, L. (2001). *Performance of Strip Seals in Iowa Bridges, Pilot Study.* Ames: Iowa Department of Transportation.

Bowman, M. D., Hagan, B. D., & Hurdle, W. D. (2022). *Steel Bridge Coating Evaluation and Rating Criteria.* Indianapolis, IN: Indiana Department of Transportation.

Breit, W. (1997). Untersuchungen zum kritischen korrosionsauslösenden Chloridegehalt für Stahl in Beton (Dissertation in German). *Schriftenreihe Aachener Beiträge zur Bauforschung*.

Briaud, J.-L. (2008). Case Histories in Soil and Rock Erosion: Woodrow Wilson Bridge, Brazos River Meander, Normandy Cliffs, and New Orleans Levees. *Journal of Geotechnical and Geoenvironmental Engineering*.

Briaud, J.-L., James, R. W., & Hoffman, S. B. (1997). *NCHRP Synthesis 234 Settlement of Bridge Approaches (The Bump at the Ends of the Bridge).* Washington, D.C.: National Academy Press.

BSSA. (n.d.). *Calculation of Pitting Resistance Equivalent Numbers (PREN)*. Retrieved November 5, 2024, from British Stainless Steel Associtaion: https://bssa.org.uk/bssa_articles/calculation-of-pitting-resistance-equivalent-numbers-pren/

Caicedo, J. M., Weiger, G., Ziehl, P., & Rizos, D. (2011). *Simplifying Bridge Expansion Joint Design and Maintenance.* Columbia: University of South Carolina.

Caltrans. (2010). *Protection of Reinforcement Against Corrosion Due to Chlorides, Acids, and Sulfates. Memo to Designers 10-5.* Department of Transportation, State of California.

Chang, L.-M., & Lee, Y.-J. (2001). *Evaluation and Policy for Bridge Deck Expansion Joints*. West Lafayette: Indiana Department of Transportation and Purdue University.

Collins, R. (2018, March 20). Hot-Dip Galvanized Steel Bridges. American Galvanizers Association.

Corrosiveness of Various Atmospheric Test Sites as Measured by Specimens of Steel and Zinc. (1968). *Metal Corrosion in the Atmosphere* (pp. 360-391). Philadelphia, PA: ASTM.

Cramer, S. D., & Covino, Jr., B. S. (Eds.). (2003). *ASM Handbook Volume 13A, Corrosion: Fundamentals, Testing, and Protection* (Vol. 13A). Materials Park, OH: ASM International.

Crampton, D. D., Holloway, K. P., & Fraczek, J. (2013). *Assessment of Weathering Steel Bridge Performance in Iowa and Development of Inspection and Maintenance Techniques*. Ames, IA: Iowa Department of Transportation.

D.S. Brown. (2013, January). *New Construction Expansion Joint Solutions*. Retrieved from D.S. Brown Expansion Joint Selection Guide.

D.S. Brown. (2016, August). *Maintenance Expansion Joint Solutions*. Retrieved from D.S. Brown Expansion Joint Selection Guide.

D.S. Brown. (2020, August). *Matrix 502 Asphaltic Expansion Joint System*. Retrieved from D.S. Brown Installation.

D.S. Brown. (2021, March). *New Construction Expansion Joint Systems*. Retrieved from D.S. Brown Expansion Joint Selection Guide.

D.S. Brown. (2022, September). *Structural Bearing Assemblies*. Retrieved 2023, from D.S. Brown Product: https://dsbrown.com/wp-content/uploads/2020/12/B_Bearings_Pot_Elasto_Disc_Isolation_BRO_v005.pdf

Dornsife, R. J. (2000). Expansion Joints. In L. Duan, E.-F. Chen, W.-F. Chen, & L. Duan (Eds.), *Bridge Engineering Handbook*. Boca Raton: CRC Press LLC.

Dupont, B., & Allen, D. (2002). *Movements and settlements of highway bridge approaches*. Kentucky Transportation Center.

ElBatanouny, M. K., Hawkins, K. A., & Krauss, P. D. (2021). *NCHRP Synthesis 566 Practices for Bridge Approach Systems*. Washington, D.C.: National Academy of Sciences.

ElBatanouny, M. K., Hawkins, K. A., Abdelrahman, M. A., Lawler, J. S., Nelson, T. D., & Krauss, P. D. (2022). *Guide to Remediate Bridge Deck Cracking, Report TR-782*. Ames, Iowa: Iowa Highway Research Board.

ElBatanouny, M. K., Hawkins, K. A., Pham, L., Krauss, P. D., & Stauffer, P. (2023). *Improvement of Approach Smoothness on Integral Abutment Bridges*. Pierre, SD: South Dakota Department of Transportation.

ElBatanouny, M. K., Hawkins, K. A., Rende, N. S., & Krauss, P. D. (2018). *Performance Evaluation of Recent Improvements of Bridge Abutments and Approach Backfill*. Ames, IA: Iowa Highway Research Board.

Ellor, J. A., Young, W. T., & Repp, J. (2004). *NCHRP Report 528: Thermally Sprayed Metal Coatings to Protect Steel Pilings: Final Report and Guide*. Washington, D.C.: The National Academies Press.

FHWA. (1989). *Technical Advisory 5140.22, Uncoated Weathering Steel in Structures*. Washington, D.C.: Federal Highway Administration.

FHWA. (2018). *Bridge Maintenance Reference Manual*. Retrieved from Florida Department of Transportation, Office of Maintenance.

FHWA. (2022, May 25). *National Bridge Inventory (NBI) - Based on the SNBI*. Retrieved from Bridges & Structures: https://www.fhwa.dot.gov/bridge/pubs/Memo-Implementation_Specifications_National_Bridge_Inventory_FINAL.pdf

fib. (2006). *Bulletin No. 34, Model Code for Service Life Design*. Lausanne, Switzerland: Fédération internationale du béton.

Fischel, M. (2001). *Evaluation of Selected Deicers Based on a Review of the Literature*. Colorado Department of Transportation.

Fletcher, F. B. (2011). *Improved Corrosion-Resistant Steel for Highway Bridge Construction*. McLean, VA: Federal Highway Administration.

Fletcher, F. B., Townsend, H. E., & Wilson, A. D. (2003). Corrosion Performance of Improved Weathering Steels for Bridges. Orlando, FL: National Steel Bridge Alliance World Bridge Symposium.

Fu, C., & Angelilli, C. (2007). *Investigation of the Performance of Elastomeric Bearings on Maryland Concrete Bridges*. Baltimore: Maryland State Highway Administration.

Guthrie, W. S. (2005). Performance of Concrete Bridge Deck Joints.

Hannigan, P., Rausche, F., Likins, G., Robinson, B., & Becker, M. (2016). *Design and Construction of Driven Pile Foundations, FHWA-NHI-16-009 (Volume I), FHWA-NHI-16-010 (Volume II), and FHWA-NHI-16-064 (Volume III)*. Federal Highway Administration (FHWA).

Hassiotis, S., Khodair, Y., Roman, E., & Dehne, Y. (2006). *Evaluation of Integral Abutments*. Hoboken: New Jersey Department of Transportation.

HDR. (2014). *Bridge Inspection Manual*. Iowa Department of Transportation.

Helsel, J. L., & Lanterman, R. (2022). Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work. *AMPP Annual Conference + Expo*. San Antonio, TX: Association for Materials Protection and Performance.

Hoppe, E., Weakley, K., & Thompson, P. (2016). Jointless Bridge Design at the Virginia Department of Transportation. *Transportation Research Procedia, 14*, 3943-3952.

Iconic pedestrian bridge features stainless steel. (2012, April 19). *Roads & Bridges*. Retrieved from https://www.roadsbridges.com/bridges/article/10707298/outokumpu-stainless-inc-iconic-pedestrian-bridge-features-stainless-steel

IMOA. (2018, January). Innovative bridge uses stainless steel. *MolyReview*. Retrieved from https://www.imoa.info/download_files/molyreview/excerpts/18-1/Innovative_bridge.pdf

Indiana Department of Transportation. (2024). *2013 Design Manual*. Retrieved November 21, 2024, from in.gov: https://www.in.gov/dot/div/contracts/design/Part%204/Chapter%20409%20-%20Abutment,%20Bent,%20Pier,%20and%20Bearing.pdf

Iowa DOT. (2018, July 30). *Bridge Standards - Microstation V8 (8" Deck, F-Shape)*. Retrieved from Iowa DOT: https://iowadot.gov/bridge/Bridge-and-Culvert-Standards/Bridge-V8i-Standards

Iowa DOT. (2020, January 31). *Construction Manual - Chapter 10 Environmental*. Retrieved from Electronic Reference Library (ERL): https://iowadot.gov/erl/current/CM/content/CM%2010.50.htm

Iowa DOT. (2022). *Erosion & Sediment Control Field Guide*. IowaDOT.

Iowa DOT. (2023). *Bridge Standards*. Retrieved November 21, 2024, from iowa.dot.gov: https://iowadot.gov/bridge/standards/IntegralBridges.pdf

Iowa DOT. (2023). *LRFD Design Manual.* Iowa DOT.

Iowa DOT. (2023). *Standard Specifications for Highway and Bridge Construction*. Ames, IA: Iowa Department of Transportation.

Iowa DOT Bridges and Structures Bureau. (2024). *LRFD Bridge Design Manual*. Ames: Iowa DOT.

Iowa DOT. (n.d.). *County Bridge Standards - Microstation V8*. Retrieved 02 10, 2025, from Iowa Department of Transportation: https://iowadot.gov/bridge/Bridge-and-Culvert-Standards/County-Bridge-Standards

Iowa DOT. (July 2024). *LRFD Bridge Design Manual*. Iowa Department of Transportation Bridges and Structures Bureau.

IowaDOT. (2019). *2019 - 2028 Transportation Asset Management Plan*. Ames, IA: Iowa Department of Transportation.

ISO. (2012). *ISO 16204: Durability – Service life design of concrete structures.* International Organization for Standardization (ISO), Geneva, Switzerland.

ISO. (2012, February 1). ISO 9223 Corrosion of metals and alloys - Corrosivity of atmospheres - Classification, determination and estimation. Switzerland: International Organization for Standardization.

ISO. (2012, February 1). ISO 9224 Corrosion of metals and alloys - Corrosivity of atmospheres - Guiding values for the corrosivity categories. Switzerland: International Organization for Standardization.

ISO. (2017, November). ISO 12944-2 Paints and varnishes - Corrosion protection of steel structures by protective paint systems Part 2: Classification of environments. Switzerland: International Organization for Standardization.

Kelly, V. R., Findlay, S. E., Schlesinger, W. H., Menking, K., & Chatrchyan, A. M. (2010). *Road Salt: Moving Toward the Solution.* Cary Institute of Ecosystem Studies.

Knudsen, O., Matre, H., Dorum, C., & Gagne, M. (2019). Experiences with Thermal Spray Zinc Duplex Coatings on Road Bridges. *Coatings*.

Kogler, R. (2015). *Steel Bridge Design Handbook: Corrosion Protection of Steel Bridges*. Washington, D.C.: Federal Highway Administration.

Kulicki, J., Wassef, W., Mertz, D., Nowak, A., Samtani, N., & Nassif, H. (2015). *Bridges for Service Life Beyond 100 Years: Service Limit State Design*. Washington, D.C.: SHRP 2 Report S2-R19B-RW-1, Transportation Research Board of the National Academies.

Lawler, J. S., Kurth, J. C., Garrett, S. M., & Krauss, P. D. (2021). Statistical Distributions for Chloride Thresholds of Reinforcing Bars. *ACI Materials Journal, 118*(2), 13-20.

Lehman, D. E., Roeder, C. W., Larson, R., & Curtin, K. (2003). *Cotton Duck Bearing Pads: Engineer Evaluation and Design Recommendations*. University of Washington, Department of Civil and Environmental Engineering, Seattle, Washington.

Liu, Z., Dahlberg, J., Phares, B., & Mousavi, S. (2022). *Evaluation of Galvanized and Painted Galvanized Steel Piling, TR-766*. Iowa Highway Research Board.

Long, J., Olson, S., Stark, T., & Samara, E. (1998). Differential Movement at Embankment-Bridge Structure Interface in Illinois. *Transportation Research Record*.

Main, R. (1967). Michigan's Galvanized Bridge - First in the United States. *Highway Research Record*, 29 - 35.

Malla, R. B., Shaw, M. T., Shrestha, M. R., & Boob, S. (2006). *Sealing of Small Movement Bridge Expansion Joints*. Technical Report, Storrs.

Mameng, S. H., Backhouse, A., McCray, J., & Gedge, G. (2018). Experience of duplex stainless steels as structural materials for bridges. *IOP Conf. Series: Materials Science and Engineering. 419*. IOP Publishing.

McConnell, J., Shenton III, H. W., Bai, T., & Rupp, J. T. (2024). *Weathering Steel Performance Data Collection*. McLean, VA: Federal Highway Administration.

McCrum, R. L., & Arnold, C. J. (1985). *Current Status Report, Effects of Corrosion on Unpainted Weathering Steel Bridges*. Lansing, MI: Michigan Department of Transportation, Testing and Research Division.

McDad, B., Laffrey, D. C., Dammann, M., & Medlock, R. D. (2000). *Performance of Weathering Steel in TxDOT Bridges*. Texas Department of Transportation.

McEleney, B. (2005, October). A Primer on Weathering Steel. *Structure Magazine*.

Mekkawy, M., White, D., Suleiman, M., & Sritharan, S. (2005). Simple Design Alternatives to Improve Drainage and Reduce Erosion at Bridge Abutments. *Proceedings of the 2005 Mid-Continent Transportation Research Symposium*. Iowa State University.

Michael Baker, J. I. (2015). *I-43 Leo Frigo Memorial Bridge Investigation Report*. Wisconsin Department of Transportation.

Miller, A. M., & Jahren, C. T. (2014). *Rapid Replacement of Bridge Deck Expansion Joints Study - Phase I*. Ames: Institute for Transportation Iowa State University.

MnDOT. (2022). *MnDOT Service Life Design Guide for Bridges*. MnDOT.

Morandeira, D., Miller, E., & Jahren, C. T. (2020). *Rapid Bridge Deck Joint Repair Investigation - Phase III*. Ames: Institute for Transportation Iowa State University.

Morley, J., & Bruce, D. (1983). *Survey of steel piling performance in marine environments.* European Commission: Document EUR 8492 EN.

Moser, R., Halland, B., Kahn, L., Singh, P., & Kurtis, K. (2011). *Durability of Precast Prestressed Concrete Piles in Marine Environment: Reinforcement Corrosion and Mitigation.* Office of Materials and Research, Georgia Department of Transportation .

Mullins, G., Rajan, S., & Sagues, A. (2014). *DESIGN AND CONSTRUCTION OF PRECAST PILES WITH STAINLESS REINFORCING STEEL* . Florida Department of Transportation.

Nahra, M., Walton, R., & Rost, J. (1999). *Hydro-Surface Preparation and Coating for Painted Structural Steel.* Ames, IA: Iowa Department of Transportation.

National Cooperative Highway Research Program. (2003). *Bridge Deck Joint Performance: A Synthesis of Highway Practice.* Washington, D.C.: Transportation Research Board.

National Highway Institute. (2008). *Highway Slope Maintenance and Slide Restoration, NHI Course No. 132081.* FHWA.

NCHRP. (2024, October 14). TPF-5(493) Investigation of Dual Grade / Hybrid Steel Plate Girders Utilizing Stainless Steel. Retrieved from https://www.pooledfund.org/Details/Study/723

NCHRP Project 12-108. (2020). *Web-Only Document 269: Guide Specification for Service Life Design of Highway Bridges.* Washington, D.C.: The National Academies Press. Retrieved from https://doi.org/10.17226/25672

Nejad, S., & McGormley, J. C. (2024). *Design Coefficients of Friction for MoDOT PTFE Bearings.* Wiss, Janney, Elstner Associates, Inc., MIssouri Department of Transportation, Federal Highway Administration.

Nickel Institute. (2014). Guidelines for Corrosion Prevention. In *Stainless Steels in Architecture, Building and Construction.* Nickel Institute.

Ocel, J. (2021). *Historical Changes to Steel Bridge Design, Composition, and Properties.* Washington, D.C.: Federal Highway Administration.

Phares, B., Shafei, B., & Shi, W. (2020). *Evaluation of the Performance of A1010 Bridge Steel.* Ames, IA: Iowa Department of Transportation.

Poursaee, A., Rangaraju, P., & Ding, L. (2016). *Evaluation of H-pile Corrosion Rates for WI Bridges Located in Aggressive Subsurface Environments.* Wisconsin Department of Transportation.

Provines, J. T., Sharp, S. R., & Moruza, A. K. (2023). *Documentation and Evaluation of ASTM A709 Grade 50CR Steel Bridges on Virginia's Eastern Shore.* Richmond, VA: Virginia Department of Transportation.

Provines, J. T., Sharp, S. R., Ozbulut, O., & Daghash, S. (2019). *Maintenance-Free Corroion-Resistant Steel Plate for Bridges.* Richmond, VA: Virginia Department of Transportation.

Revie, R. W. (Ed.). (2006). *Uhlig's Corrosion Handbook, 2nd Ed.* Hoboken, NJ: John Wiley & Sons, Inc.

Roberge, P. R. (2006). *Corrosion Basics, An Introduction (2nd Ed.).* Houston, TX: NACE International.

Schaefer, V. R., & Koch, J. C. (1992). *Void Development Under Bridge Approaches.* Pierre, SD: South Dakota Department of Transportation.

Seo, J., Ha, H., & Briaud, J.-L. (2002). *Investigation of Settlement at Bridge Approach Slab Expansion Joint: Numerical Simulations and Model Tests.* Austin, TX: Texas Department of Transportation.

Sharp, S. R., Provines, J. T., Moruza, A. K., Via, Jr., W. F., & Harrop, K. N. (2019). *Virginia's First Corrosion-Resistant ASTM A1010 Steel Plate Girder Bridge.* Richmond, VA: Virginia Department of Transportation.

Shenton III, H. W., & Mertz, D. R. (2016). *Guidelines for Maintaining Smll Movement Bridge Expansion Joints.* Newark: University of Deleware.

Short Span Steel and Bridge Alliance. (n.d.). PennDOT Uses Corrosion-Resistant Steel for Flex Beam Pilot Bridge Project.

SSPC. (2011). *SSPC Protective Coatings Glossary: acronyms and Terms Related to Coating Industrial Steel and Concrete Structures as well as Failure analysis and Regulations.* Pittsburgh, PA: SSPC.

SSPC. (2012, November 8). Technology Guide No. 19 Selection of Protective Coatings for Use Over Galvanized Substrates. The Society for Protective Coatings.

Stanton, J., Roeder, C., Helnwein, P., White, C., Kuester, C., & Craig, B. (2008). *NCHRP Report 596: Rotation Limits for Elastomeric Bearings.* Washington D.C: Transportation Research Board of the National Academies.

Stephens, M. T., Gleeson, B., Mash, J., & Li, B. (2019). *Steel Bridge Corrosion Prevention and Mitigation Strategies: Literature Review.* Pittsburgh, PA: IRISE Consortium.

Taylor, D., Ebrahimpour, A., Ibrahim, A., & Mashal, M. (2024). *Development of a Research Roadmap for ITD's Bridge Section.* Boise, ID: Idaho Transportation Department.

Thomas, M. D., & Bentz, E. C. (2000). Life-365 Computer Program for Predicting the Service Life and Life Cycle Costs of Reinforced Concrete Exposed to Chlorides.

Thomas, M., Fournier, B., & Folliard, K. J. (2013). *Alkali-Aggregate Reactivity (AAR) Facts Book.* Washington D.C.: Federal Highway Administration.

U.S. DOT. (2000). *Comprehensive Truck Size and Weight Study Volume II Issues and Background.* U.S. Department of Transportation.

UFGS. (2012). *Section 03 31 29 Marine Concrete, Division 03-Concrete*. Unified Facilities Guide Specifications (UFGS).

Van Eijsbergen, J. F. (1994). *Duplex Systems: Hot-dip Galvanizing Plus Painting.*

Vinik, P., Bottenberg, R., Brown, C., Ocel, J., Schwerdt, T., Todsen, M., & Palle, S. (2016). *Scan 15-03 Sucessful Preservation Practices for Steel Bridge Coatings.* Lawrenceville, NJ: Arora and Associates, P.C.

VTrans, I. A. (2009). *Integral Abutment Bridge Design Guidelines* (second ed.). Montpelier: State of Vermont, Agency of Transportation.

Wahls, H. E. (1990). *NCHRP Synthesis 159 Design and Construction of Bridge Approaches.* Washington, D.C.: National Academy of Sciences.

Wan, B., Foley, C. M., Ainge, S. W., & Nguyen, C. (2013). *Procedures, Cost and Effectiveness for Deteriorated Bridge Substructure Repair.* Madison, WI: Wisconsin Department of Transportation.

Wasserman, E., & Walker, J. (1996). *Integral Abutments for Steel Bridges, Highway Structures Design Handbook.* Chicago: American Iron and Steel Institute.

West, J., Laroshce, C., Koester, B., Breen, J., & Kreger, M. (1999). *State-of-the-Art Report about Durability of Post-Tensioned Bridge Substructures.* Austin, TX: Center for Transportation Research, The University of Texas at Austin.

White, D., Sritharan, S., Suleiman, M., Mekkawy, M., & Chethur, S. (2005). *Identification of the Best Practices for Design, Construction, and Repair of Bridge Approaches.* Ames, IA: Iowa Highway Research Board.

Wilson, M. L., & Tennis, P. D. (2021). *Design and Control of Concrete Mixtures* (17th ed.). Skokie, IL: Portland Cement Association.

WSDOT. (2023). *Bridge Design Manual (LRFD).* Olympia: Washington State Department of Transportation.

Zhengyu, L., Dahlberg, J., Phares, B., & Mousavi, S. (2022). *Evaluation of Galvanized and Painted Galvanized Steel Piling,* . Iowa Highway Research Board.

APPENDIX A. CONCRETE CORE SAMPLING AND CHLORIDE PROFILE FITTING

Concrete cores were collected from ten bridge decks across Iowa and their chloride concentration profiles were determined to develop and validate inputs into the service life modeling performed for the reinforced concrete elements (see Section 6.1). The locations of the ten bridges, identified by their FHWA Number, are shown in Figure A.1.

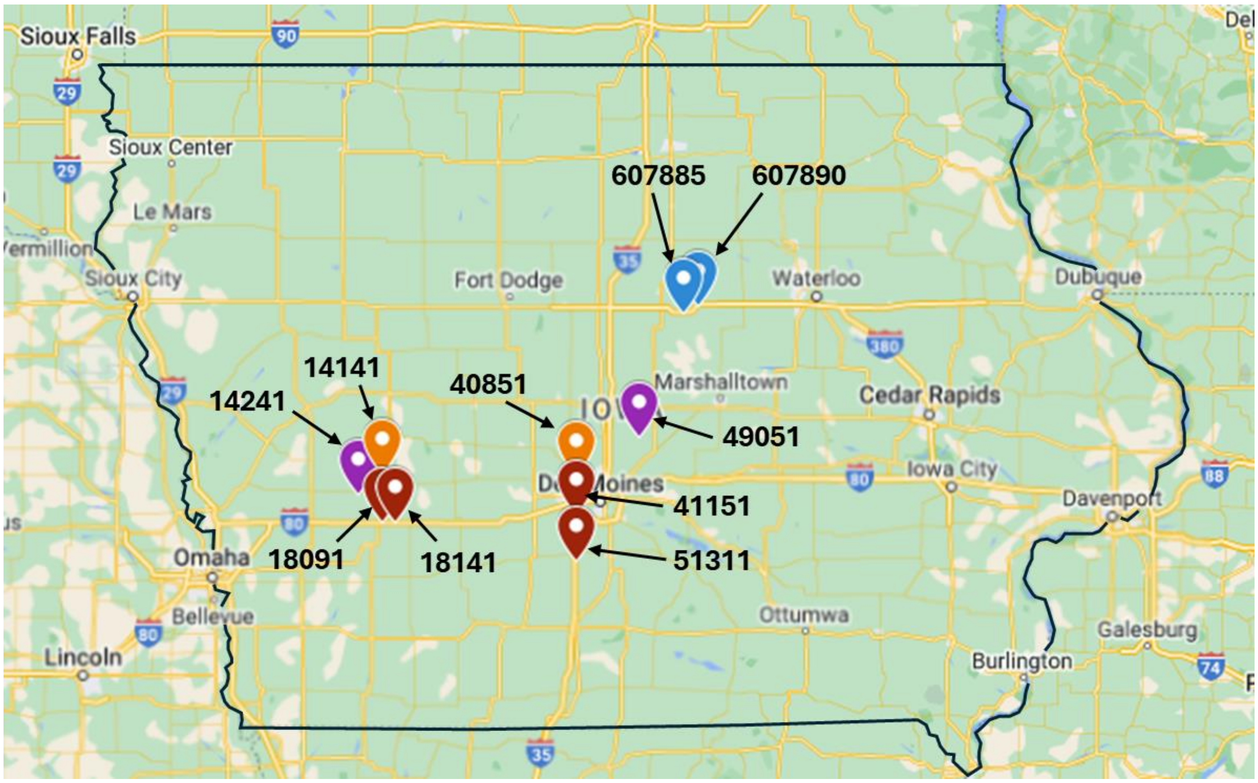

Figure A.1. Locations of the ten bridges from which deck cores were collected for chloride profile measurements. Each bridge is identified by its FHWA Number. The marker color indicates the type of route: red = interstate; orange = US or state highway on the NHS; purple = US or state highway that is not on the NHS; and blue = county highway that is not on the NHS.

The ten bridges were chosen to represent various types of routes and locations expected to experience different levels of deicing chemical application. Four types of routes, ranging from interstates to county highways off of the National Highway System (NHS), were represented. All ten bridges in this study were located in suburban or rural locations. The details of the bridges, including the type of route, the facility carried, the feature(s) intersected, and the average daily traffic (ADT) and average daily truck traffic (ADTT), are presented in Table A1. The bridges ranged from 15 to 20 years of age at the time of the core sampling.

WJE visited the ten bridges and sampled cores between October 9, 2023, and October 24, 2023. Table A2 summarizes the number of cores collected from each bridge deck, the general location from which they were collected (i.e., the lane and location with respect to the wheel paths), and the date on which the cores were extracted. All cores had a nominal diameter of 3-5/8 inches and typically ranged between 4 and 5 inches in length. A total of 81 cores were collected.

Table A1. Description of Locations and Traffic Conditions of Bridges from which Deck Cores were Collected

Bridge No.	FHWA No.	Year Built	Type of Route	Facility Carried by Structure	Feature(s) Intersected	Location Description	ADT (vehicles)	ADTT (%)	ADTT (vehicles)
1	607890	2003	County Highway (non-NHS)	S 56	US 20	Rural, agricultural lands. Agricultural processing unit nearby.	1453	16	232
2	607885	2003	County Highway (non-NHS)	D35	US 20	Rural, agricultural lands	717	12	86
3	49051	2003	US/State Highway (non-NHS)	IA 210	Indian Creek	Rural, agricultural lands. Town (Maxwell) nearby.	1660	6	100
4	40851	2006	US/State Highway (NHS)	EB/SB IA 141	Little Beaver Creek	Suburban	9800	6	588
5	41151	2003	Interstate	SB I-35	Raccoon River	Suburban	28800	14	4032
6	51311	2007	Interstate	SB I-35	Clanton Creek	Rural	11400	27	3078
7	14241	2003	US/State Highway (non-NHS)	IA 173	Indian Creek	Rural, agricultural lands. Town (Kimbalton) nearby.	1360	11	150
8	14141	2006	US/State Highway (NHS)	US 71	Bluegrass Creek	Rural, agricultural lands. Town (Audubon) nearby. Near high school.	3080	14	431
9	18091	2008	Interstate	WB I-80	E Nishnabotna River	Rural	11550	42	4851
10	18141	2003	Interstate	EB I-80	Troublesome Creek	Rural	11500	42	4830

Table A2. Summary of Collected Cores

Bridge No.	FHWA No.	Date Cored	Total No. of Cores	Lane from which Cores were Collected	No. in a Wheel Path	No. Between Wheel Paths	No. in Shoulder
1	607890	10-09-2023	7	SB lane & adjacent shoulder	3	2	2
2	607885	10-09-2023	7	NB lane & adjacent shoulder	3	2	2
3	49051	10-10-2023	10	EB lane & adjacent shoulder	5	2	3
4	40851	10-10-2023	10	SB passing lane & adjacent shoulder	4	3	3
5	41151	10-11-2023	9	Leftmost SB lane & adjacent shoulder	5	2	2
6	51311	10-11-2023	9	SB driving (right) lane & adjacent shoulder	4	3	2
7	14241	10-23-2023	7	NB lane & adjacent shoulder	2	2	3
8	14141	10-24-2023	7	SB lane & adjacent shoulder	3	2	2
9	18091	10-23-2023	8	WB passing (left) lane & adjacent shoulder	4	2	2
10	18141	10-24-2023	7	EB driving (right) lane & adjacent shoulder	3	2	2

Water-soluble chloride concentrations were measured with depth for 55 of the 81 cores. For each core tested, the grooved/tined surface was first removed by saw-cutting to obtain a flat, uniform surface; then 1/8-inch-thick slices of concrete were sampled at three to five depths between the trimmed surface and a nominal depth of 2 inches. The concrete slices were oven-dried and crushed into a fine powder passing a No. 20 sieve, then tested for water-soluble chloride content essentially in accordance with ASTM C1218-20, *Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*. The resulting chloride concentration profiles obtained with depth are presented in figures and tables for each bridge at the end of this appendix.

For each bridge deck, one additional slice of concrete, measuring approximately 1/8 to 1/4 inch in thickness, was sampled from the bottom fractured surface of the core (at nominal depths of approximately 4 to 5 inches), to obtain a "background" concentration of water-soluble chloride present due to the chloride content of the concrete constituents. An additional slice of concrete measuring approximately ½ inch in thickness was sampled from the concrete adjacent to the "background" slices and used to create thin sections for limited petrographic examination in accordance with ASTM C856. The petrographic examination was limited to identifying the type of supplementary cementitious materials (SCMs) present and visually estimating their relative proportions in the concrete, if present. The "background" chloride concentrations and SCM content estimates for each bridge deck are summarized in Table A3. SCMs (fly ash and slag cement) were detected in cores from five of the bridges. Only one bridge, FHWA No. 41151, had a background concentration of water-soluble chloride ions above typical "negligible" levels.

Table A3. Summary of Background Chloride Contents and Estimated SCM Usage per Bridge

Bridge No.	FHWA No.	Background Water-Soluble Chloride Concentration, % by wt. concrete	SCM Usage (Based on Petrography)
1	607890	< 0.003[1]	No SCMs
2	607885	< 0.003[1]	10-20% Fly Ash
3	49051	0.004[2]	< 10% Fly Ash, 40-50% Slag Cement
4	40851	< 0.003[1]	<10 % Fly Ash
5	41151	0.039[2]	No SCMs
6	51311	0.003	15-25% Fly Ash
7	14241	< 0.003[1]	15-25% Fly Ash
8	14141	< 0.003[1]	No SCMs
9	18091	< 0.003[1]	No SCMs
10	18141	< 0.003[1]	No SCMs

Notes: [1]*Reported values less than 0.003% are within the sensitivity of the test method.*

[2]*Background chloride concentration shown was measured within 2 inches of the surface but is less than the concentration measured at the bottom of core.*

The chloride profiles and SCM information obtained from the cores were used to estimate the surface chloride concentration (C_s) and the apparent 28-day diffusion coefficient (D_{28}) of the concrete used in each bridge deck. Estimates of C_s and D_{28} were obtained by fitting chloride profiles generated from deterministic model simulations of chloride movement in WJE CASLE (see Section 6.1) to the measured water-soluble chloride profiles for each core. The simulations used to fit the chloride profiles assumed that chloride exposures built up over a 5-year period from their initial background concentration to a maximum surface concentration of C_s. SCMs, where present, were assumed to affect the rate of decay for D_{28} as described in Section 6.1. For these estimates, the SCMs were assumed to be proportioned at the midpoint of the estimated dosage level for each bridge (e.g., cores from FHWA Bridge No. 49015 were modeled as having 5 percent fly ash and 45 percent slag cement).

The measured and fitted chloride profiles for each bridge are presented in the following pages. Table A4 summarizes the average C_s and D_{28} estimated for each bridge.

Table A4. Average Estimated C_s and D_{28} for each Bridge

Bridge No.	FHWA No.	Type of Route	Bridge C_s, % by wt. concrete			Bridge D_{28}, in²/yr		
			Avg.	Std. Dev.	COV	Avg.	Std. Dev.	COV
1	607890	County Highway (non-NHS)	0.326	0.106	32%	0.261	0.117	45%
2	607885	County Highway (non-NHS)	0.477	0.062	13%	0.274	0.094	34%
3	49051	US/State Highway (non-NHS)	0.956	0.098	10%	0.350	0.106	30%
4	40851	US/State Highway (NHS)	0.662	0.051	8%	0.384	0.070	18%
5	41151	Interstate	0.646	0.100	16%	0.470	0.263	56%
6	51311	Interstate	0.866	0.072	8%	0.463	0.191	41%
7	14241	US/State Highway (non-NHS)	0.847	0.109	13%	0.392	0.089	23%
8	14141	US/State Highway (NHS)	0.532	0.060	11%	0.243	0.050	21%
9	18091	Interstate	0.390	0.088	23%	0.214	0.103	48%
10	18141	Interstate	0.556	0.077	14%	0.309	0.060	19%

A.1 FHWA No. 607890 (WJE Bridge No. 1)

Type of Route: County Highway (non-NHS)

Location Description: Rural, agricultural lands. Agricultural processing unit nearby.

Age at Coring: 20 years

Background Chloride (% by wt. conc.): <0.003[6]

SCM Usage (based on petrography): No SCMs

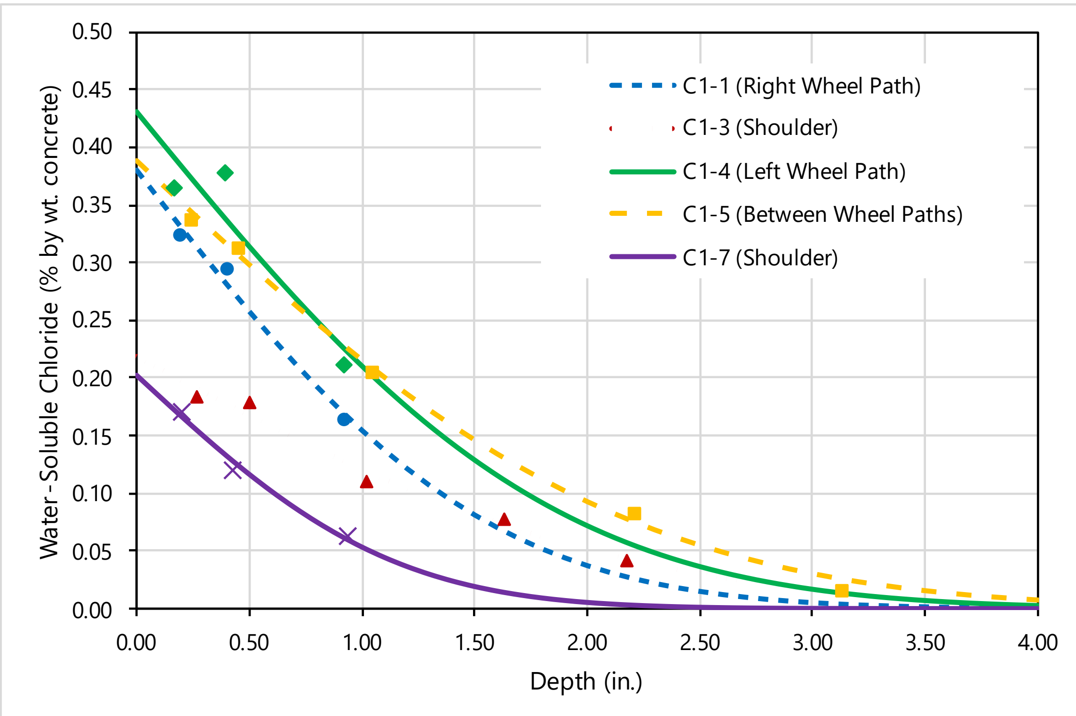

Figure A2. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 607890.

[6] Measured concentration is less than the sensitivity of the method (0.003 % by wt. conc.)

Table A5. Chloride data and chloride fitting parameters for cores from FHWA No. 607890

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C1-1	Right wheel path	0.19	0.327	0.381	0.189
		0.40	0.297		
		0.92	0.167		
C1-3	Shoulder	0.27	0.186	0.219	0.369
		0.50	0.182		
		1.02	0.113		
		1.63	0.080		
		2.18	0.044		
C1-4	Left wheel path	0.16	0.368	0.431	0.271
		0.39	0.380		
		0.92	0.214		
C1-5	Between wheel paths	0.24	0.340	0.388	0.374
		0.45	0.316		
		1.04	0.207		
		2.21	0.086		
		3.13	0.019		
C1-7	Shoulder	0.20	0.173	0.203	0.104
		0.42	0.123		
		0.94	0.066		

A.2 FHWA No. 607885 (WJE Bridge No. 2)

Type of Route: County Highway (non-NHS)

Location Description: Rural, agricultural lands

Age at Coring: 20 years

Background Chloride (% by wt. conc.): <0.003[7]

SCM Usage (based on petrography): 10-20% Fly Ash

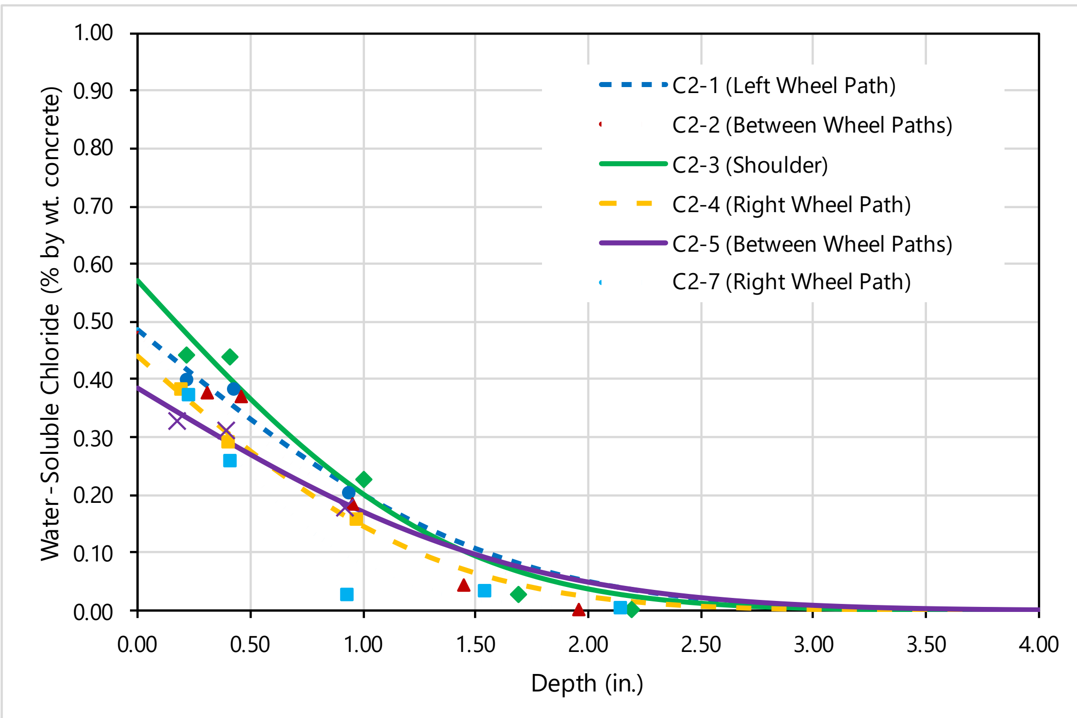

Figure A3. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 607885.

[7] Measured concentration is less than the sensitivity of the method (0.003 % by wt. conc.)

Table A6. Chloride data and chloride fitting parameters for cores from FHWA No. 607885

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C2-1	Left wheel path	0.21	0.402	0.489	0.343
		0.42	0.386		
		0.94	0.207		
C2-2	Between wheel paths	0.31	0.379	0.482	0.279
		0.46	0.374		
		0.95	0.187		
		1.45	0.046		
		1.96	0.004		
C2-3	Shoulder	0.22	0.445	0.574	0.267
		0.41	0.443		
		1.01	0.229		
		1.69	0.031		
		2.19	0.004		
C2-4	Right wheel path	0.19	0.387	0.443	0.244
		0.40	0.294		
		0.97	0.160		
C2-5	Between wheel paths	0.17	0.332	0.385	0.393
		0.39	0.314		
		0.92	0.179		
C2-7	Right Wheel Path	0.23	0.378	0.487	0.118
		0.41	0.261		
		0.93	0.030		
		1.54	0.038		
		2.14	0.009		

A.3 FHWA No. 49051 (WJE Bridge No. 3)

Type of Route: US/State Highway (non-NHS)

Location Description: Rural, agricultural lands. Town (Maxwell) nearby.

Age at Coring: 20 years

Background Chloride (% by wt. conc.): 0.004

SCM Usage (based on petrography): < 10% Fly Ash, 40-50% Slag Cement

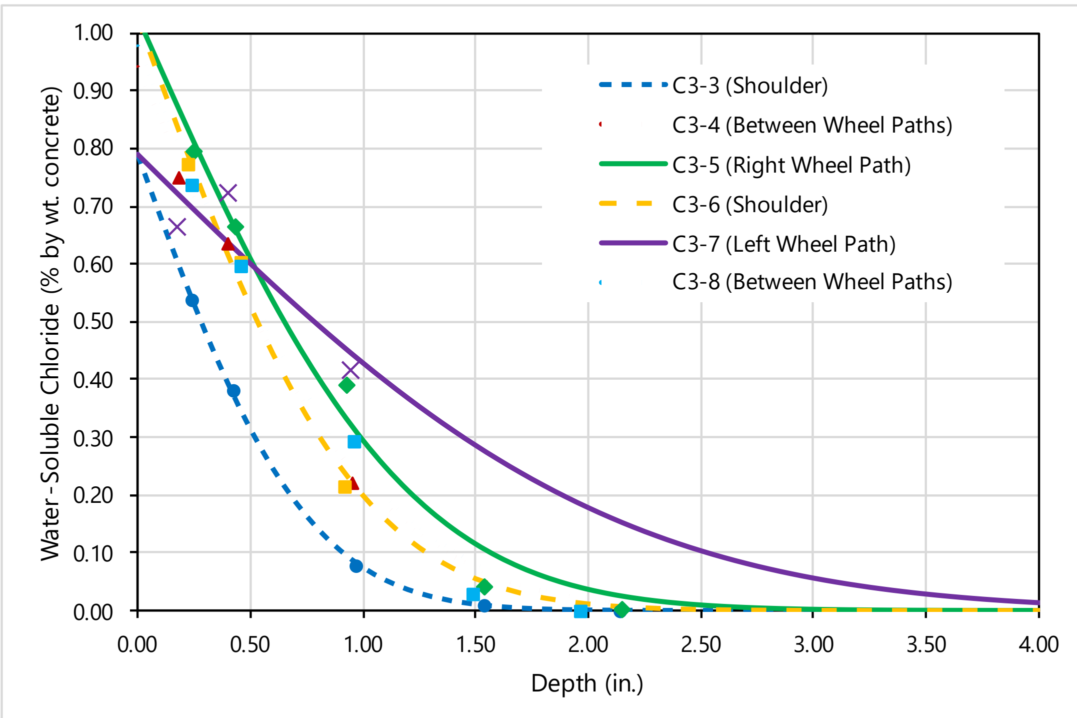

Figure A4. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 49051.

Table A7. Chloride data and chloride fitting parameters for cores from FHWA No. 49051

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C3-3	Shoulder	0.24	0.542	0.794	0.188
		0.42	0.386		
		0.97	0.081		
		1.54	0.014		
		2.14	0.005		
C3-4	Between wheel paths	0.18	0.754	0.943	0.380
		0.40	0.642		
		0.95	0.227		
C3-5	Right Wheel Path	0.25	0.802	1.032	0.470
		0.43	0.669		
		0.93	0.395		
		1.54	0.045		
		2.15	0.006		
C3-6	Shoulder	0.22	0.778	1.030	0.315
		0.46	0.607		
		0.92	0.220		
C3-7	Left wheel path	0.18	0.669	0.796*	1.448*
		0.40	0.729		
		0.95	0.422		
C3-8	Between wheel paths	0.24	0.742	0.980	0.396
		0.46	0.603		
		0.96	0.299		
		1.49	0.035		
		1.97	0.004		

** Chloride profile influenced by capillary absorption (transfer function), which was not considered in the bridge deck models; therefore, data for this core was excluded from statistics shown in Table A4.*

A.4 FHWA No. 40851 (WJE Bridge No. 4)

Type of Route: US/State Highway (NHS)

Location Description: Suburban

Age at Coring: 17 years

Background Chloride (% by wt. conc.): <0.003[8]

SCM Usage (based on petrography): <10% Fly Ash

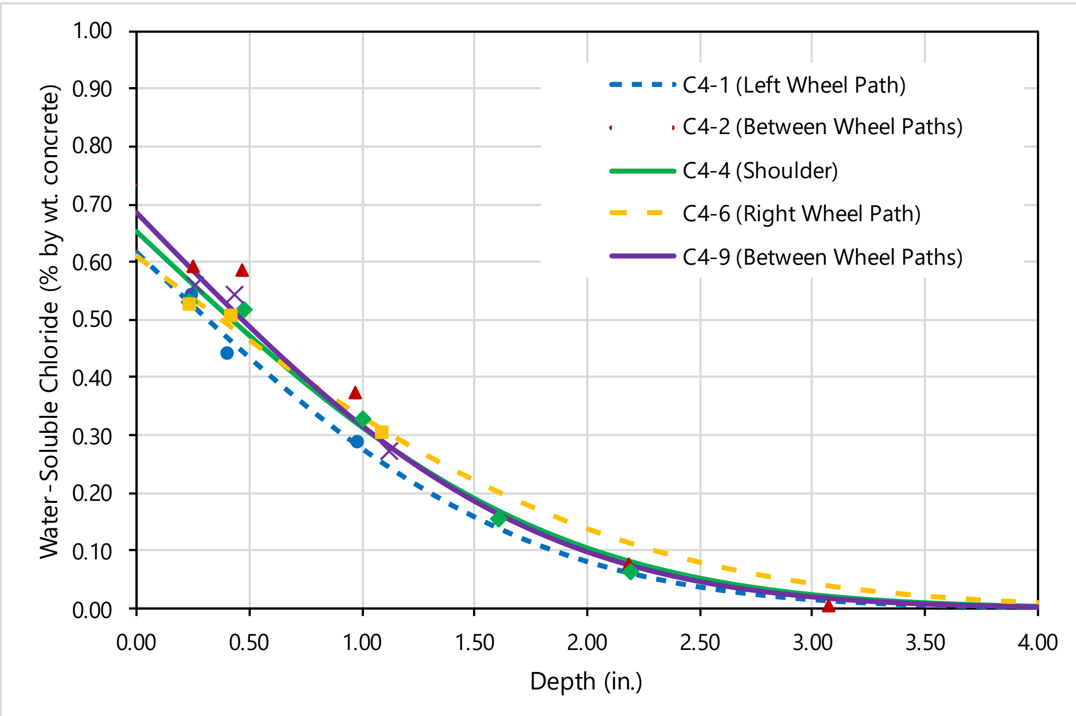

Figure A5. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 40851.

[8] Measured concentration is less than the sensitivity of the method (0.003 % by wt. conc.)

Table A8. Chloride data and chloride fitting parameters for cores from FHWA No. 40851

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C4-1	Left wheel path	0.24	0.548	0.618	0.322
		0.40	0.447		
		0.98	0.291		
C4-2	Between Wheel Path	0.25	0.595	0.735	0.378
		0.46	0.589		
		0.97	0.376		
		2.19	0.081		
		3.07	0.010		
C4-4	Shoulder	0.24	0.534	0.655	0.373
		0.48	0.520		
		1.00	0.332		
		1.61	0.158		
		2.19	0.067		
C4-6	Right wheel path	0.23	0.530	0.613	0.502
		0.41	0.510		
		1.09	0.309		
C4-9	Between wheel paths	0.26	0.563	0.689	0.342
		0.44	0.547		
		1.12	0.275		

A.5 FHWA No. 41151 (WJE Bridge No. 5)

Type of Route: Interstate

Location Description: Suburban

Age at Coring: 20 years

Background Chloride (% by wt. conc.): 0.039

SCM Usage (based on petrography): No SCMs

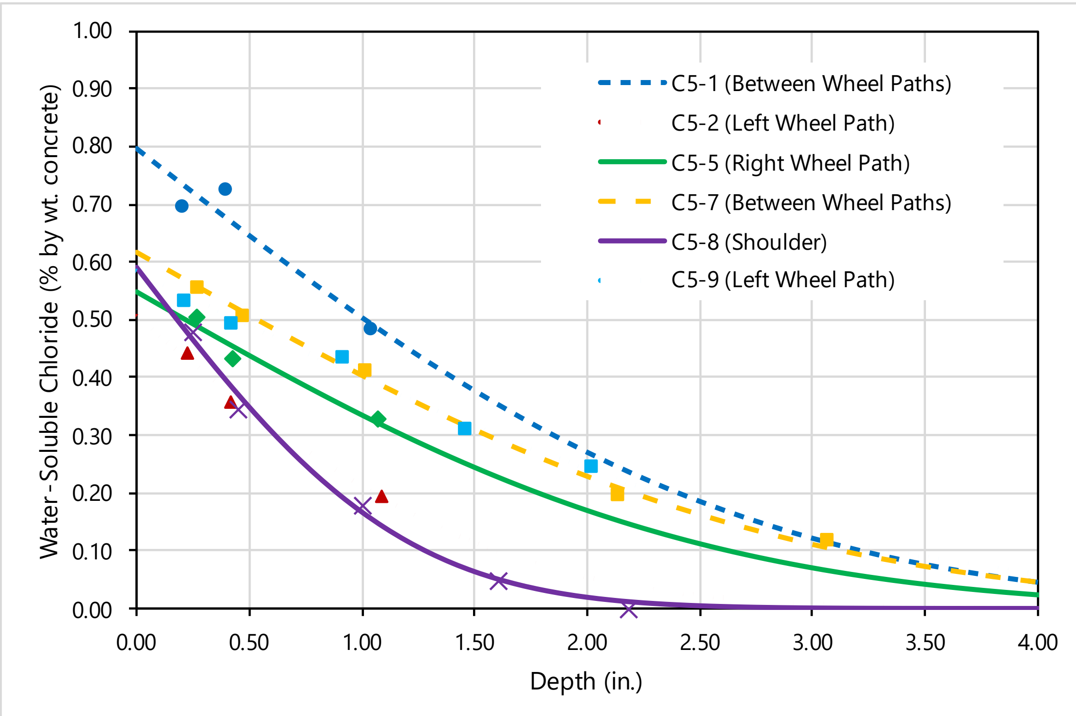

Figure A6. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 41151.

Table A9. Chloride data and chloride fitting parameters for cores from FHWA No. 41151

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C5-1	Between wheel paths	0.20	0.736	0.836	0.570
		0.39	0.766		
		1.04	0.527		
C5-2	Left wheel path	0.23	0.482	0.546	0.199
		0.42	0.399		
		1.09	0.235		
C5-5	Right wheel path	0.27	0.544	0.588	0.503
		0.43	0.472		
		1.07	0.368		
C5-7	Between Wheel Path	0.26	0.597	0.656	0.651
		0.47	0.549		
		1.01	0.452		
		2.13	0.237		
		3.06	0.159		
C5-8	Shoulder	0.25	0.519	0.629	0.112
		0.45	0.386		
		1.00	0.219		
		1.60	0.089		
		2.18	0.039		
C5-9	Left wheel path	0.21	0.574	0.622	0.786
		0.42	0.536		
		0.91	0.477		
		1.46	0.354		
		2.02	0.288		

A.6 FHWA No. 51311 (WJE Bridge No. 6)

Type of Route: Interstate

Location Description: Rural

Age at Coring: 16 years

Background Chloride (% by wt. conc.): 0.003

SCM Usage (based on petrography): 15-25% Fly Ash

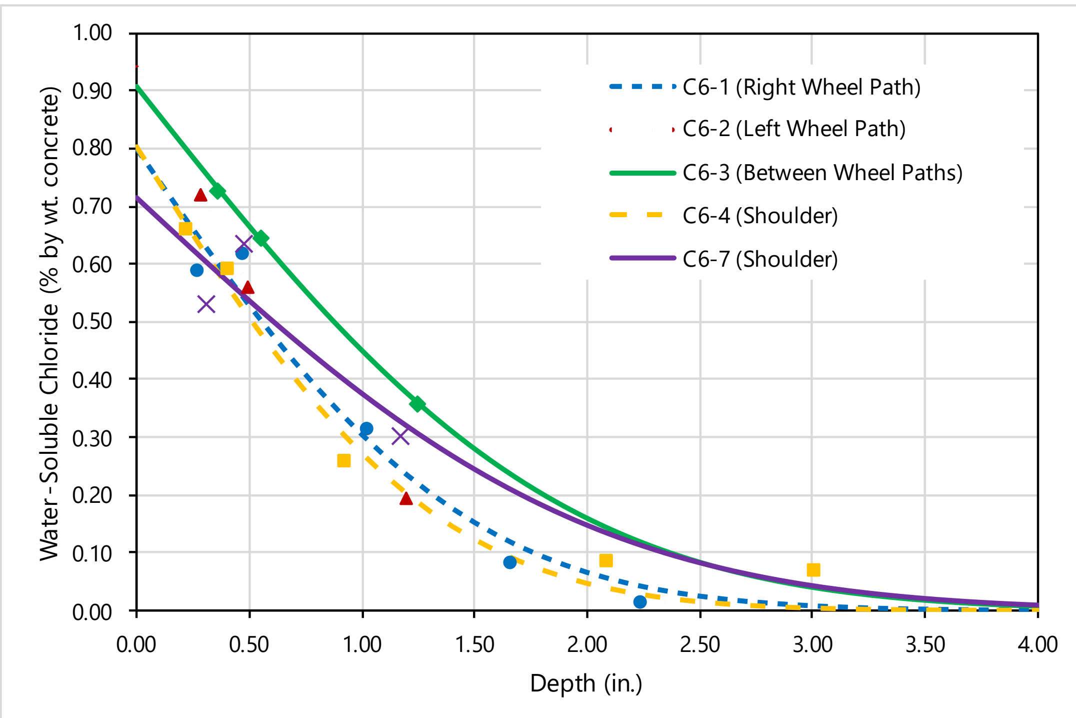

Figure A7. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 51311.

Table A10. Chloride data and chloride fitting parameters for cores from FHWA No. 51311

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C6-1	Right Wheel Path	0.26	0.593	0.803	0.443
		0.47	0.623		
		1.02	0.319		
		1.65	0.087		
		2.24	0.019		
C6-2	Left wheel path	0.28	0.724	0.944	0.303
		0.49	0.565		
		1.20	0.199		
C6-3	Between wheel paths	0.36	0.731	0.910	0.737
		0.55	0.650		
		1.25	0.361		
C6-4	Shoulder	0.21	0.666	0.806	0.371
		0.40	0.596		
		0.92	0.263		
		2.09	0.091		
		3.01	0.074		
C6-7	Shoulder	0.31	0.536	0.716*	0.847*
		0.48	0.638		
		1.17	0.306		

** Chloride profile influenced by capillary absorption (transfer function), which was not considered in the bridge deck models; therefore, data for this core was excluded from statistics shown in Table A4.*

A.7 FHWA No. 14241 (WJE Bridge No. 7)

Type of Route: US/State Highway (non-NHS)

Location Description: Rural, agricultural lands. Town (Kimbalton) nearby.

Age at Coring: 20 years

Background Chloride (% by wt. conc.): <0.003[9]

SCM Usage (based on petrography): 15-25% Fly Ash

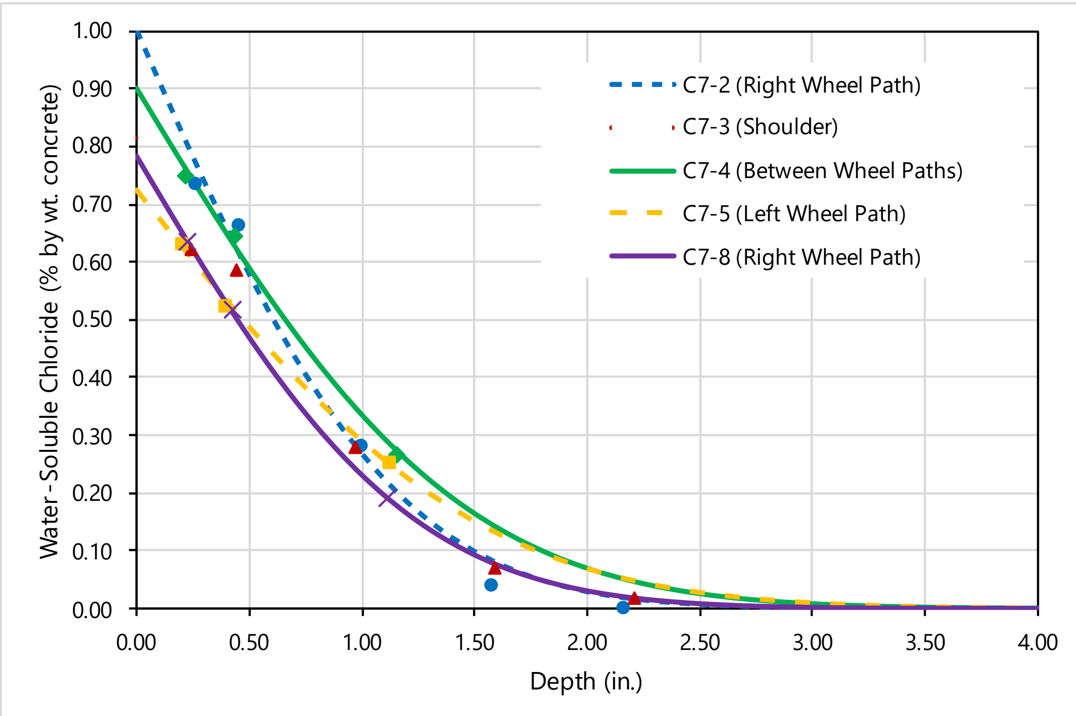

Figure A8. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 14241.

[9] Measured concentration is less than the sensitivity of the method (0.003 % by wt. conc.)

Table A11. Chloride data and chloride fitting parameters for cores from FHWA No. 14241

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C7-2	Right wheel path	0.26	0.739	1.004	0.293
		0.45	0.667		
		0.99	0.286		
		1.57	0.045		
		2.16	0.004		
C7-3	Shoulder	0.24	0.626	0.815	0.376
		0.45	0.590		
		0.97	0.282		
		1.59	0.075		
		2.21	0.020		
C7-4	Between wheel paths	0.22	0.752	0.903	0.454
		0.43	0.648		
		1.15	0.270		
C7-5	Left wheel path	0.20	0.636	0.726	0.511
		0.39	0.528		
		1.12	0.257		
C7-8	Right wheel path	0.23	0.637	0.786	0.329
		0.42	0.520		
		1.11	0.193		

A.8 FHWA No. 14141 (WJE Bridge No. 8)

Type of Route: US/State Highway (NHS)

Location Description: Rural, agricultural lands. Town (Audubon) nearby. Near high school.

Age at Coring: 17 years

Background Chloride (% by wt. conc.): <0.003[10]

SCM Usage (based on petrography): No SCMs

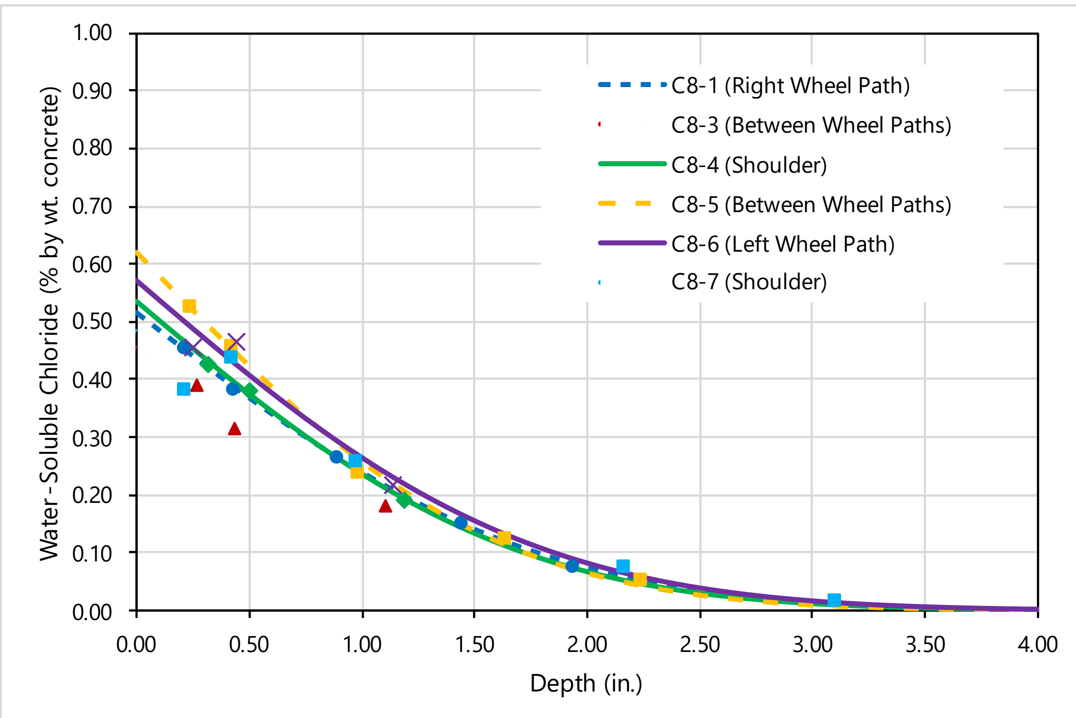

Figure A9. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 14141.

[10] Measured concentration is less than the sensitivity of the method (0.003 % by wt. conc.)

Table A12. Chloride data and chloride fitting parameters for cores from FHWA No. 14141

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C8-1	Right wheel path	0.21	0.459	0.518	0.243
		0.43	0.387		
		0.89	0.271		
		1.44	0.157		
		1.94	0.079		
C8-3	Between wheel paths	0.27	0.393	0.456	0.217
		0.44	0.320		
		1.10	0.185		
C8-4	Shoulder	0.32	0.429	0.537	0.220
		0.50	0.383		
		1.19	0.193		
C8-5	Between Wheel Path	0.23	0.532	0.623	0.196
		0.42	0.462		
		0.98	0.242		
		1.63	0.129		
		2.23	0.058		
C8-6	Left Wheel path	0.25	0.459	0.572	0.242
		0.44	0.468		
		1.14	0.221		
C8-7	Shoulder	0.21	0.387	0.484	0.338
		0.41	0.444		
		0.97	0.262		
		2.16	0.082		
		3.09	0.022		

A.9 FHWA No. 18091 (WJE Bridge No. 9)

Type of Route: Interstate

Location Description: Rural

Age at Coring: 15 years

Background Chloride (% by wt. conc.): <0.003[11]

SCM Usage (based on petrography): No SCMs

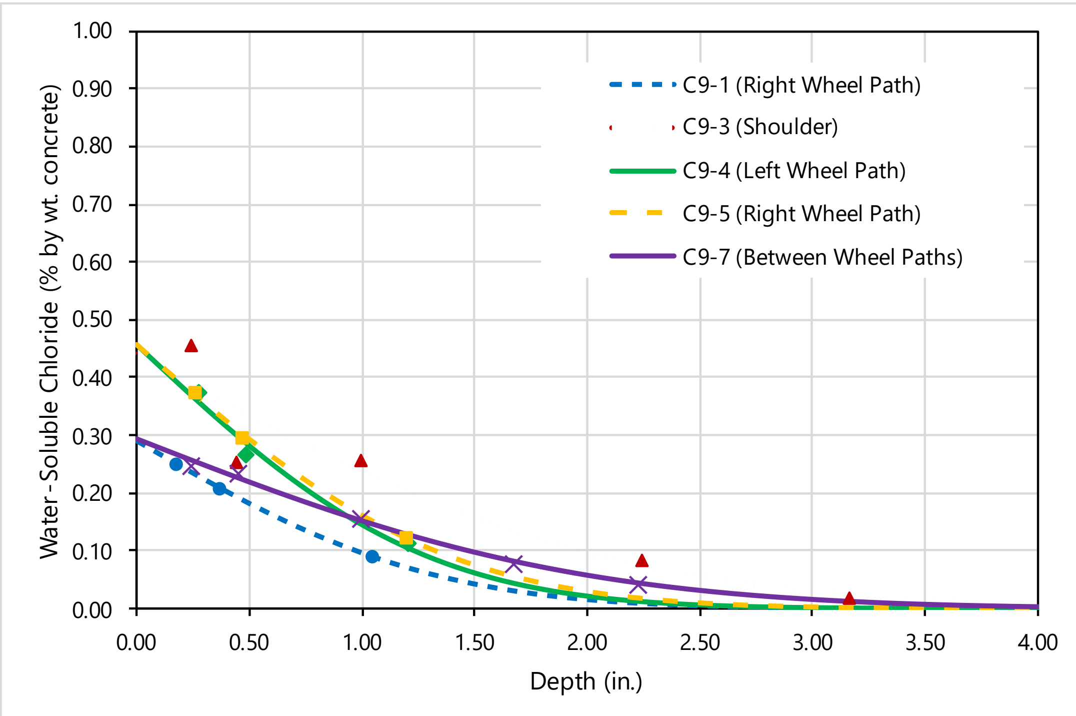

Figure A10. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 18091.

[11] Measured concentration is less than the sensitivity of the method (0.003 % by wt. conc.)

Table A13. Chloride data and chloride fitting parameters for cores from FHWA No. 18091

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C9-1	Right wheel path	0.17	0.252	0.291	0.138
		0.36	0.212		
		1.04	0.092		
C9-3	Shoulder	0.24	0.460	0.443	0.343
		0.44	0.256		
		1.00	0.259		
		2.24	0.087		
		3.16	0.020		
C9-4	Left wheel path	0.27	0.376	0.460	0.130
		0.48	0.269		
		1.21	0.116		
C9-5	Right wheel path	0.26	0.377	0.458	0.151
		0.46	0.299		
		1.19	0.127		
C9-7	Between Wheel Path	0.24	0.251	0.297	0.310
		0.45	0.238		
		1.00	0.160		
		1.67	0.080		
		2.23	0.044		

A.10 FHWA No. 18141 (WJE Bridge No. 10)

Type of Route: Interstate

Location Description: Rural

Age at Coring: 20 years

Background Chloride (% by wt. conc.): <0.003[12]

SCM Usage (based on petrography): No SCMs

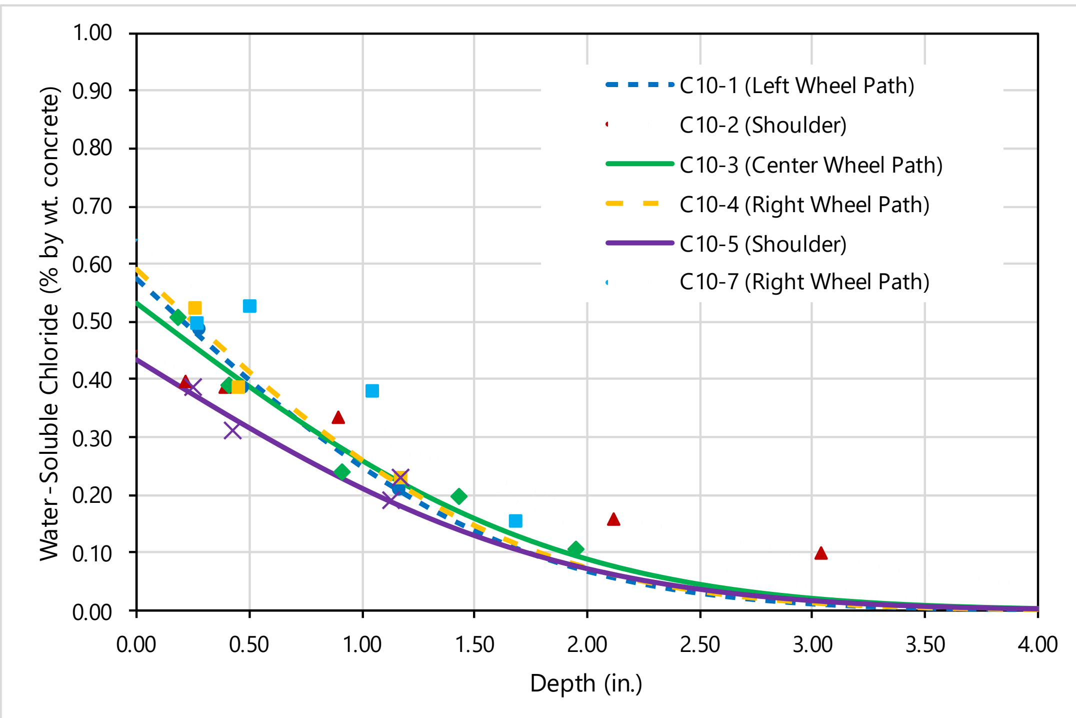

Figure A11. Measured (data points) and fitted chloride profiles (lines) for cores collected from FHWA No. 18141.

[12] Measured concentration is less than the sensitivity of the method (0.003 % by wt. conc.)

Table A14. Chloride data and chloride fitting parameters for cores from FHWA No. 18141

Core ID	Location	Depth (in.)	Water-Soluble Chloride Content (% by wt. conc.)	Fitted C_s (% by wt. conc.)	Fitted D_{28} (in.^{2}/yr)
C10-1	Left wheel path	0.27	0.491	0.575	0.250
		0.47	0.392		
		1.17	0.215		
C10-2	Shoulder	0.22	0.401	0.448*	0.871*
		0.39	0.389		
		0.90	0.339		
		2.12	0.163		
		3.03	0.103		
C10-3	Center wheel path	0.18	0.512	0.535	0.320
		0.41	0.393		
		0.92	0.244		
		1.43	0.200		
		1.95	0.109		
C10-4	Right wheel path	0.26	0.529	0.593	0.259
		0.45	0.391		
		1.17	0.233		
C10-5	Shoulder	0.25	0.392	0.436	0.319
		0.42	0.316		
		1.13	0.196		
C10-7	Right wheel path	0.27	0.500	0.640	0.398
		0.50	0.532		
		1.05	0.383		
		1.69	0.160		
		2.21	0.087		

** Core located near crack; excluded from statistics shown in Table A4.*