Service Life Design Reference Guide

November 2022

Source: Modjeski and Masters

U.S. Department of Transportation
Federal Highway Administration

Sponsored by

Federal Highway Administration
Office of Bridges and Structures
FHWA-HIF-22-052

FOREWORD

This document is intended to act as a "road map" of service life design concepts and methods for bridge owners and designers. The objective of service life design is to complete an assessment of the potential deterioration mechanisms affecting structural elements and to appropriately design those elements accordingly in order to achieve a target service life duration. The service life design process should be implemented from project outset through all project stages: from design, to construction, and through all operation phases.

The focus of the document is on North American design practice and references for application of service life design principles are provided for highway bridges composed of both concrete and steel. The document provides an introduction to service life design, the service life design process, and the different concepts of service life design. This is followed by a chapter with example problems. It can be used by bridge owners and designers to improve the performance of bridges built using the methods and references contained in this document.

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TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No. FHWA-HIF-22-052	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle Reference Guide for Service Life Design of Bridges	5. Report Date November 2022
	6. Performing Organization Code:
7. Author(s) Travis Hopper, Anne-Marie Langlois, Thomas Murphy	8. Performing Organization Report No.
9. Performing Organization Name and Address COWI North America 138 13th Street East, Suite 400 North Vancouver, BC V7L 0E5 Canada Modjeski and Masters 100 Sterling Parkway, Suite 302 Mechanicsburg, PA 17050	10. Work Unit No. 11. Contract or Grant No. 693JJ319D000021
12. Sponsoring Agency Name and Address Office of Bridges and Structures Federal Highway Administration 1200 New Jersey Ave SE Washington, DC 20590	13. Type of Report and Period Covered May 2020 – May 2021 14. Sponsoring Agency Code

15. Supplementary Notes
This document was produced under contract 693JJ319D000021 under management of Wood Environment & Infrastructure Solutions, Inc.

16. Abstract
This document is intended to act as a nonbinding "road map" of service life design concepts and methods for bridge owners and designers. The focus of the document is on North American design practice and references for application of service life design principles are provided for highway bridges composed of both concrete and steel. The document mainly concentrates on the voluntary guidelines within published works from the SHRP2 R19A project and the AASHTO Guide Specification for Service Life Design of Highway Bridges. Three chapters covering an introduction to service life design, the service life design process, and the different concepts of service life design make up the main body of the document, followed by a chapter with example problems. There are three example problems that follow the nonbinding AASHTO Guide Specification methodology, accompanied by a comparison to similar examples from previously published and nonbinding SHRP2 R19A documents.

17. Key Words service life design, bridge design, durability	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161. http://www.ntis.gov

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 88	22. Price N/A

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1,000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or “metric ton”)	Mg (or “t”)
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or “t”)	megagrams (or “metric ton”)	1.103	short tons (2,000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	2.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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LIST OF ABBREVIATIONS

AAR	alkali aggregate reaction
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ACR	alkali-carbonate reaction
AE	air entrained
ASR	alkali-silica reaction
FHWA	Federal Highway Administration
fib	International Federation for Structural Concrete
GGBS	ground granulated blast furnace slag
HPC	high performance concrete
ISO	International Organization for Standardization
LCCA	life cycle cost analysis
LRFD	Load and Resistance Factor Design
NCHRP	National Cooperative Highway Research Program
OPC	ordinary portland cement
SCM	supplementary cementitious material
SHRP2	Strategic Highway Research Program 2
W/CM	water-to-cement ratio

EXECUTIVE SUMMARY

This document is intended to act as a nonbinding "road map" of service life design concepts and methods for bridge owners and designers. The focus of this document is on North American design practice and references for application of service life design principles are provided for highway bridges composed of both concrete and steel. Therefore, this document concentrates on the published works from the SHRP2 R19A project, as well as the AASHTO Guide Specification for Service Life Design of Highway Bridges. Federal law and regulations do not require the use of these resources.

The objective of service life design is to complete an assessment of the potential deterioration mechanisms affecting structural elements and to appropriately design those elements accordingly to achieve a target service life duration. The service life design process should be implemented from project outset through all project stages: from design, to construction, and through all operation phases.

The core of a service life methodology is connecting design decisions to their effects on the achieved service life of a bridge. This involves not only knowing deterioration rates, and general deterioration modeling, but also being able to directly tie changes in deterioration behavior to engineering design quantities. Because deterioration occurs over long time-frames, obtaining the quality data needed to create these connections can be difficult. Accelerated testing and the use of real-world field data are options, but both have their drawbacks. There will likely always be less precision involved in service life design approaches vs. other aspects of bridge design.

The methods that are currently utilized in bridge service life design represent the state-of-the-art; however, they do contain limitations that should be understood by the larger bridge community. This should not deter the use of any available nonbinding tools to improve the performance of bridges built using the methods and references described in this document.

CHAPTER 1. INTRODUCTION

Service life design principles have been gaining broader acceptance as a tool to improve the performance of highway bridges and optimize the limited infrastructure funding available, as demonstrated by the SHRP2 R19A implementation report (Bartholomew 2019). This document is intended to act as a nonbinding "road map" of service life design concepts and methods for bridge owners and designers. The focus of this document is on North American design practice and references for application of service life design principles to highway bridges composed of both concrete and steel.

One of the early nonregulatory documents for the application of service life design principles is fib Bulletin 34, Model Code for Service Life Design (fib 2006). This document presents strategies for service life design of concrete structures. One methodology included in the fib Bulletin 34 is the concept of chloride-induced corrosion, modeled by chloride diffusion through concrete according to Fick's second law, as a limit state to the service life of concrete components. This ties concrete cover dimensions, concrete quality, reinforcement type, and exposure to the service life of these components.

In the United States, the nonbinding SHRP2 research project R19A addressed the basis of service life design methodologies. The implementation part of the SHRP2 R19A project was focused on the application of fib Bulletin 34 and produced data specific to the United States. Following SHRP2 R19A, the NCHRP 12-108 research project was initiated to create a specification for the implementation of service life design principles into typical design practice. This led to the development of the AASHTO Guide Specification for *Service Life Design of Highway Bridges* (2020) (herein "AASHTO Guide Specification"). The use of the AASHTO Guide Specification is not required by Federal law.

Chapters 2 and 3 discuss the process of service life design and the various aspects to consider, and provide information on which references may be appropriate for each aspect. Chapter 4 presents three worked examples for service life design that are completed following the information provided in the AASHTO Guide Specification. Comparison is made to the results of similar examples from SHRP2 R19A implementation. Federal law and regulations do not require the use of either resource.

CHAPTER 2. SERVICE LIFE DESIGN PROCESS

The objective of service life design is to complete an assessment of the potential deterioration mechanisms affecting structural elements, and to design those elements accordingly to achieve a target service life duration.

The service life design process should be implemented from project outset through all project stages. Figure 1 provides a step-by-step design process flowchart listing typical service life inputs for the project during the design, construction, and operation phases.

During the initial project planning and design stages, a project-specific definition for service life and the duration of the service life for replaceable and non-replaceable components should be established. Not all components of the structure need to have the same service life; components identified as replaceable are expected to have a shorter service life than the overall structure. Service life consideration during the design phase consists of first identifying and characterizing the exposure conditions and associated deterioration mechanisms. Design strategies to address the applicable deterioration mechanisms are then selected. The next step is to perform a verification of the service life through establishment of materials, design, and construction parameters. The design phase ends with the development of Construction Specifications and plans for inspection and maintenance of the structure.

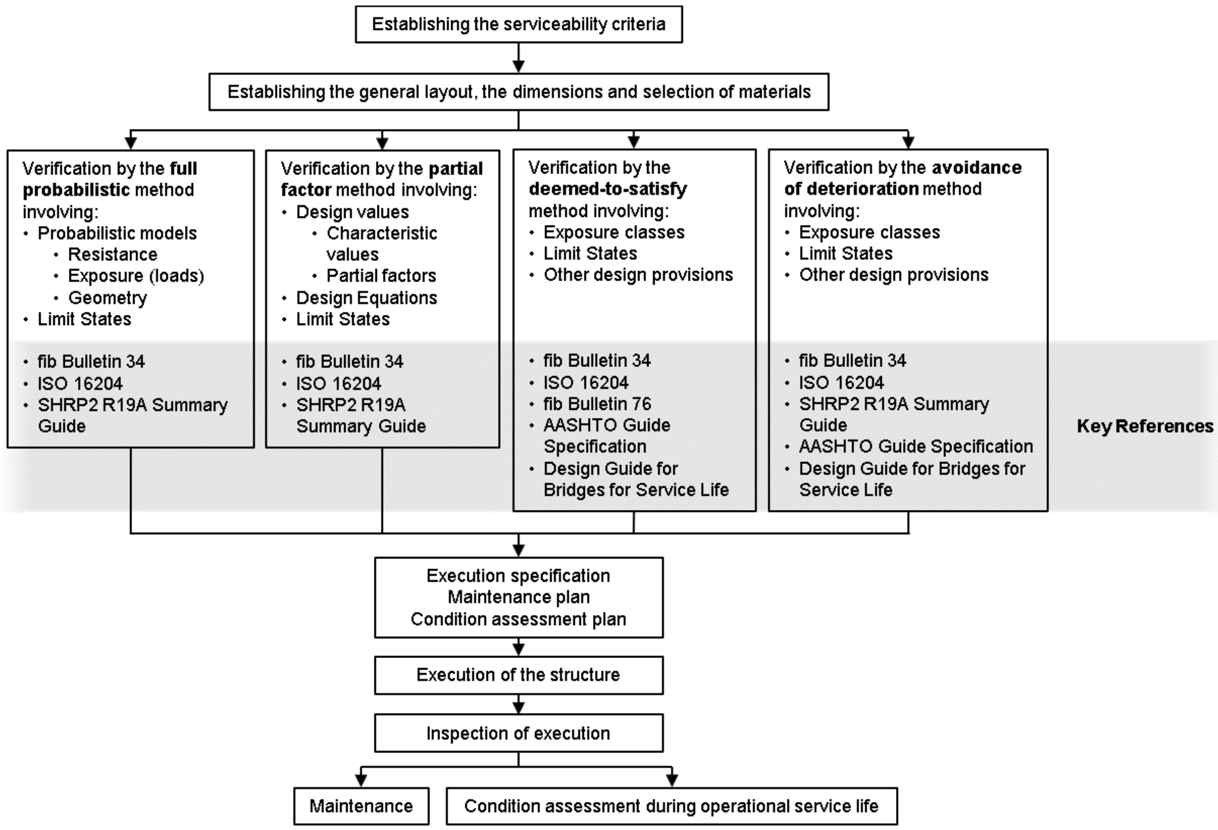
Construction, including quality testing, is then performed. Final inspection of the executed works marks the end of the construction phase. Inspection and maintenance plans can be updated during that phase. At this point, a service life design manual can be compiled with all design assumptions, presumed preservation, maintenance, and replacement schedules, and any pertinent construction notes. During the operation phase, inspection, monitoring, and maintenance works are carried out in accordance with the associated developed plans, the service life design manual, and the condition of the bridge.

The process for service life design is documented in various references in addition to the AASHTO Guide Specification. Additional references include:

- The Design Guide for Bridges for Service Life (Azizinamini et al. 2014)
- SHRP2 R19A Summary Guide (Pease et al. 2019)
- NCHRP Project 12-108 (Murphy et al. 2019)

Federal law and regulations do not require the use of any of these references.

The origins of service life design can be traced through fib Bulletin 34 (2006), ISO 16204 (2012), and fib Bulletin 76 (2015), which provides detailed information on service life design in addition to those listed above. These nonregulatory reference documents focus on concrete structures, but many of their basic concepts can and have been applied to other materials.



Source: adapted from ISO (2012) ©ISO. This material is adapted from ISO 16204:2012, with permission of the American National Standards Institute (ANSI) on behalf of The International Organization for Standardization. All rights reserved.

Figure 1. Flowchart. Service life design flowchart.

CHAPTER 3. REVIEW OF INFORMATION

This chapter presents an overview of the different concepts of service life design and provides references pointing the reader to the appropriate document for further information. The concepts are presented in the same order as the flowchart presented in Figure 1.

TARGET SERVICE LIFE

A clear, concise definition of service life can ensure the entire project team (owner, designer, contractor) have a common understanding of the durability objectives.

FHWA regulations do not define "service life", however the AASHTO Guide Specification provides the following nonbinding definitions:

- Target Service Life: *The assumed period of time the bridge is expected to remain in operation, without rehabilitation or significant repair, and with only routine maintenance (intended life). This would include replacement of renewable elements.*

- Renewable Element: *An element designed to be replaceable within the service life of the structure.*

The target service life is described in qualitative terms in the AASHTO Guide Specification using the following three categories:

- Normal—*For nonreplaceable components of typical highway bridges.*

- Enhanced—*For nonreplaceable components of bridges with either high initial capital cost, high ADT, significant social context, high consequences of serviceability failure, or other criteria warranting an increased service life as determined by the Owner.*

- Maximum—*For nonreplaceable components of bridges with either higher initial capital cost, higher ADT, critical social context, unacceptable consequences of serviceability failure, or other criteria warranting a maximum service life as determined by the Owner.*

The calibration performed in the AASHTO Guide Specification is based on the Normal service life being 75 years, Enhanced service life being 100 years and the Maximum service life being 150 years.

Further discussion about service life terms can be found in the NCHRP 12-108 report as well as the SHRP2 R19A Summary Guide. Use of these resources is not required by Federal law or regulations.

ENVIRONMENTAL EXPOSURE CONDITIONS

For service life design, the demands on a structure are determined by the surrounding environment. The environment determines the types and severity of environmental exposure conditions that a bridge structure will be subjected to throughout its service life. Ambient environmental conditions that affect the service life of bridges include temperature, humidity and wetness, contaminants and debris, and chemical characteristics such as the presence of corrosive ions such as chlorides or sulfates (NACE 2012). These conditions may be partitioned by variations in severity within the same bridge or even within the same bridge component. In addition, a bridge and its components may be exposed to multiple environmental conditions simultaneously.

It is always important to consult governing local specifications first. Any information related to local environmental exposure within these documents should take precedence over specifications that are national in scope. Local specifications often contain detailed information on how to describe and classify environmental exposure conditions at the bridge site.

Exposure Zones

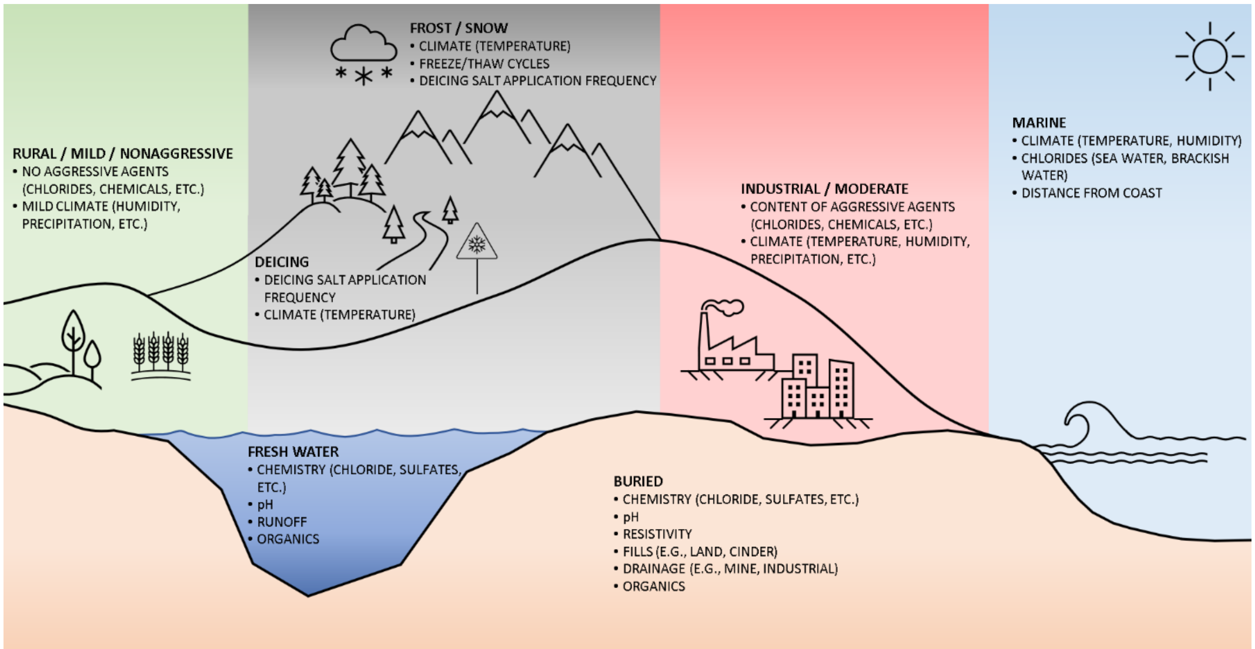
Exposure zones are used in design to define the severity of environmental exposure. Environmental exposure zones typically are separated into two categories: macro exposure zones that account for local site conditions and micro exposure zones that are specific to each component (Kogler 2015).

Macro Exposure Zones

Macro exposure zones account for a structure’s local site conditions. Factors such as proximity to a coastline and temperature are often used to define these zones. Common macro exposure zones include:

- Rural/Mild/Non-aggressive
- Industrial environment
- Marine/Coastal environment
- Deicing environment
- Frost and snow exposure
- Buried exposure

Figure 2 shows several common exposure zones and corresponding design considerations.



Source: FHWA

Figure 2. Illustration. Common macro exposure zones and their considerations.

Some design documents used by governments in the United States and around the world provide information on establishing macro exposure zones based on environmental site conditions pertinent to their jurisdiction. Examples include the Australian Standard AS5100.5 (2017), the Florida DOT Structures Manual (FDOT 2021), the Georgia DOT Bridge and Structures Design Manual (GDOT 2020), the Oregon Bridge Design Manual (ODOT 2020), and several Caltrans Memos to Designers (Caltrans 2010a, Caltrans 2010b).

Micro Exposure Zones

Micro exposure zones are used to describe the environmental loads for individual components or regions of a structure. Common micro exposure zones include:

- Buried
- Submerged
- Tidal/Water Level
- Direct Deicing
- Indirect Deicing
- Splash/Spray
- Atmospheric

- Interior (e.g., inside of box beams)
- Other Exterior

Table 1 lists common micro exposure zones and compares their inclusion in the AASHTO Guide Specification, the SHRP2 R19A Summary Guide, and several ACI documents. None are required by Federal law or regulations.

Table 1. Micro exposure zone comparison.

Micro Exposure Zone	AASHTO Guide Specification	SHRP2 R19A Summary Guide	ACI
Buried	X	X	-
Submerged	X	X	X[1, 3]
Tidal/Water Level	X	X	-
Splash	X	X	X[1]
Spray	X	X	X[3]
Indirect Deicing	X	X	X [2, 3]
Direct Deicing	X	X	X [2, 3]
Atmospheric	X	X	X[1]
Interior	X	-	-
Other Exterior	X	-	-

X = included
[1]ACI 357R (1984)
[2]ACI 362.1R (2012)
[3]ACI 318 (2019)

Similar to macro exposure zones, information on micro exposure zones is contained in regional design documents that supplement or supersede voluntary national specifications like those in Table 1. Examples include the Florida DOT Structures Manual (FDOT 2021), and several Caltrans design documents (Caltrans 2010a, Caltrans 2010b, Caltrans 2019).

Additional discussion on exposure zones is provided in Section 4 of the SHRP2 R19A Summary Guide and Chapter 3 of the NCHRP 12-108 report. Use of these resources is not required by Federal law or regulations.

DETERIORATION MECHANISMS

Common deterioration mechanisms to bridge structures are described below.

Concrete Structures

Chloride Induced Corrosion

AASHTO Guide Specification article C4.1.1 notes that chloride induced corrosion is typically caused by the ingress of chlorides into concrete via diffusion or through extensive cracks that are not well controlled. SHRP2 R19A Summary Guide Clause 4.3.1.6 explains chloride ions from seawater or de-icing salts can penetrate the concrete through the pore solution. A concentration of chloride ions in excess of the critical chloride threshold can initiate depassivation of the reinforcement, and eventually, corrosion.

Carbonation Induced Corrosion

AASHTO Guide Specification article C4.1.2 indicates that carbonation induced corrosion is caused by the ingress of atmospheric carbon dioxide, which reacts with the hydrated cement paste. This reaction acts to lower the pH of the pore water and breaks down the passive protection layer surrounding the reinforcing steel, thus permitting corrosion to initiate.

Freeze-thaw Attack

As described in SHRP2 R19A Summary Guide clause 4.3.1.1, freeze-thaw cycles can cause deterioration (cracking) when the pore structure of the cement paste is not designed with a sufficiently fine entrained air system, the concrete is critically saturated, and the water in the pores freezes to ice and expands.

Alkali Aggregate Reaction

AASHTO Guide Specification article C4.1.4 indicates that alkali aggregate reaction (AAR) refers to damage that results from a reaction between the alkali hydroxide in concrete and certain forms of silica, which is commonly found in aggregates. This reaction results in the formation of a gel that expands in the presence of water inducing tension stresses in concrete that can cause cracking. There are two types of AAR: alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). Alkali-silica reaction is generally more common.

External Sulfate Attack

External sulfate attack occurs when concrete is in contact with sulfate-bearing environment such as soil or groundwater. As explained by AASHTO Guide Specification article C4.1.5, a sulfate attack is caused by the ingress of sulfate ions that react with hydrated cement paste to form an expansive crystalline product known as ettringite.

Delayed Ettringite Formation

SHRP2 R19A Summary Guide Clause 4.3.1.4 indicates that internal sulfate attack that can occur in concrete cured at elevated temperatures such as in precast units or mass concrete placements.

Abrasion

AASHTO Guide Specification article C4.1.6 states that abrasion refers to the progressive section loss of concrete due to mechanical wear (e.g., studded tires or ice floes). This section loss can lead to inadequate cover which can reduce resistance to carbonation and chloride induced corrosion, ride quality issues for bridge decks, and, in some cases, reduced structural capacity.

Scaling

SHRP2 R19A Summary Guide Clause 4.3.1.2 indicates that the expansion of water because of freezing and thawing cycles combined with the use of de-icing chemicals can lead to scaling, which is a general loss of surface mortar.

Preservice Cracking

AASHTO Guide Specification article C4.1.7 indicates that preservice cracking of concrete (particularly concrete bridge decks) is caused by restraining the volume change that occurs within early-age concrete. This volume change is due to both thermal effects (that result from a drop in temperature following hydration) and shrinkage (inclusive of autogenous, drying, and plastic). As these volume changes are restrained by the girders, cross-frames, supports, etc., tension stresses develop that may overcome the tensile strength of the early-age concrete, thus resulting in cracking. Such cracks (if they are sufficiently large) provide additional pathways for carbon dioxide, chlorides, water, and sulfates thus increasing the severity of exposure and reducing the resistance of concrete to the deterioration mechanisms described above.

Chemical Attack

ACI 201.2R-16 (2016) indicates that concrete is rarely attacked by chemicals in their solid form. To produce a significant attack on concrete, aggressive chemicals would be in solution and above some minimum threshold concentration to drive the chemical reactions that diminish its engineering properties. Although concrete may perform satisfactorily in a variety of exposure conditions where aggressive chemicals are present, some kinds of chemical environments will significantly shorten the service life of even the best concrete unless specific measures are taken. Examples of aggressive chemical exposures include acids, industrial chemicals, and wastewaters.

Steel Structures

Corrosion

The primary deterioration mechanism for steel is corrosion. The steel corrosion reaction is an electro-chemical process requiring the presence of moisture and oxygen to convert the iron in steel into one of its oxides. The two methods of acceleration for this process are:

- Stimulation of either the anodic or cathodic portion of the corrosion reaction by aggressive ions such as chlorides (e.g., from road de-icing salts) or sulfur dioxide (e.g., from atmospheric pollution); and

- The presence of well-defined local galvanic cells, which can result when rust, dirt or crevices cause differential access to air or from the placement of dissimilar metals in contact.

As a result, three types of corrosion (pitting corrosion, crevice corrosion, galvanic corrosion) are most relevant to the use of bare, coated, and weathering steel.

Information on steel corrosion is provided in the AASHTO Guide Specification Article 5.1.1. Use of the specification is not a Federal requirement.

Fatigue

Fatigue in metals is the process of initiation and growth of cracks under action of repetitive tensile loading cycles. Should cracking of the steel initiate and its growth be allowed to go unattended long enough, failure of the member or connected members can result when the member cross-section is sufficiently reduced such that the member can no longer carry the stresses imposed. When this critical stage is reached, the crack growth becomes unstable and sudden failure occurs.

Information on fatigue is provided in Section 6 of the AASHTO LRFD Bridge Design Specifications (2017b) (incorporated by reference at 23 CFR 625.4(d)(1)(v)) and the nonbinding AASHTO Guide Specification Article 5.1.2.

Exposure Classes

Exposure Classes are a means to tie exposure zones to deterioration mechanisms. Typically, multiple exposure classes are available for a given deterioration mechanism. Once a deterioration mechanism is identified, an exposure class is assigned base on the severity of the environment (i.e., exposure zone).

Concrete

Common concrete exposure classes used in North American codes are listed in Table 2. Use of these codes are not a Federal requirement.

Table 2. Concrete exposure classes.

Deterioration Mechanism	AASHTO Guide Specification[1]	ACI
Nonaggressive (NA) / No Risk	C-NA1: interior exposure C-NA2: other exterior exposure	—
Carbonation-induced Corrosion	—	—
Chloride-induced Corrosion (C) (Nonmarine)	C-D1: atmospheric in deicing salts environment C-D2: indirect deicing salts C-D3: direct deicing salts – low C-D4: direct deicing salts – high	C0: dry/protected from moisture C1: exposed to moisture but not external chloride source C2: exposed to moisture and external chloride source (deicing salts, brackish water)[2]
Chloride-induced Corrosion (C) (Marine)	C-M1: marine – atmospheric C-M2: marine – submerged C-M3: marine – tidal or splash/spray zone	C2: exposed to moisture and external chloride source (saltwater)[2]
Freeze-thaw Attack (FT or F)	FT0: not exposed FT1: limited exposure without chlorides FT2: frequent exposure without chlorides FT3: frequent exposure with chlorides	F0: not applicable F1: moderate F2: severe F3: very severe[2, 3]
Sulphate Attack (S)	S0: low S1: moderate S2: severe S3: very severe	S0: not applicable S1: moderate S2: severe S3: very severe[2, 4]
Chemical Attack	—	—
DEF	—	—
In Contact with Water (W)	W0: dry in service or low permeability not required W1: in contact with water and low permeability needed	W0: dry in service W1: in contact with water where low permeability not needed W2: in contact with water where low permeability needed[2]

[1]AASHTO Guide Specification Table 2.2.2.1-1
[2]ACI 318-19 Table 19.3.1.1
[3]ACI 201.2R-16 Table 4.2.3.1a
[4]ACI 201.2R-16 Table 6.1.4.1a

Steel

Exposure Classes for common steel deterioration mechanisms, corrosion and fatigue, are listed in Table 3.

Table 3. Steel exposure classes.

Deterioration Mechanism	AASHTO Guide Specification[1]	AASHTO LRFD[2]
Corrosion	C1: very low C2: low C3: medium C4: high C5I: very high industrial C5M: very high marine	—
Fatigue	—	Detail Categories A through E'

[1]AASHTO Guide Specification Table 2.2.2.2-1
[2]AASHTO LRFD Bridge Design Specifications (2017b) (23 CFR 625.4(d)(1)(v)) Table 6.6.1.2.3-1

DESIGN STRATEGIES

There are two main types of design strategies: the design to resist approach and the avoidance approach. Both are described below. Further information is available in the NCHRP 12-108 report (Chapter 2), the AASHTO Guide Specification (Section 1), and the SHRP2 R19A Summary Guide (Section 2). Use of these resources is not required by Federal law or regulations.

Design to Resist Approach

Full Probabilistic

Verification of the design to resist approach using a full probabilistic model is considered the most sophisticated strategy, which is similar in nature to the statistical basis behind the Load and Resistance Factor Design (LRFD) method employed by many structural design codes. A full probabilistic approach can be readily implemented for chloride-induced corrosion of reinforced concrete, such as that presented in fib Bulletin 34 (2006). Models for other deterioration mechanisms are not sufficiently mature at this time for broad implementation on bridge design projects.

In this approach, the environmental exposure conditions (analogous to loads) and the material resistance are represented as distributions to account for the inherent variation of these variables. A probabilistic modeling process is used (e.g., Monte Carlo analysis) to establish performance-based resistance parameters needed to provide an established acceptable level of safety, typically measured by the probability of failure or reliability index. It is noted that probabilistic-based models, with wide international acceptance, do not exist for all deterioration mechanisms affecting concrete, steel, and other construction materials. Therefore, knowledge of the available tools is important for the development of design parameters for service life.

Several documents and tools published as part of the SHRP2 R19A project are available to help the engineer implement a full probabilistic approach on bridge projects. The SHRP2 R19A project's objective was to provide training materials and tools for engineers and hence constitutes a good source of practical information.

The NCHRP 12-108 report (Chapter 3) and the AASHTO Guide Specification (Appendix A) contain a design framework to implement a full probabilistic approach in bridge design practice. Use of the framework is not required by Federal law or regulations.

Partial Safety Factor

As described in fib Bulletin 34, the partial safety factor strategy is a deterministic approach wherein the variable nature of the environmental exposure conditions and material resistances are accounted for using partial safety factors (e.g., load and resistance factors). The factors are calibrated against the full probabilistic approach to provide, as a minimum, the same reliability index. As is done with LRFD for structural design, partial safety factors are applied to design input parameters to verify that the applicable limit state is satisfied. This strategy has not been sufficiently developed at this time.

Examples of the partial safety factor strategy are available in fib Bulletin 34 and International Standards Organization (ISO) 16204 for carbonation-induced reinforcement corrosion. The SHRP2 R19A Summary Guide (Appendix D) presents a theoretical partial safety factor design methodology for chloride-induced reinforcement corrosion under certain exposure conditions. Use of these examples and methodologies is not required under Federal law or regulations.

The AASHTO Guide Specification utilizes a design approach for chloride-induced corrosion based on achieving a target reliability index for certain combinations of cover, concrete type, and reinforcing steel type. However, this is not a true partial safety factor strategy.

Deemed-to-Satisfy

The deemed-to-satisfy strategy utilizes rules and provisions that are based on the traditional performance of design practices. These provisions are used together, in some cases, with the exposure conditions, to verify the design to resist approach for a given limit state. This is by far the most common approach to service life design.

Typically, deemed-to-satisfy strategies in codes are not based on physical and/or chemical models, but on practical experience (fib 2006). For certain deterioration mechanisms, deemed-to-satisfy rules have been shown to provide adequate durability performance. However, in cases where the project-specific exposure conditions do not fit certain criteria (i.e., are out of scope) or their severity is underestimated, these deemed-to-satisfy rules can ultimately fail to provide a sufficiently durable structure.

In other limited instances, deemed-to-satisfy provisions are calibrated using a full probabilistic approach. fib Bulletin 76 (fib 2015) describes this process in detail, in which the reliability of deemed-to-satisfy concrete cover values from various national codes are checked using a full probabilistic approach. Using a similar methodology, the deemed-to-satisfy cover values for chloride induced corrosion contained in the voluntary AASHTO Guide Specification were calibrated using a full probabilistic model.

Examples of deterioration mechanisms that are reliably addressed by deemed-to-satisfy rules are:

- Sulfate attack of concrete. For sulfate attack, design codes (e.g., ACI 318) commonly specify that the exposure to sulfates in the soil and/or groundwater to be quantified and corresponding mitigation measures, which have been shown to be effective, are prescribed.

- Freeze-thaw resistance of concrete. Design codes commonly specify that air entrainment to protect the concrete against freeze-thaw cycles.

Information on the deemed-to-satisfy strategy is presented in the SHRP2 R19A Summary Guide (Section 2) and the NCHRP 12-108 report (Chapter 2). Deemed-to-satisfy provisions specific to a deterioration mechanism and/or element are provided in Sections 4 (Concrete), 5 (Steel), 6 (Foundations and Retaining Walls), and 7 (Renewable Elements) of the AASHTO Guide Specification, and Section 4 (Concrete and Steel) of the SHRP2 R19A Summary Guide. Use of these provisions and this information is not required under Federal law or regulations.

Avoidance Approach

In the avoidance approach, the deterioration is avoided completely through selection of non-reactive or inert materials, or by removing the element from the exposure condition. For example, using non-reactive aggregates to avoid ASR, using stainless-steel reinforcement to avoid corrosion, or the use of protection systems to separate the structural element from the aggressive media. The avoidance approach can be cost prohibitive and non-practical and therefore, its use should be justified by designers before being implemented on projects.

Information on the avoidance approach is presented in the SHRP2 R19A Summary Guide (Section 2). Avoidance design approaches for certain deterioration mechanisms are included in Section 4 of the SHRP2 R19A Summary Guide, and are built into many of the provisions in Chapters 4 through 7 of the AASHTO Guide Specification. Neither use of SHRP2 R19A nor the AASHTO Guide Specification are required under Federal law or regulations.

Mitigation

Design approaches to mitigate the effects of deterioration mechanisms may include the application of one or more of the four strategies that are listed in Figure 1 and that were discussed in previous sections.

Concrete

Based on the location of the structure and the exposure conditions, one or more of the deterioration mechanisms may be applicable to a particular concrete structure. Once the exposure conditions are identified, the design strategies and sources summarized in Table 4 can be used to address the identified deterioration mechanisms.

Table 4. Available design strategies and references for the mitigation of concrete deterioration mechanisms (Use not required under Federal law or regulations)

Deterioration Mechanism	Full Probabilistic	Partial Safety Factor	Deemed-to-Satisfy	Avoidance
Chloride Induced Corrosion	• SHRP2 R19A Summary Guide Table 4-7 • AASHTO Guide Specification Appendix A	• SHRP2 R19A Summary Guide Appendix D	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Articles 4.2.4 and 4.3	• AASHTO Guide Specification Articles 4.2.4 and 4.3
Carbonation Induced Corrosion	—	—	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Article 4.2.4	—
Freeze-thaw Attack	—	—	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Articles 4.2.1 and 4.2.2 • ACI 201.2R Articles 4.2 and 4.3	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Article 4.3 • ACI 201.2R Articles 4.2 and 4.3, Tables 4.2.3.1a-c, Table 4.2.3.2.4
Alkali Aggregate Reaction	—	—	• SHRP2 R19A Summary Guide Table 4-6 • ACI 201.2R Article 5.4	• SHRP2 R19A Summary Guide Table 4-6 • ACI 201.2R Article 5.4
Sulfate Attack	—	—	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Articles 4.2.1 and 4.2.3 • ACI 201.2R Table 6.1.4.1b	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Article 4.2.1 • ACI 201.2R Article 6.1, Table 6.1.4.1b
Delayed Ettringite Formation	—	—	• AASHTO Guide Specification Article 4.1.5 • ACI 201.2R Article 6.2 and Table 6.2.2.2	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Article 4.1.5 • ACI 201.2R Article 6.2 and Table 6.2.2.2
Abrasion	—	—	• ACI 201.2R Chapter 10 • AASHTO Guide Specification Articles 4.1.6 and 4.2.4.2.2	• ACI 201.2R Chapter 10
Scaling	—	—	• SHRP2 R19A Summary Guide Table 4-6 • AASHTO Guide Specification Article 4.2.2 • ACI 201.2R Article 4.2	—
Preservice Cracking	—	—	• AASHTO Guide Specification Articles 4.1.7 and 4.2.4.2.1c	• AASHTO Guide Specification Article 4.3
Chemical Attack	—	—	• ACI 201.2R Chapter 7, Table 7.1b	• ACI 201.2R Chapter 7, Table 7.1b

Steel

Designers should refer to the sources listed in Table 5 for information on the available design strategies to address steel deterioration mechanisms.

Table 5. Available design strategies and references for the mitigation of steel deterioration mechanisms (Not Federal requirements, except as otherwise noted)

Deterioration Mechanism	Full Probabilistic	Partial Safety Factor	Deemed-to-Satisfy	Avoidance
Corrosion	—	—	• SHRP2 R19A Summary Guide Table 4-8 • AASHTO Guide Specification Article 5.2, Appendix 5A • FHWA SBDH Volume 19[2] • Design Guide for Bridges for Service Life[3] Sections 6.4 and 6.5	• SHRP2 R19A Summary Guide Table 4-8 • AASHTO Guide Specification Articles 5.2 and 5.3 • FHWA SBDH Volume 19 • Design Guide for Bridges for Service Life Section 6.4
Fatigue	—	• AASHTO LRFD BDS[1] Article6.6.1	• AASHTO Guide Specification Article 5.3.1.2	• AASHTO Guide Specification Article 5.3.1.2

[1]AASHTO LRFD Bridge Design Specifications (2017b) (23 CFR 625.4(d)(1)(v))
[2]FHWA Steel Bridge Design Handbook (Kogler 2015) (nonregulatory handbook)
[3]Azizinamini et al. (2014)

LIMITATIONS AND FUTURE WORK

Limitations

The core of a service life methodology is connecting design decisions to their effects on the achieved service life of a bridge. This involves not only knowing deterioration rates, and general deterioration modeling, but being able to directly tie changes in deterioration behavior to engineering design quantities, for instance concrete permeability and cover depths. Because deterioration occurs over long time-frames, obtaining the quality data needed to create these connections is extremely difficult. Accelerated testing is one method, but it may not comprehensively capture all deterioration mechanisms and can give inaccurate deterioration rates compared to in-service materials. Using real-world field data is another option, but inevitably the information obtained relates to the materials and construction used in the bridge, which is decades old and may no longer relate to current practices in design or construction. Given these constraints, there will likely always be less precision involved in service life design approaches, when compared to other aspects of bridge design.

The methods that are currently utilized in bridge service life design represent the state-of-the-art; however, they do contain limitations that should be understood by the larger bridge community. The existence of these limitations should not deter use of the tools now available to improve the performance of bridges built using the methods and references described in this document.

The following are limitations related to the service life design of concrete structures:

- Cracking is not explicitly considered in the chloride-induced corrosion model. The model of chloride transport through concrete that is used in the fully probabilistic and the deemed-to-satisfy approaches for corrosion prevention assumes that any cracking present will not affect the diffusion process. In order for this to be accurate, cracking should be controlled in concrete through crack control measures as per the structural design code, appropriate mix design, placement and curing methods. See further discussion in the section on future work.

- Depending on the level of rigor desired, implementation of service life design can be time and/or cost intensive. The full probabilistic strategy discussed in the SHRP2 R19A Summary Guide and uses results of material testing during the design and construction stages to establish several distributions used in the service life model. The strategy also is outlined in Appendix A of the AASHTO Guide Specification. Similarly, the calibrated deemed-to-satisfy strategy in the AASHTO Guide Specification may be enhanced if material testing by owners is performed upfront before design to verify standard mixes meet the minimum assumptions used to establish the design provisions. Use of the Guide Specification and the Summary Guide is not required by Federal law or regulations.

- The influence of multiple concurrent deterioration mechanisms typically is not considered in models used for design. One example is cracking of concrete due to freeze-thaw accelerating chloride-induced corrosion.

Limitations for the service life design of steel structures include:

- Many of the design provisions are deemed-to-satisfy and based on corrosion estimates rather than a specific deterioration model.

- Occasional maintenance activities to renew protective coatings used on steel portions of a structure. If this is not done at the appropriate points in time, the service life of the bridge may not be achieved.

- Corrosion is often grouped into one category and designed for accordingly. The different types of corrosion that structural steel is often susceptible to (e.g., crevice, pitting, galvanic) and the nuances between them are often not separated in design. Corrosion in steel bridges is often highly localized, and tied to the performance of drainage and joint system. Depending on the type, location, and severity of corrosion, the service life can be greatly affected.

For members that are partially or completely buried in soil (e.g., foundations, retaining walls), the following limitations are relevant, in addition to those outlined above for concrete and steel components:

- Differences in design approaches throughout the industry create inconsistencies. For example, the specification for the design of a steel pile may classify soil aggressivity based on certain values for pH or chloride concentration, whereas the design of steel soil reinforcement for a MSE wall in the same soil according to a different specification may use different pH or chloride concentration limits to classify aggressivity. This can result in different protection strategies being chosen, even if all other factors remain identical.

Potential Future Work

Documented approaches to service life design in North America are still relatively new compared to other aspects of bridge design. The understanding of service life is continually in development and design methodology will need to be updated appropriately in the future. Several topics for potential future work have been laid out in detail in other documents, such as the NCHRP 12-108 report and the SHRP2 R19A Summary Guide.

Potential future concrete structures research topics include:

- Reinforcement critical chloride threshold. Calibration of the chloride threshold for different reinforcement types could be found by 1) improving and standardizing test methods, 2) studying the influence of environmental exposure conditions on critical chloride threshold, and 3) increasing the dataset size for various reinforcement types through additional testing.

- Further calibration of the chloride-induced corrosion model. This includes both research on the actual concrete parameters such as the ageing coefficient that describes the evolution of the chloride migration coefficient over time, and the quantification of the exposure conditions such as the chloride surface concentration.

- The influence of concrete cracking on the service life of concrete structures. Properties of cracking (e.g., width, depth, frequency. etc.), susceptibility of mixes to cracking, and timing of cracking are all factors that merit consideration.

For steel structures, the lack of a well-developed deterioration model for corrosion is arguably the main hindrance slowing the advancement of service life design. Any model should properly account for variations in environmental exposure conditions, as well as the differences in corrosion resistance among steel types, grades, and other protection strategies (e.g., coatings).

IMPLEMENTING SERVICE DESIGN LIFE ON PROJECTS

The AASHTO Guide Specification was developed specifically to implement service life design principles into current bridge design by providing "practical guidance to designers and owners on design decisions that affect the durability of highway bridges" (AASHTO 2020, p. 1-2). A variety of methods are applied, from deemed to satisfy through more rigorous methods. The SHRP2 R19A Summary Guide for Service Life Design also provides information for owners on how to implement service life design, with an emphasis on the full probabilistic approach.

Designing for service life is an additional effort and there are multiple scenarios where owners can introduce design for service life:

- *Signature Bridges*: Signature Bridge projects are situations where an advanced and detailed service life design specific to the structure and project can be undertaken. The additional effort and cost for service life are rather small compared to the magnitude and importance of the project. For this type of project, it may make sense to perform a full probabilistic assessment for service life of concrete structures. A design specific to the structure considering the exact exposure conditions present and the materials to be used can allow cost savings compared to using standard specifications and details. A similar level of effort for service life design may not make sense for a small bridge project where budget can be limited.

- *Package Projects*: Service life design can be implemented for a project where multiple structures are packaged together. For example, it could make sense to perform service life design for a number of structures located in a similar environment and having similar geometry and materials (e.g., interchanges, twin and sister bridges). In this case it may make sense to perform a full probabilistic assessment for service life design of concrete structures where efficiency is gained by performing the design once for a number of structures. Just as for Signature Bridges above, cost savings can be made given the magnitude of materials used on the project.

- *Standard Specifications and Details*: Service life design can be used to update the owner's standard specifications. For example, a full probabilistic assessment could be performed assuming general exposure conditions application to a given region. This could be similar to the AASHTO Guide Specification where general exposure zones are used to develop design parameters for materials. This approach allows implementation of service life design in standard and smaller projects without a significant design stage effort.

- *Alternative Delivery Projects:* The SHRP2 R19A Summary Guide for Service Life Design includes a Request for Proposal Examples for Alternative Delivery Projects implementing service life design.

CHAPTER 4. WORKED EXAMPLES

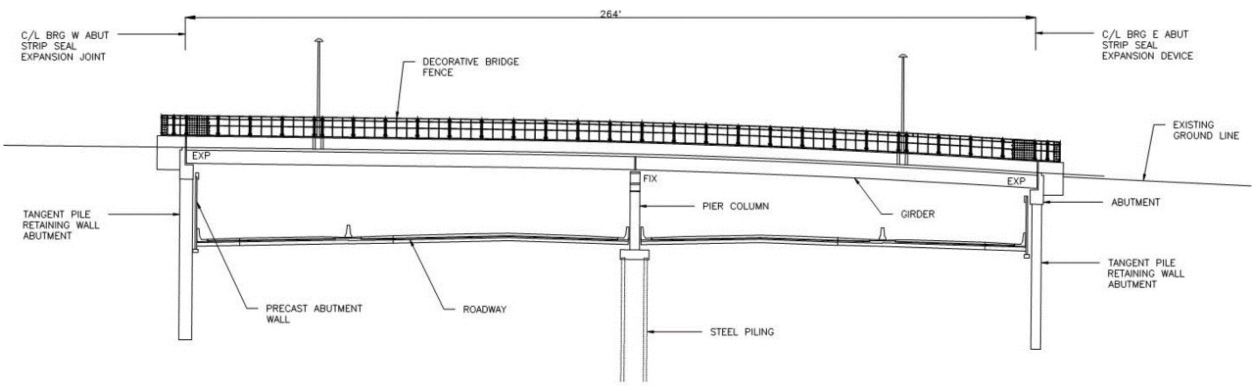
This chapter presents a step-by-step design process for three different bridges in different environmental conditions. The first two examples were first developed through the SHRP2 R19A project and are repeated here using the AASHTO Guide Specification. The third example is new and demonstrates a service life design of a signature structure. The chapter concludes with a comparison of the SHRP2 R19A and AASHTO Guide Specification methodologies using one of the examples. Use of these methodologies is not required by Federal law. They are included here for illustrative purposes only.

The examples are targeted toward bridge owners and bridge engineers who are performing a service life design using the AASHTO Guide Specification. They are provided solely for to demonstrate the methodology of the AASHTO Guide Specification. Service life design of actual bridge structures will vary from the examples depending on local and project specific characteristics. There is no Federal requirement to use the Guide Specification.

EXAMPLE 1: MULTI-SPAN STEEL GIRDER BRIDGE SUBJECT TO HEAVY DEICING SALT APPLICATION AND FREEZE-THAW CYCLES

Introduction

A conventional multiple span composite-deck highway overpass bridge is designed for service life in accordance with the AASHTO Guide Specification for Service Life Design of Highway Bridges (2020) (herein “Guide Specification”). The bridge is located in the Northeast United States where deicing salts are heavily used and freeze-thaw cycles are common. Figure 3 shows the general elevation of the bridge.



Source: Pease et al. (2019)

Figure 3. Illustration. Bridge general elevation.

Location

The bridge has the following location information:

- New York, NY

- Highway under the bridge with a minimum vertical clearance of 16-feet and 12-feet shoulder widths.
- Urban environment with periods of snow and freeze-thaw cycles.
- Heavy use of deicing salts.
- Some sulfate present in the soil: 0.14 percent by mass of soluble sulfate.

Components and Features

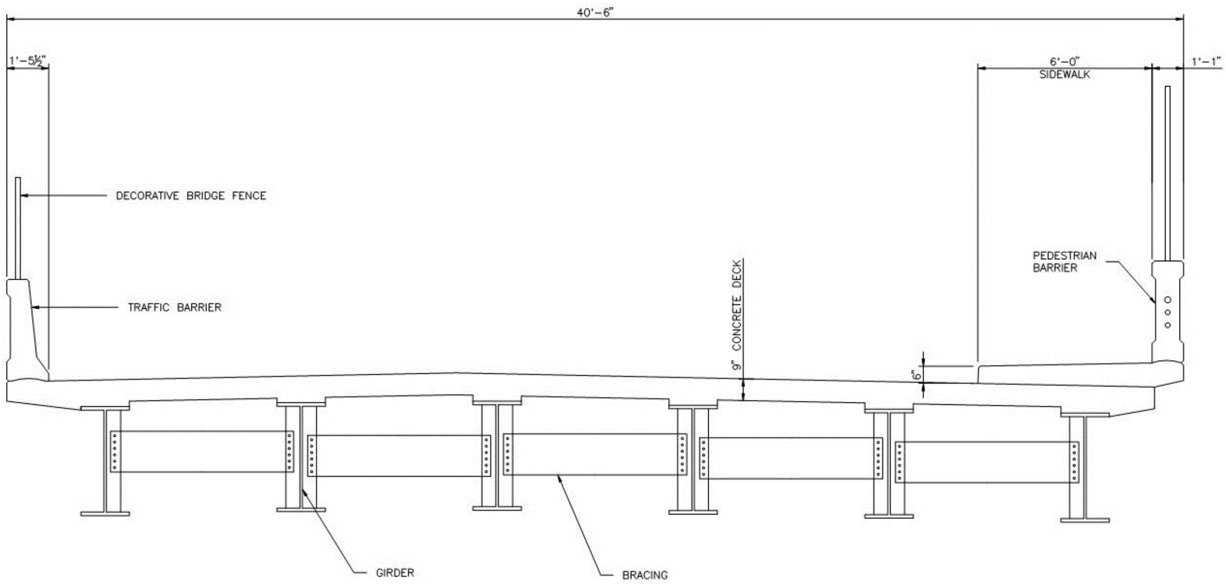
The bridge is a typical highway bridge with features described below.

Superstructure

The superstructure is composed of the following components:

- 2 span continuous steel girders with span lengths of 139-feet and 125-feet (total bridge length equals 264-feet).
- Composite cast-in-place concrete deck, 9-inches thick.
- Elastomeric expansion bearings at the abutments, fixed bearings at the piers.
- Two 12-feet wide traffic lanes with a 6-feet wide sidewalk on one side
- A pedestrian barrier adjacent to the sidewalk, and a traffic barrier on the opposite side of the roadway. Both barriers support a decorative fence.
- Concrete deck without an overlay or waterproofing membrane.
- Assumed that uncoated (black) steel reinforcement will be used throughout unless otherwise governed by the design. The deck will use epoxy coated reinforcement, at minimum.

The typical superstructure cross-section is shown in Figure 4.



Source: Pease et al. (2019)

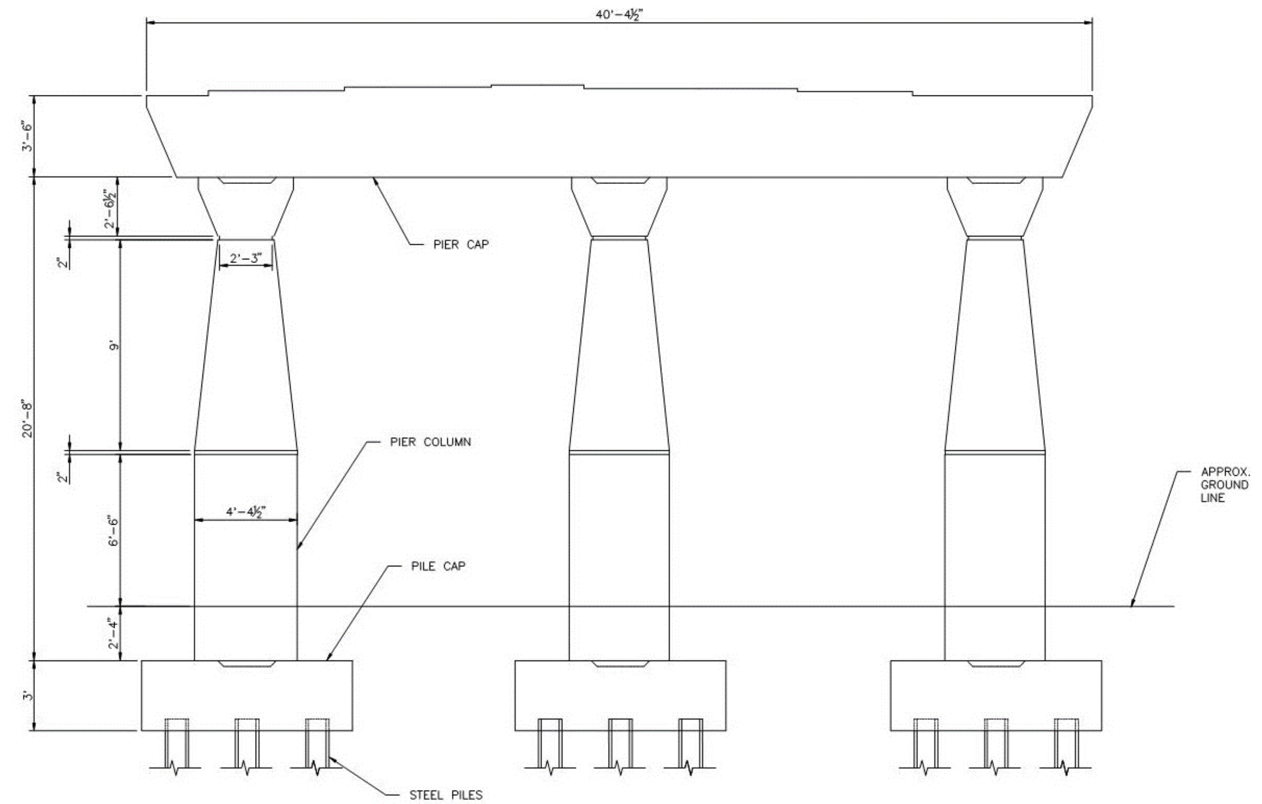
Figure 4. Illustration. Typical superstructure cross-section.

Substructure

The substructure is composed of the following components:

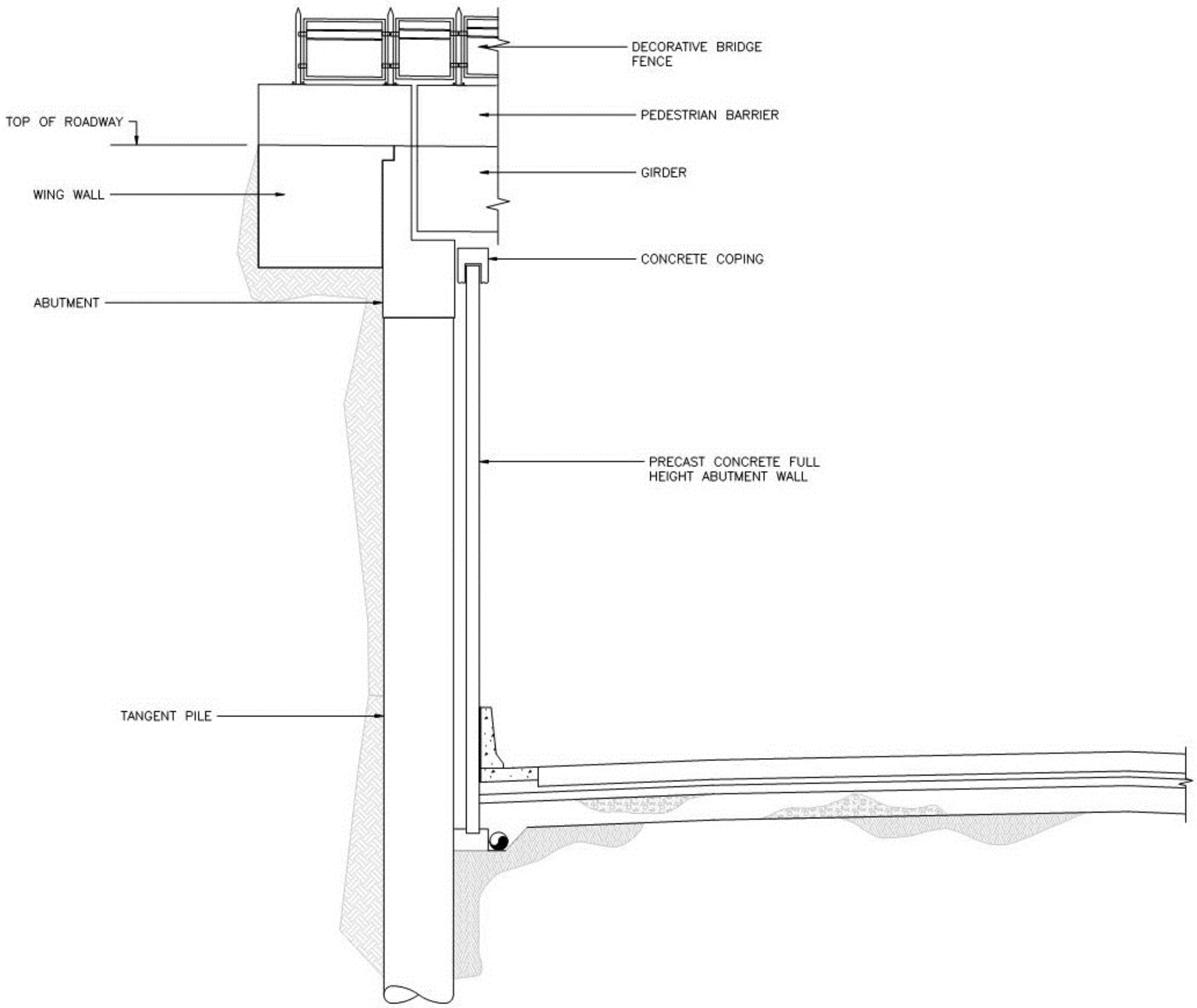
- Three-column pier supported by pile caps and steel H-piles driven into bedrock.
- Abutments supported on reinforced concrete tangent piles.
- Full height precast walls in front of abutments as a protection measure.
- Expansion joints between abutments and concrete deck. A jointless alternative will be evaluated.
- Assumed that uncoated reinforcement used everywhere unless otherwise governed by the design.

Elevation views of the pier and abutments are shown in Figure 5 and Figure 6, respectively.



Source: Pease et al. (2019)

Figure 5. Illustration. Pier elevation view.



Source: Pease et al. (2019)

Figure 6. Illustration. Abutment elevation view.

Classification

The bridge is classified in terms of target service life and the environment according to Section 2 of the Guide Specification.

Target Service Life

The target service life for nonreplaceable and renewable bridge components is determined using Guide Specification Article 2.1.1.

Nonreplaceable

The structure is classified as Normal according to Article 2.1.1 and Table 2.1.1-1 of the Guide Specification since it is a typical highway bridge and there are no criteria provided that warrant an increased service life category. Therefore, the Good level of qualitative practice is assigned to nonreplaceable components as summarized in Table 6.

Table 6. Service life category selection for nonreplaceable components.

Category	Component	Bridge Description	Level of Qualitative Practice
Normal	• Foundations • Abutments • Piers • Structural steel • Deck	Typical Bridge	Good

Renewable

The target service lives for renewable components are selected by the Engineer. As described in Article 3.2.2 of the Guide Specification, the design should account for the interaction of bridge components, replacement of renewable elements, and the service life of individual components in relation to the total bridge service life.

Any expansion joints employed on the bridge will need to be replaced multiple times throughout the bridge's service life. Based on the ranges provided in Table 7.1.1-1 of the Guide Specification, a service life range between 8 and 50 years is achievable for joints. Staying within this range and considering whole divisions of 75, a reasonable target service life is 25 years. In addition, a jointless design will be evaluated as an alternative.

The target service life for bearings will be selected depending on the joint configuration. Certain bearing types have the potential to last at least 75 years based on Article 7.2.2 of the Guide Specification. For the expansion joint option, conservatively assuming bearing deterioration from joint failure will result in one bearing replacement, a target service life equal to half of the bridge service life (or about 40 years) is selected. In the event the jointless option is selected, a target service life of 75 years for the bearings is reasonable.

The target service life for paint and coating systems will vary depending on the Owner's available standard paint and coating systems and their experience with these products. At minimum, 25 years is desired. The actual service life of paint and coating systems is designed in later sections of this example.

The barriers, decorative fence, and the protective walls in front of the abutments are all assumed to be renewable components. The target service lives of the fence and barrier are set to be equal to each other such that these components can be replaced at the same time for practical purposes. For all three of these components, a target service life equal to approximately half of the bridge service life is selected (40 years). Barriers may need earlier replacement due to impact damage.

Target service lives for renewable components are summarized in Table 7.

Table 7. Target service lives for renewable components.

Component	Target Service Life (years)
Bearings	40
Expansion Joints	25
Paint/Coatings	25
Barriers/Fence	40
Precast Abutment Walls	40

Environmental Classification

Exposure Zones

Macro

Based on the provided location and climate features and referencing Article 2.2.1.1 of the Guide Specification, the following can be inferred regarding the macro exposure zones:

- The heavy use of deicing salts places the superstructure in a Deicing Zone
- The highway under the bridge also puts the substructure in a Deicing Zone

No additional location of climate information is provided so it is assumed that other macro exposure zones are not applicable.

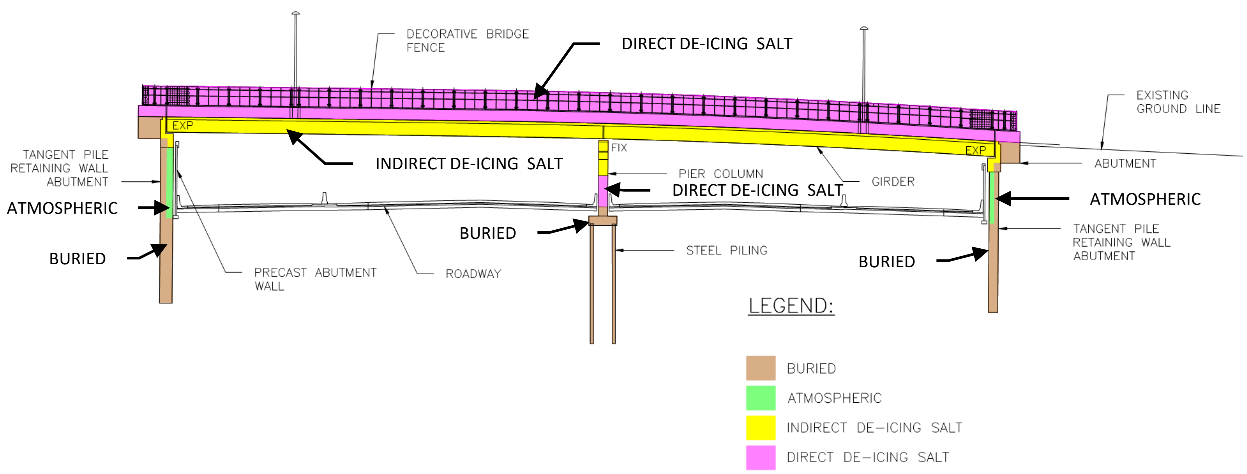
Micro

Referencing the criteria from Article 2.2.1.2 of the Guide Specification, applicable micro exposure zones for each component are listed in Table 8. Note that based on the provided vertical clearance (16 ft) and shoulder width (12 ft), and using the limits from Figure 2.2.1.2-2 of the Guide Specification, the components above and adjacent to the highway are within the roadway splash/spray zone.

Table 8. Micro exposure zones.

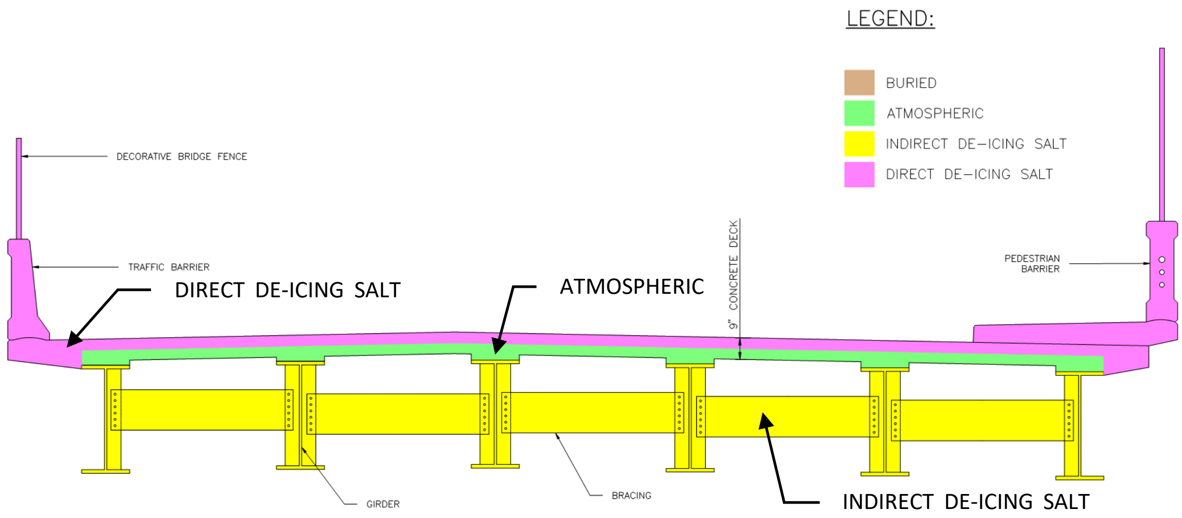
Exposure Zone	Component	Description
Buried	• Tangent Piles • H-Piles • Pile Caps • Abutments (wing walls, base) • Pier Columns (base)	• Permanently buried in soil.
Atmospheric	• Tangent Piles (front face) • Deck (bottom)	• Not exposed to soil, water, or deicing salts.
Indirect Deicing	• Abutments (backwall face, seat) • Precast Abutment Walls (top portion) • Pier Columns (top portion) • Pier Cap • Bearings • Structural Steel	• Indirect exposure to deicing salts due to runoff and joint failure. • Indirect exposure to deicing salts within the roadway splash/spray zone of the highway below the bridge.
Direct Deicing	• Precast Abutment Walls (bottom portion) • Pier Columns (bottom portion) • Deck (top) • Barriers/Fence	• Directly exposed to deicing salts.

Figure 7 through Figure 10 visually show the exposure zones assigned to the bridge components.



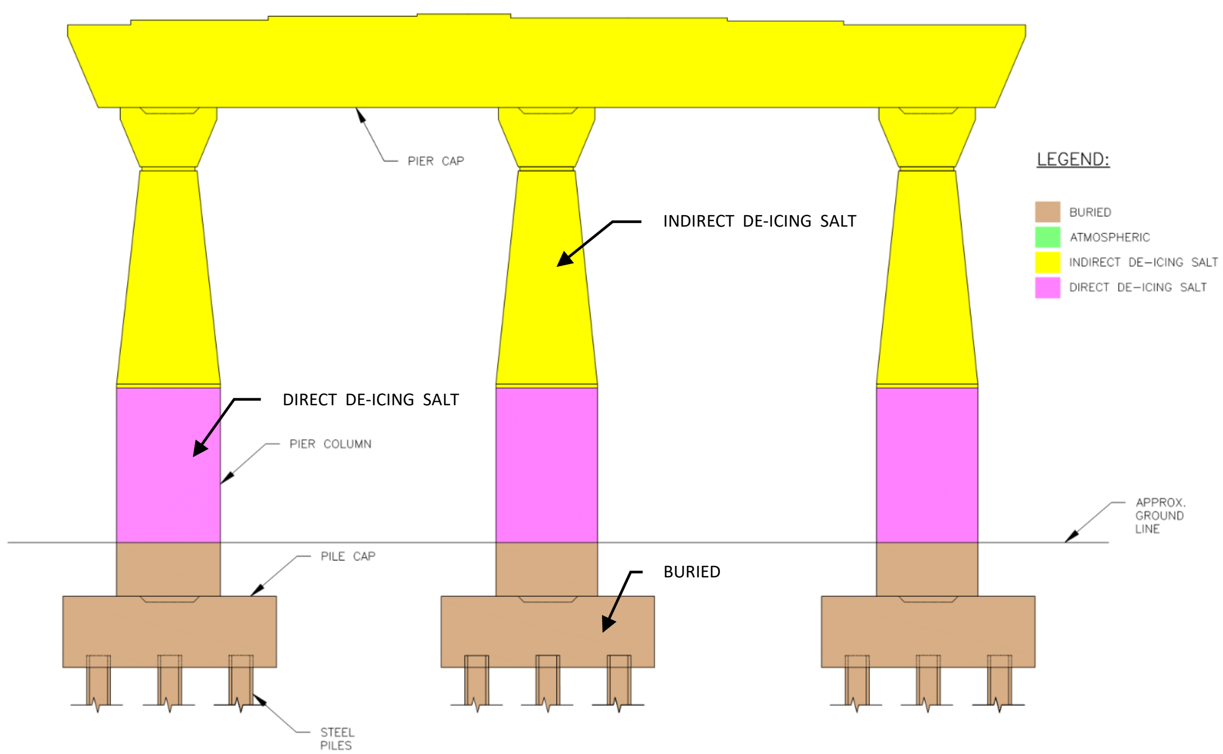
Source: adapted from Pease et al. (2019)

Figure 7. Illustration. Exposure zones shown in bridge general elevation view.



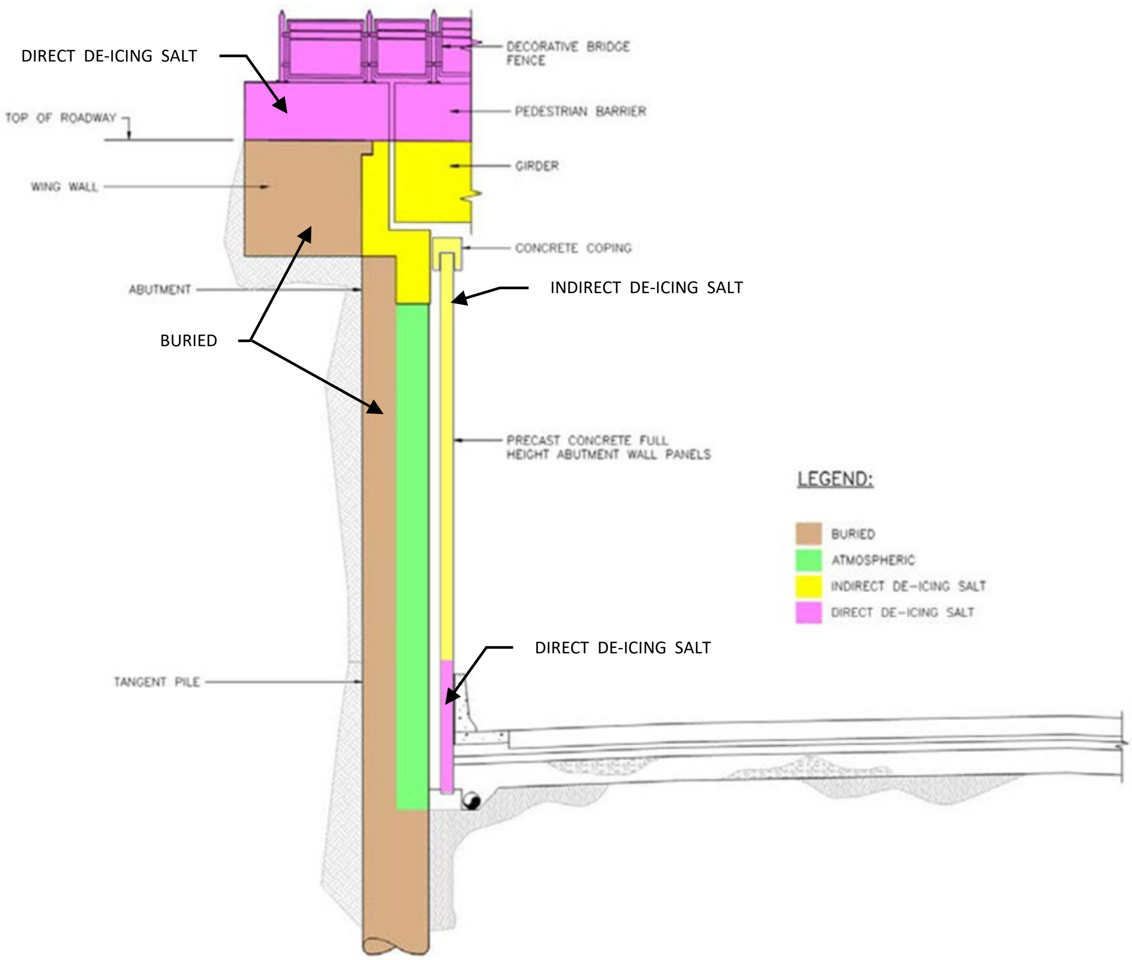
Source: adapted from Pease et al. (2019)

Figure 8. Illustration. Exposure zones for superstructure components.



Source: adapted from Pease et al. (2019)

Figure 9. Illustration. Exposure zones for pier components.



Source: adapted from Pease et al. (2019)

Figure 10. Illustration. Exposure zones for abutment components.

Exposure Classes

The previously described exposure zones, applicable deterioration mechanisms, and the criteria in Guide Specification Table 2.2.2.1-1 and 2.2.2.2-1 for concrete and steel components, can be used, respectively, to assign exposure classes. Assigned exposure classes for concrete components are shown in Table 9, and those for steel components are given in Table 10.

Table 9. Concrete exposure class assignments.

Exposure Zone	Components	Exposure Conditions	Chloride Induced Corrosion	Carbonation Induced Corrosion	Freeze-thaw	AAR	Sulfate Attack	Abrasion	Preservice Cracking
Buried	• Abutment tangent piles • Abutments • Pile caps • Pier columns	• Freeze-thaw above the frost line • Sulfates in the soil	C-B	-	FT1	X	S1	-	-
Atmospheric	• Abutment tangent piles • Deck soffit	• Some airborne chlorides and carbon dioxide • Freeze-thaw potential	C-D1	C-D1	FT2	X	-	-	X
Indirect Deicing	• Precast abutment walls • Abutments • Pier columns • Pier cap • Bearings	• Cycles of wetting and drying due to roadway splash/ spray and runoff • Freeze-thaw potential with frequent exposure to water and chlorides	C-D2	C-D2	FT3	X	-	-	X
Direct Deicing	• Pier columns • Deck top surface	• Direct exposure to deicing salts • Freeze-thaw potential with frequent exposure to water and chlorides	C-D4	C-D4	FT3	X	-	X (Deck top surface)	X

"X" indicates applicable deterioration mechanism but without a specific Exposure Class in the nonbinding Guide Specification for that deterioration mechanism

Table 10. Steel exposure class assignments.

Exposure Zone	Components	Exposure Conditions	Corrosion	Fatigue
Buried	• H-Piles	• Sulfates in the soil • Moisture in soil	See Section 6	-
Indirect Deicing	• Girders • Diaphragms	• Moderate salinity area	C4	F
Direct Deicing	• Fence	• High salinity area	C5M	-

Design of Concrete Components

Material Parameters

Service life design parameters for concrete components can be determined according to Section 4 of the Guide Specification. Use of the Guide Specification is not a Federal requirement. Based on material availability and Owner standard mixes, the following concrete classes can be specified for the project, as defined in Section 8 of the *AASHTO LRFD Bridge Construction Specifications 4th edition* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv)):

- An ordinary portland cement (OPC) mix with air entrainment (AE) for general structural applications: Class A(AE)
- An OPC mix with AE typically reserved for substructures: Class B(AE)
- An OPC mix with AE for thin elements: Class C(AE)
- A high-performance concrete (HPC) mix with 20 percent fly ash replacement of portland cement: Class A(HPC)

The minimum material properties of each available mix are shown in Table 11. The concrete mix design type (or simply "concrete type") is also included for later use in determining concrete cover when designing for corrosion. The concrete class and concrete type should not be confused for one another. In the Guide Specification, the concrete class is the name given to a particular mix whereas the concrete type broadly categorizes the concrete based on cementitious material constituents and, more specifically, its chloride diffusion resistance. Article 4.2.4.2.2 of the Guide Specification includes descriptions of each concrete type.

Table 11. Available concrete mixes.

Concrete Class[1]	W/CM	Min f'_c (ksi)	Size of Coarse Aggregate[3]	Air Content Range (%)	SCM Substitutions (% mass of total cementitious material)	Concrete Type[4]
A(AE)	0.45	4.0	1.0 in. to No. 4	4.5 – 7.5	-	OPC
B(AE)	0.55	2.5	2.0 in. to 1.0 in. and 1.0 in. to No. 4	3.5 – 6.5	-	OPC
C(AE)	0.45	4.0	0.5 in. to No. 4	5.5 – 8.5	-	OPC
A(HPC)	0.45	5.0[2]	1.5 in. to 0.25 in.	5.0 – 8.0[2]	20% Class F Fly Ash	OPCFA

[1]As defined in Section 8 of the *AASHTO LRFD Bridge Construction Specifications* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv))
[2]As specified in the contract documents or Owner standard specifications
[3]per AASHTO M43 (ASTM D448). Not a Federal requirement.
[4]Article 4.2.4.2.2 of the Guide Specification

General

Determine the general material parameters according to Guide Specification Article 4.2.1. Based on Table 4.2.1-1 of the Guide Specification, a maximum W/CM of 0.45 will be used since the bridge is exposed to deicing chemicals. Discussion for each relevant exposure class is contained in later sections.

Freeze-Thaw Attack

Using Guide Specification *Table* 4.2.1-1, Exposure Class FT concrete material parameters are summarized in Table 12. Additional parameters for target air content and corresponding available concrete classes are determined according to Guide Specification Table 4.2.2-1 and are presented in Table 13.

Table 12. Concrete material parameters for freeze-thaw attack.

Exposure Class	Component	Max W/CM	Min f'_c (ksi)	Additional Parameters	Min Concrete Class[1]
FT1	• Tangent Piles • Pile Caps • Abutments • Pier Columns	0.55	3.5	See Table 13	B(AE)
FT3	• Abutments • Pier Columns • Pier Caps • Deck	0.40	5.0	See Table 13	A(HPC)

[1]As defined in the *AASHTO LRFD Bridge Construction Specifications* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv))

Table 13. Additional parameters for freeze-thaw attack.

Nominal Maximum Aggregate Size (in)	Target Air Content for FT1 (%)	Available Concrete Class	Target Air Content for FT2, FT3 (%)	Available Concrete Class
0.5	5.5	C(AE)	7.0	C(AE)
1.0	4.5	A(AE)	6.0	A(AE)
1.5	4.5	A(HPC)	5.5	A(HPC)

Note that for elements in Exposure Class FT3 there are no available concrete mixes with a maximum W/CM of 0.40 or less. To correct this, the maximum W/CM of the Class A(HPC) concrete will be lowered to 0.40 and the adjustment will be specified in the contract documents. Also note that for elements in Exposure Class FT3, the cementitious materials limits from Guide Specification Table 4.2.2-2 apply. Therefore, for this project, the Class A(HPC) mix from Table 11 can have a maximum fly ash replacement of 25 percent.

Sulfate Attack

Components susceptible to sulfate attack should meet the concrete material parameters from Table 4.2.1-1 of the Guide Specification, as summarized in Table 14. Following the additional parameters of Guide Specification Article 4.2.3, the components in Exposure Class S1 would use a concrete with one of the cementitious materials types listed in Table 15.

Table 14. Concrete material parameters for sulfate attack.

Exposure Class	Component	Max W/CM	Min f'_c (ksi)	Additional Parameters	Min Concrete Class[1]
S1	• Tangent Piles • Pile Caps • Abutments • Pier Columns	0.50	4.0	See Table 15	A

[1]As defined in the *AASHTO LRFD Bridge Construction Specifications* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv))

Table 15. Additional parameters for sulfate attack.

Component	Exposure Class	Min Cementitious Materials Type
• Tangent Piles • Pile Caps • Abutments • Pier Columns	S1	ASTM C150 Type II ASTM C595 Type IP, IS, or IT with (MS) Designation. ASTM C1157 Type MS

Alkali Aggregate Reaction

From Table 9, all concrete components are potentially susceptible to alkali aggregate reaction (AAR). Based on Guide Specification Article 4.1.4, general approaches to reduce the risk for AAR that are applicable to this bridge include the use of low-alkali cement, nonreactive aggregates, and SCMs. Additional provisions from AASHTO R80 should be followed to mitigate AAR risks.

Abrasion

The top surface of the deck is susceptible to abrasion, particularly for this deicing environment where studded tires and plows may be used. Per Article 4.1.6 of the Guide Specification, the abrasion resistance is proportional to the concrete strength. The concrete to be used for the deck has a specified minimum compressive strength of 5 ksi which should provide sufficient abrasion resistance. In addition, the minimum cover for the top of the deck is increased by 1/4-inch in accordance with Article 4.2.4.2.2 of the Guide Specification.

Preservice Cracking

For those components susceptible to preservice cracking from Table 9, one of the main methods to reduce its potential will be to properly cure the concrete, as specified in Article 4.1.7 of the Guide Specification. The Owner specifies the concrete to be cured for 7 days minimum, which should provide protection from cracking if properly performed. In addition, the use of fly ash in the deck concrete mix will help to reduce the heat of hydration and subsequently the risk of preservice cracking.

Corrosion

Initial design assumptions for each concrete component were taken as follows based on common practice and Owner standard specifications:

- Tangent Piles: Uncoated reinforcement with 2-inches of cover.
- Pile Caps: Uncoated reinforcement with 3-inches of cover.
- Abutments: Uncoated reinforcement with 2-inches of cover.
- Pier Columns: Uncoated reinforcement with 2-inches of cover.
- Pier Caps: Uncoated reinforcement with 2-inches of cover.
- Deck: Epoxy coated bars with 3-inch top cover and 1-1/2-inch bottom cover.

For all epoxy coated bars, Class A reinforcement was conservatively assumed which is similar to black uncoated reinforcement. Referencing Guide Specification Table 4.2.4.2.2-1 and the available concrete mixes from Table 11, design options for each component in terms of concrete type, reinforcement class, and cover are listed in Table 16. Note that improved reinforcement classes were not considered for this example.

Table 16. Exposure Class C design options.

Exposure Class	Component	Concrete Type	Reinforcement Class	Cover (in)[1]
C-B	• Tangent Piles • Pile Caps • Abutments • Pier Columns	OPCFA OPC	A A	1.0 2.5
C-D1	• Tangent Piles	OPCFA OPC	A A	2.0 2.0
C-D1	• Deck Soffit	OPCFA OPC	A A	1.5 1.5
C-D2	• Abutments • Pier Columns • Pier Caps	OPCFA OPC	A A	2.5 N/A
C-D4	• Pier Columns • Deck Top and Overhangs	OPCFA OPC	A A	3.0 N/A

[1]"N/A" indicates cover is too large for the combination of Concrete Type and Reinforcement Class

Where possible, the same concrete type should be selected for similar elements for practicality purposes and ease of construction. For example, rather than specifying two different concrete types for the foundation elements (i.e., tangent piles and pile caps), the cover was adjusted and one concrete type was specified. Also, it is impractical to vary the concrete type within one element (e.g., pier columns, deck) with the Exposure Class. Rather, the concrete type was selected to meet the most stringent Exposure Class.

Based on the options listed in Table 16, the following observations and design decisions can be made:

- Tangent Piles: Increase the cover to 2-1/2-inches and use OPC concrete.

- Pile Caps: Reduce the cover to 2-1/2-inches and use OPC concrete.

- Abutments: Increase the cover to 2-1/2-inches and use OPCFA in order to meet Exposure Class C-D2.

- Pier Columns: Increase the cover to 3-inches and use OPCFA concrete in order to meet Exposure Class C-D4.

- Pier Caps: Increase the cover to 2-1/2-inches and use OPCFA in order to meet Exposure Class C-D2.

- Deck: Use OPCFA in order to be consistent with the exposure class parameters for both the top and bottom surfaces. Add 1/4-inch to the top cover to account for abrasion.

The concrete type, reinforcement class, and cover values should be based on the governing corrosion Exposure Class as given in Table 17.

Table 17. Selected Exposure Class C designs.

Component	Controlling Exposure Class	Concrete Type	Reinforcement Class	Cover (in)
Tangent Piles	C-B	OPC	A	2.5
Pile Caps	C-B	OPC	A	2.5
Abutments	C-D2	OPCFA	A	2.5
Pier Columns	C-D4	OPCFA	A	3.0
Pier Caps	C-D2	OPCFA	A	2.5
Deck	C-D4	OPCFA	A	1.5 (Bottom) 3.25 (Top)

Summary

The final mix designs are shown in Table 18. The following adjustments were made as a result of the design specifications:

- Class B(AE) was eliminated from consideration because it did not meet any of the Exposure Class parameters for any component.

- The minimum compressive strength of Class A(AE) was increased to 4.5 ksi such that this concrete class could be used for the tangent piles and pile caps and meet the general parameters of Table 13 and the corrosion parameters of Table 16. This will be used for the OPC mix from Table 17; therefore, the W/CM will be reduced to 0.40 in accordance with Article 4.2.4.2.2 of the Guide Specification.

- Class A(HPC) will be used to meet the OPCFA concrete type from Table 17. The W/CM of Class A(HPC) will be reduced to 0.40 in accordance with Exposure Class FT3 and Article 4.2.4.2.2 of the Guide Specification for concrete cover.

- Class C(AE) will be used for renewable concrete components (see later sections). The W/CM is reduced to 0.40 in accordance with Exposure Class FT3.

Table 18. Final concrete mixes.

Concrete Class[1]	W/CM	Min f'_c (ksi)	Size of Coarse Aggregate[3]	Target Air Content (%)	SCM Substitutions (% mass of total cementitious material)	Cementitious Materials Type
A(AE)	0.40[4]	4.5[4]	1.0 in. to No. 4	4.5	-	Type II or Type IP
C(AE)	0.40[4]	4.0	0.5 in. to No. 4	7.0	-	Type II or Type IP
A(HPC)	0.40[4]	5.0[2]	1.5 in. to 0.25 in.	5.5[2]	20-25% Class F Fly Ash	Type II or Type IP

[1]As defined in the *AASHTO LRFD Bridge Construction Specifications* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv))
[2]As specified in the contract documents or Owner standard specifications
[3]Per AASHTO M43 (ASTM D448) (nonbinding)
[4]Adjusted to meet design parameters

The final concrete type, concrete class, reinforcement class, and cover to be used for each component are listed in Table 19.

Table 19. Concrete material design summary.

Component	Concrete Type	Concrete Class	Reinforcement Class	Cover (in)
Tangent Piles	OPC	A(AE)	A	2.5
Pile Caps	OPC	A(AE)	A	2.5
Abutments	OPCFA	A(HPC)	A	2.5
Pier Columns	OPCFA	A(HPC)	A	3.0
Pier Caps	OPCFA	A(HPC)	A	2.5
Deck	OPCFA	A(HPC)	A	1.5 (Bottom) 3.25 (Top)

Other Protection Strategies

In addition to the material design parameters determined previously, other strategies to consider for the protection of the concrete elements include sealers, proper detailing, and deck-specific design.

Surface Sealers

Due to the heavy use of deicing salt application anticipated for this bridge, as well as the presence of the highway below creating a splash/spray zone, there is a high chance of chloride exposure for many of the exposed concrete surfaces. Surface sealers, as discussed in Guide Specification Article 4.3.1, are an added layer of protection when used in conjunction with the previously designed material properties and cover dimensions.

A state-approved coating-type sealer is available for use on the project. Therefore, this sealer will be specified to be applied to certain concrete surfaces. In general, these surfaces will be those assigned Exposure Class C-D2 or worse, which include:

- Abutment backwalls, pedestals, seats, and exposed vertical surfaces.
- Pier caps and exposed surfaces of the pier columns.

Deck-Specific Design

It is not a common practice to use an overlay and waterproofing membrane as described in Guide Specification Articles 4.3.3.1 and 4.3.3.2, respectively, on a new bridge deck; however, it is not exceptionally unusual either. For this design, these protection strategies will not be employed at initial construction, but should be considered in the future to extend the deck service life.

Another strategy is to use a deck-specific sealer. Typical Owner specifications include a penetrating sealer on the riding surface of all new bridge decks. Therefore, an approved silane or siloxane penetrating sealer will be specified in the contract documents.

In addition, the concrete curing suggestions in Guide Specification Article 4.3.3.3 should be considered for this project. Specific to this design:

- A deck placement sequence should be developed due to the continuous span condition.
- Make the cure period as long as possible (preferably 14 days).
- Deck loads should be restricted until sufficient strength has been verified.
- Local environmental and weather conditions should be accounted for during construction, and appropriate limitations should be adhered to (e.g., temperature, wind).

Design of Steel Components

Design the steel components, which include the H-piles, girders, and diaphragms, according to Section 5 of the Guide Specification. Deterioration mechanisms include corrosion and fatigue.

Corrosion Protection

Article 5.2.1 of the Guide Specification is used in this example to develop corrosion protection strategies for the girders and diaphragms. The design of H-piles for corrosion is discussed in a subsequent section.

Steel Type Selection

Based on the considerations listed in Article 5.2.1.2 of the Guide Specification, uncoated weathering steel may not be suitable for the C4 exposure zone. Since the bridge is a highway overpass, there is the potential for tunnel-like conditions in which excessive roadway spray from the highway below is deposited on the girders and diaphragms. In combination with the heavy use of deicing salts, these exposure zone conditions are not ideal for uncoated weathering steel without a site-specific study. Therefore, a coating system is used in this example. While Table 5.2.1-1 of the Guide Specification does not specifically address the exposure class of the girders and diaphragms (C4), it will be used to assume a protection strategy. For a Normal service life category and conservatively assuming the next worse exposure class (C5-I), the suggested protection strategy is to use a coated non-weathering steel. This is summarized in Table 20.

Table 20. Steel corrosion protection strategy selection.

Component	Service Life Category	Exposure Class	Corrosion Protection Strategy
• Girders • Diaphragms	Normal	C4	Coated Non-Weathering Steel Grade

Unless otherwise dictated by the structural design, the commonly available nonregulatory ASTM A709 Grade 50 may be specified for the girders and diaphragms.

Coating System Design

Options for coating systems include galvanization, metallization, and paint. Galvanization is assumed to be impractical due to girder segment length limitations. Metallization has been determined to be cost-ineffective for this project. Therefore, a paint system will be used. Governing standard specifications include options for a three-coat system with a zinc primer. The available paint systems are listed in Table 21. Using the information provided in Guide Specification Appendix A5 (Helsel and Lanterman, 2018), the practical maintenance time, P, for these paint systems is estimated based on exposure class.

Table 21. Available paint coating systems and their practical maintenance times.

Type	Coating System	Surface Preparation	Number of Coats	Min. DFT (mils)	C3 (years)	C4[1] (years)	C5I (years)	C5M (years)
Epoxy Zinc	Epoxy Zinc/ Epoxy/ Polyurethane	Blast	3	10	20	17	14	14
Organic Zinc	Organic Zinc/ Epoxy/ Polyurethane	Blast	3	10	18	15	13	13

[1]Average of C3 and C5I

The chosen paint system will not last for the entire service life; therefore, the systems will have to be maintained at regular intervals. The maintenance intervals will vary depending on the exposure class and the type of coating system. Table 22 shows the maintenance painting sequence options for each combination of exposure class from Table 10 and available coating type from Table 21. The timing for initial touch-up, maintenance repaint (M), and full repaint (R) given in Table 22 follows the information provided in Appendix A5 of the Guide Specification.

Table 22. Maintenance painting sequence options.

Exposure Class	Coating Type	Initial Touch-Up (Year)[1]	Maintenance Repaint, M (Year)[2]	Full Repaint, R (Year)[3]
C4	Epoxy Zinc	17	25	34
C4	Organic Zinc	15	22	30
C5M	Epoxy Zinc	14	21	28
C5M	Organic Zinc	13	19	26

[1]Equal to Practical Maintenance Time, P
[2]Equal to 1.5P
[3]Equal to M+0.5P

Using the information from Table 22, the timing of painting for each component, exposure class, and coating type is calculated as shown in Table 23. The timing should be designed such that the total life of the coating system (including repainting multiple times) exceeds the target service life of the component that the coating system is applied to. Based on the Normal target service life category (approximately 75 years) from Table 7, the Epoxy Zinc coating system will be selected for the girders and diaphragms since this system includes the least number of painting operations. The Organic Epoxy coating system could also be considered as a viable option assuming the superstructure or total bridge is replaced exactly at 75 years. However, this is an unconservative assumption given that many bridges are left in service beyond their design service life. For the fence, the target service life is 40 years (see Table 7); therefore the Epoxy Zinc coating system will also be used. This choice plans for a single full repaint with no additional touch-up and has the added benefit of using one coating system for all components on the project.

A copy of Table 23 should be included in the Service Life Design Manual for bridge management to timely schedule maintenance painting activities.

Table 23. Design maintenance painting timing.

Component	Exposure Class	Coating Type	Initial Painting (Year)	P_1 (Year)	M_1 (Year)	R_1 (Year)	P_2 (Year)	M_2 (Year)	R_2 (Year)	P_3 (Year)	M_3 (Year)
Girders, Diaphragms	C4	Epoxy Zinc	0	17	25	34	51	59	68	85[1]	–
Girders, Diaphragms	C4	Organic Epoxy	0	15	22	30	45	52	60	75	82[1]
Fence	C5M	Epoxy Zinc	0	14	21	28	49[1]	–	–	–	–
Fence	C5M	Organic Epoxy	0	13	19	26	39	58[1]	–	–	–

[1]Indicates component replacement prior to this year

Service life, cost, and difficulty of application and re-application should be considered in selecting a coating system. The choice of coating system should be based on a life cycle cost analysis (LCCA), as outlined in Section 8 of the Guide Specification. Compliance with the Guide Specification is not required by Federal law. For brevity, a LCCA is not performed for this example.

Fatigue Design

The designer must refer to Section 6 of the *AASHTO LRFD Bridge Design Specifications* (2017b) (23 CFR 625.4(d)(1)(v)) to determine the design for each fatigue detail.

Design of Foundations

Tangent Piles

A design of the tangent piles following the nonbinding Guide Specification would proceed as outlined herein. The design for the tangent piles falls under the provisions of Article 6.3.1 of the Guide Specification (Nongravity Cantilever Walls), which defers to Article 6.2.1. Based on the sulfate concentration in the soil of 0.14 percent, a Protection Index (P_I) value of 2 should be assigned according to Article 6.2.1.1. Using P_I = 2 and Table 6.2.1.1-1 of the Guide

Specification, the choice of Exposure Class C-B from Table 9 is justified and does not need to be modified.

In terms of protection strategies, from Table 6.2.1.2-1 a $P_I = 2$ is contingent on the use of appropriate cover, concrete mix type, and reinforcement class from Table 4.2.4.2.2-1 of the Guide Specification. These parameters were previously addressed in the concrete material design. As an added protection strategy, the design could specify a 1-foot thick encapsulation of compacted nonaggressive structural fill as listed in Article 6.2.1.2 of the Guide Specification

Pile Caps

The design for the pile caps is included in Article 6.2.1 of the Guide Specification and therefore will be the same as above for the tangent piles.

H-Piles

As previously assigned in Table 10, corrosion is a concern for the H-piles. Based on the high sulfate concentration in the soil combined with the likelihood of high chloride concentration water within the soil due to deicing-laden runoff, the example assigns a P_I of 2 to the local deterioration environment from Tables 6.2.2.1-1 and 6.2.2.1-2 of the Guide Specification.

From Table 6.2.2.2.1-1 of the Guide Specification, a $P_I = 2$ is contingent on the steel area of the piles being increased as well as an additional protection strategy to be employed. Using Figure 6.2.2.2.1-1 of the Guide Specification and conservatively assuming the 95 percent maximum probable corrosion rate for piles in the Buried Zone, the corrosion loss is calculated as 0.15 inches (i.e., 0.0020 inches/year multiplied by 75 years). Therefore, a thickness of 3/16-inch should be added to each side of each pile and a new section size should be selected accordingly. Options for the second protection strategy include:

- Protective coating
- Concrete encasement
- Cathodic protection, or
- Special steel alloy.

Based on the type and size of the bridge, the most cost-effective strategy is likely to be a protective coating. An approved coating should be selected and specified in the contract documents.

Design of Renewable Elements

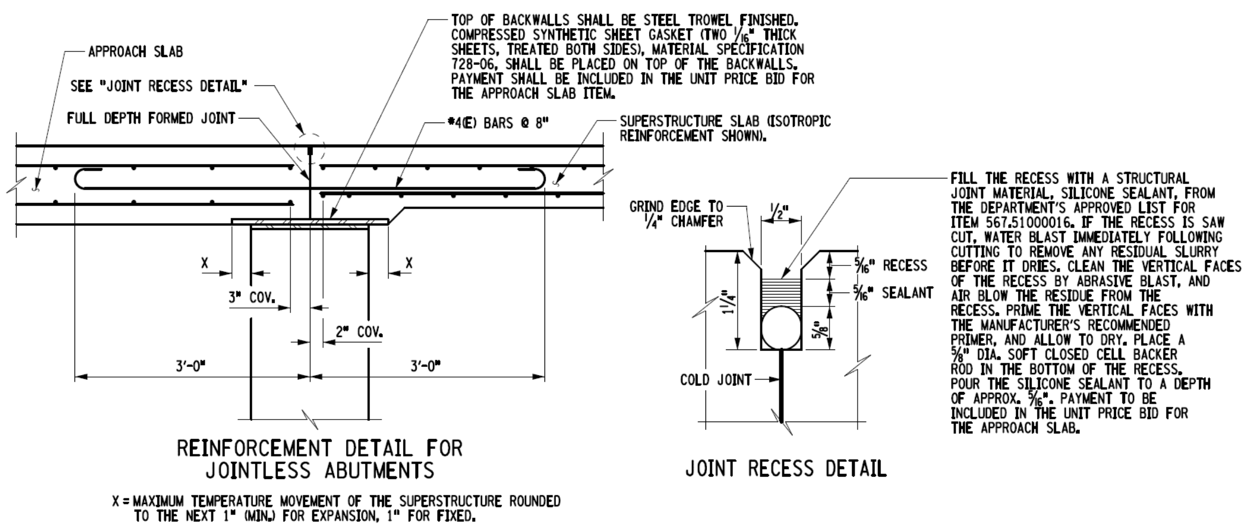
Expansion Joints

The heavy use of deicing salts in combination with the likelihood of failure if conventional expansion joints are used make the design of the joints a critical aspect of the service life design of the bridge. The Owner’s policy is to eliminate deck joints wherever possible. This bridge is a good candidate for a jointless deck system because it is non-skewed and has a length that is manageable for expansion. Considerations for jointless systems are outlined in Guide Specification Article 7.1.2.

For this bridge, a viable option is to make the deck slab continuous with the approach slab over the abutment backwall. Key aspects of this design include:

- Only a construction joint separates the deck slab and approach slab
- The temperature expansion and contraction are handled at the far ends of the approach slabs
- The superstructure accommodates expansion and contraction through the expansion bearings and by the approach slab sliding over the abutment backwall
- A bond breaker is applied to the top of the backwall, between the approach slabs and wingwalls, and to the approach slab over the subgrade at the expansion end.

An example of the jointless system described above is shown in Figure 11.



Source: NYSDOT (2019)

Figure 11. Illustration. Jointless system at abutments (NYSDOT 2019).

The use of a jointless system could allow for the revision to previous aspects of the service life design. For example, several exposure class assignments were based on the assumption that expansion joints were present at the abutments and that they would fail at some point (e.g., abutments, bearings). Using a jointless system allows the designer to revisit and possibly revise the exposure class assignments and dependent design parameters. For this example, the previously determined design parameters will conservatively remain unrevised.

While the proposed jointless detail from Figure 11 eliminates a conventional joint, there is still a small joint recess at the cold joint between the approach slab and deck slab that is filled with a structural joint material. Routine cleaning of this joint recess should be specified in the Service Life Design Manual.

Bearings

Design the bearings for service life in accordance with Article 7.2 of the Guide Specification. The bridge Owner commonly uses steel-reinforced elastomeric pads for this bridge type. Based on Table 7.2.1-1 and Article 7.2.2 of the Guide Specification, these types of bearings have a low cost, generally need low maintenance, and have the potential to last 75 to 100 years. Therefore, steel-reinforced elastomeric bearings will be specified for both the fixed and expansion bearings. The use of these bearings must be confirmed with a structural design in accordance with the *AASHTO LRFD Bridge Design Specifications* (2017b) (23 CFR 625.4(d)(1)(v)).

The steel components of the bearings should be protected from corrosion using a type of coating, as referenced in Article 7.2.3 of the Guide Specification. Anchor bolts for the bearings are commonly available galvanized; therefore, galvanization will be specified for all other steel components of the bearings (e.g., sole plates) for consistency.

Proper and routine maintenance activities, including inspection, cleaning, and evaluation of the bearing assemblies, typically are specified in a Service Life Design Manual.

Barriers

Based on the available concrete mixes from Table 11 and common barrier types, the following assumptions are made:

- Use Class C(AE) concrete (for thin elements)
- Epoxy-coated bars with 2-inches of cover (standard Owner practice)
- Stainless steel bars through the barrier to deck connection

The barriers were assigned to Exposure Classes FT3 and C-D4 from Table 9. In order to satisfy the parameters of Exposure Class FT3 from Table 12 and Table 13, the Class C(AE) mix will be modified to have a maximum W/CM of 0.40, a minimum f'_c of 5.0 ksi, and a minimum air content of 7 percent. Article 4.2.4.2.2 of the Guide Specification for Exposure Class C-D4 is not intended for the design of replaceable concrete components such as barriers due to the shorter target service life. It is reasonable to assume that the mix adjustments made above for freeze-thaw coupled with the use of epoxy-coated bars and standard cover will provide sufficient durability to meet the 40-year target service life of the barriers. In addition, using stainless steel bars to cross the construction joint between the barrier and deck reduces the risk of corrosion in this deterioration prone region.

Fence

The design of the protective coating for the steel decorative fence was previously demonstrated. In addition to coating the fence, the fence attachments to the barriers (i.e., anchor bolts) should receive a type of corrosion resistant coating. To be consistent with other aspects of the design, galvanized anchor bolts will be specified. The galvanization will prevent corrosion-induced fracture of the anchor bolts leading to fence detachment, as well as prevent cracking and spalling of the concrete barrier as a result of anchor bolt corrosion.

Precast Walls

Based on the available concrete mixes from Table 11 and common precast wall types, the following assumptions are made:

- Use Class C(AE) concrete (for thin elements)
- The walls are 5-inches thick with uncoated reinforcement with 1-1/2-inches of cover.

Similar to the barriers, the precast walls were also assigned to Exposure Classes FT3 and C-D4 from Table 9. The same adjusted Class C(AE) concrete mix mentioned above for barriers will be specified for the precast walls to meet Exposure Class FT3. As was stated in the barrier design, the replaceable precast walls do not need to meet the Exposure Class C-D4 parameters from Article 4.2.4.2.2 of the Guide Specification. The relatively small cover of 1-1/2-inches assumed for the precast walls and use of an OPC mix may not meet the target service life of 40 years. However, the walls are nonstructural and their replacement prior to the target service life is not detrimental to the service life of other components. Therefore, no additional adjustments will be made to the reinforcement type or cover.

Detailing

In this example, detailing measures will be employed to enhance durability, with a focus on drainage and conveying moisture off of the structure. This will be accomplished by:

- Providing sufficient scupper and downspout capacity at the abutments and piers, particularly at the lowest elevation abutment.
- Sloping the abutment seats (See nonbinding Guide Specification Figure C4.3.2-5).
- Detailing drip grooves in the deck overhang to limit moisture from reaching the girder top flanges (see nonbinding Guide Specification Figure C4.3.2-5).
- Specifying water stops at all cold joints including at the base of the precast abutment walls, between the abutment and wing walls, between the abutment and tangent piles, and at the base of the pier columns (see nonbinding Guide Specification Figures C4.3.2-8 and C4.3.2-9).
- Detailing to avoid debris traps, such as limiting the number of transverse stiffeners if possible and by providing adequate clip sizes and clearance where they are needed (See nonbinding Guide Specification Figure C5.3.1.1-1).
- Providing drip bars on the girder bottom flanges near the girder ends (low elevation points) so that moisture is shed from the girder prior to reaching the abutments (see nonbinding Guide Specification Figure C5.3.1.1-2).

- Extending any downspouts below the girder bottom flange (see nonbinding Guide Specification Figures C4.3.2-1 and C5.3.1.1-8).

Design Summary

Summaries of the service life design results for the concrete and steel components are provided in Table 24 and Table 25, respectively.

Design approaches for renewable elements include:

- Expansion Joints: Use a jointless system with expansion joints moved to the far ends of the approach slabs.

- Bearings: 40-year target service life. Use steel-reinforced elastomeric bearing pads with galvanized assembly parts.

Table 24. Service life design summary for concrete components.

Component	Target Service Life[1] (years)	Concrete Type	Concrete Class	Reinforcement Type	Cover (in)	W/CM	Minimum f'c (ksi)	Target Air Content (%)	Additional Parameters
Tangent Piles	–	OPC	A(AE)	Uncoated (black)	2.5	0.40	4.5	4.5	• Type II or IP Cement • 1-foot nonaggressive structural fill encapsulation
Pile Caps	–	OPC	A(AE)	Uncoated (black)	2.5	0.40	4.5	4.5	• Type II or IP Cement • 1-foot nonaggressive structural fill encapsulation
Abutments	–	OPCFA	A(HPC)	Uncoated (black)	2.5	0.40	5.0	5.5	• Type II or IP Cement • 20-25% Class F Fly Ash • Coating-type sealer for C-D2 surfaces
Pier Columns	–	OPCFA	A(HPC)	Uncoated (black)	3.0	0.40	5.0	5.5	• Type II or IP Cement • 20-25% Class F Fly Ash
Pier Caps	–	OPCFA	A(HPC)	Uncoated (black)	2.5	0.40	5.0	5.5	• Type II or IP Cement • 20-25% Class F Fly Ash • Coating-type sealer for C-D2 surfaces
Deck	–	OPCFA	A(HPC)	Epoxy-coated	1.5 (Bottom) 3.25 (Top)	0.40	5.0	5.5	• Type II or IP Cement • 20-25% Class F Fly Ash • Penetrating sealer for top surface
Barriers	40	OPC	C(AE)	Epoxy-coated	2.0	0.40	5.0	7.0	–
Precast Abutment Walls	40	OPC	C(AE)	Uncoated (black)	1.5	0.40	5.0	7.0	–

[1]For replaceable components only

Table 25. Service life design summary for steel components.

Component	Target Service Life[1] (years)	Corrosion Protection Strategy	Additional Parameters
Girders and Diaphragms (Superstructure Steel)	–	Non-Weathering Steel Grade with Epoxy Zinc coating system	• Fatigue design in accordance with the *AASHTO LRFD Bridge Design Specifications* (2017b) (23 CFR 625.4(d)(1)(v))
H-Piles	–	Increase section size by 3/16 in. min all around	• Protective coating
Fence	40	Epoxy Zinc coating system	• Galvanized anchor bolts

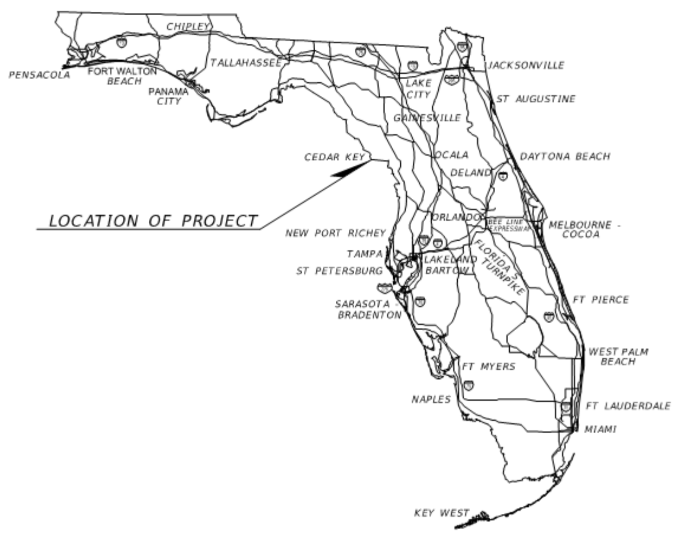
[1]For replaceable components only

EXAMPLE 2: MULTI SPAN PRESTRESSED CONCRETE GIRDER BRIDGE IN A COASTAL ENVIRONMENT EXPOSED TO SEA WATER AND/OR BRACKISH WATER

Introduction

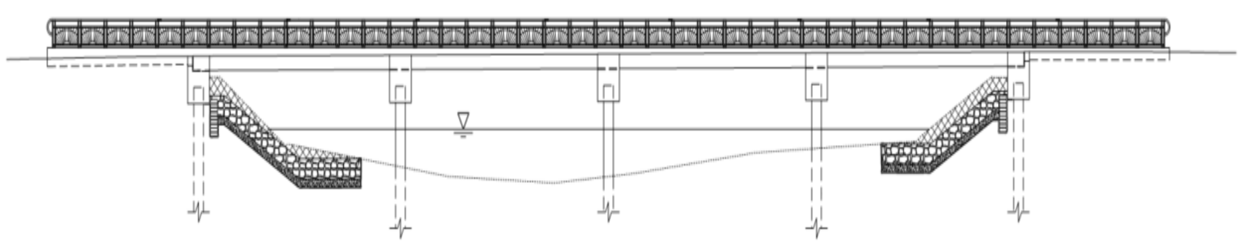
Location

This example is for a conventional multi span prestressed concrete girder bridge near the Cedar Key in Florida as shown in Figure 12. The bridge is on an island in sea water with salinity of 30,000 ppm. Figure 13 shows the elevation view of the bridge.



Source: Pease et al. (2019)

Figure 12. Location of the bridge in Cedar Key, Florida, USA.



Source: Pease et al. (2019)

Figure 13. Bridge elevation.

Components and Features

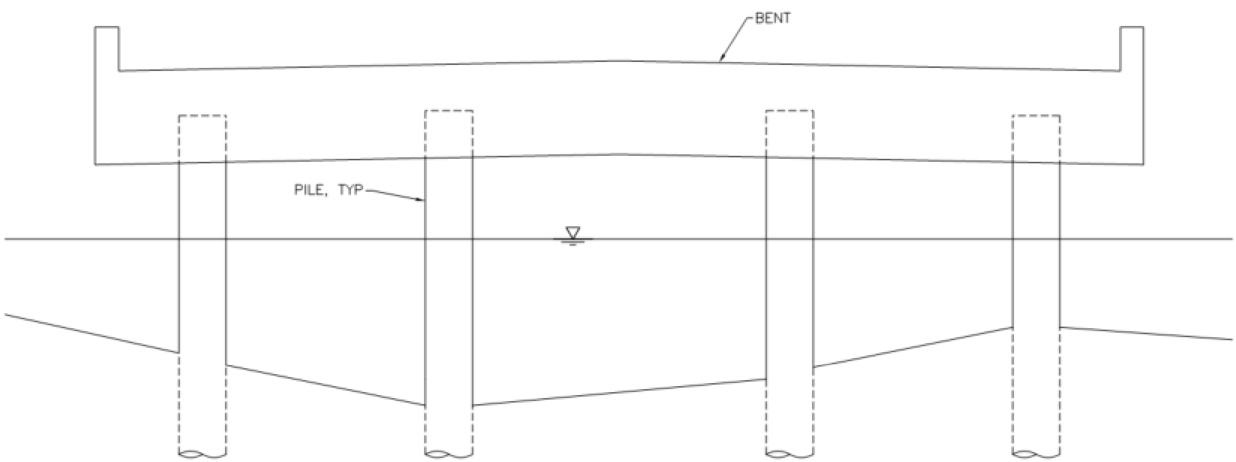
This is a conventional low-level multi-span pre-stressed concrete girder bridge.

Substructure

The substructure is composed of the following components:

- There are 5 bents, each supported on 4 stainless steel prestressed concrete piles.
- Expansion joints are located between the abutments and the concrete deck.
- Uncoated reinforcement (black steel) is used everywhere in the substructure except stainless steel is used in the piles.
- There are mass concrete components with a least dimension of 48 inches.

The typical substructure cross-section is shown in Figure 14.



Source: Pease et al. (2019)

Figure 14. Typical substructure cross-section.

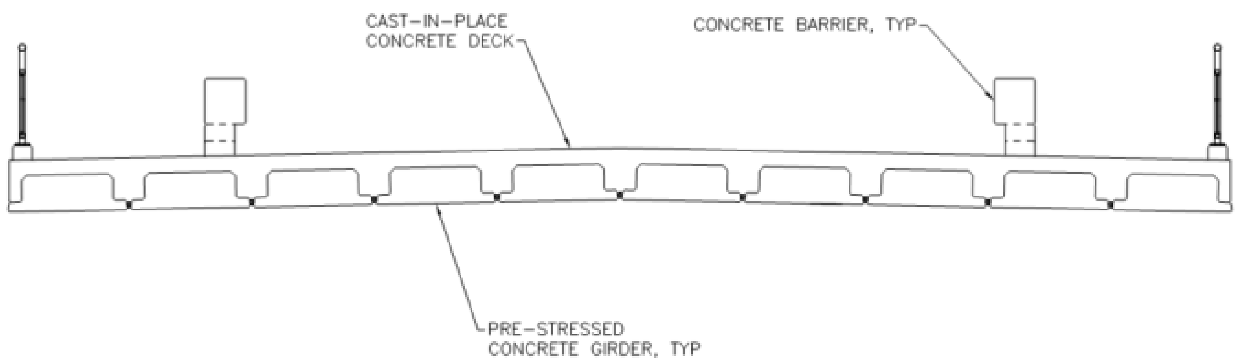
Superstructure

The superstructure is composed of the following components:

- Main span of 172 ft consisting of 4 spans at 43 ft each. Approach slabs are 30 ft long.
- Deck system is comprised of pre-stressed concrete girders with cast-in-place concrete deck.
- Over the end bents and abutments, the girders are supported on one bearing pads and at the intermediate bents, the girders are supported on two bearing pads.
- Deck carries two traffic lanes, each with a width of 10 ft., a shoulder of 3 ft., and a sidewalk with a width of 6 ft. on each side.

- Concrete barriers between the roadway and the sidewalks, and pedestrian railing along the outside edges of the deck.
- There is no asphalt or waterproofing membrane.
- Uncoated reinforcement (black steel) is used everywhere in the superstructure.

The typical superstructure cross-section is shown in Figure 15.



Source: Pease et al. (2019)

Figure 15. Typical superstructure cross-section.

Classification

The service life category for this bridge as directed by the Owner is normal.

Target Service Life

The target service life for nonreplaceable and renewable bridge components is determined using Guide Specification Article 2.1.1.

Nonreplaceable

This is a typical structure and classified as Normal according to Article 2.1.1. The Good level of qualitative practice is assigned to nonreplaceable components based on Guide Specification Table 2.1.1-1. Table 26 provides a summary of the service life category for the nonreplaceable components.

Table 26. Service life category of nonreplaceable components.

Category	Component	Bridge Description	Level of Qualitative Practice
Normal	• Bents • Piles • Girders • Deck	Typical Bridge	Good

Renewable

The target service lives for renewable components are selected by the Engineer. As described in Article 3.2.2 of the Guide Specification, the design should account for the interaction of bridge components, replacement of renewable elements, and the service life of individual components in relation to the total bridge service life.

Any expansion joints employed on the bridge will need to be replaced multiple times throughout the bridge's service life. Based on the ranges provided in Table 7.1.1-1 of the Guide Specification, a service life range between 8 and 50 years is achievable for joints. For this project, a target service life of 25 years is selected for the expansion joints.

The target service life for bearings will be selected assuming joint failure will accelerate bearing deterioration. Certain bearing types have the potential to last 75 years or more based on Article 7.2.2 of the Guide Specification. Conservatively assuming the bearing deterioration from joint failure will result in one bearing replacement, a target service life equal to half of the bridge service life (or about 40 years) is selected.

For the barrier, a target service life equal to approximately half of the bridge service life is selected (40 years) as they usually are in need of replacement at an earlier time due to impact damage.

Table 27 provides a summary of target service life for the renewable components.

Table 27. Target service life of renewable components.

Component	Target Service Life (years)
Bearings	40
Expansion Joints	25
Barriers	40
Pedestrian Handrailing	40

Environmental Classification

Exposure Zones

<u>Macro</u>
Based on the location of the bridge and climate features and referencing Article 2.2.1.1, the macro environment exposure zone for this structure is Marine with exposure to airborne salts and direct contact with sea water.

Based on the data set available from a nearby weather station in Cedar Key, the average annual temperature is 70.95˚F and a standard deviation of 1.24˚F. There is no known occurrence of snow in this area and no use of deicing salt is expected at this bridge.

Micro
Referencing the criteria from Article 2.2.1.2, Table 28 shows a summary of micro exposure zones identified for the main members of the bridge. These identified exposure conditions are graphically shown in Figure 16 and Figure 17.

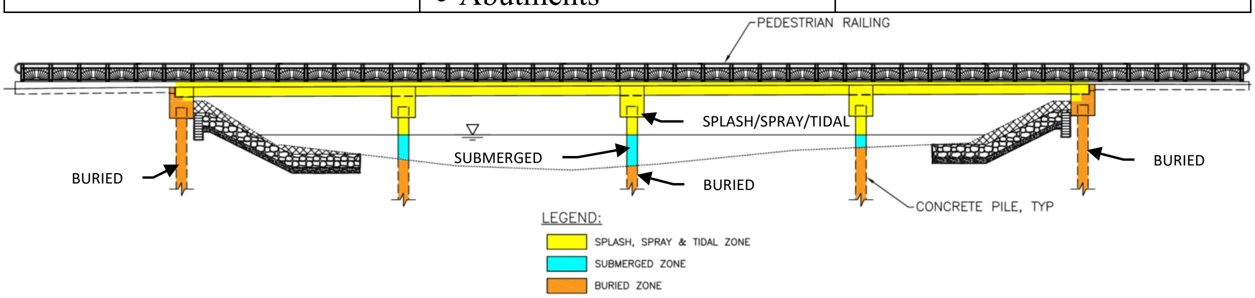
The deck soffit and top of the girders are buried in concrete while the top of the deck and soffit of the girders are at the tidal zone. For these elements, the most stringent Exposure Zone that governs the design is the tidal zone and the design for these two elements are determined accordingly.

At the abutments, the exposed surfaces in the vicinity of the girder seat are in the tidal zone while the remaining surfaces and piles are in the buried zone. The exposed top portions of the bents are in the tidal zone, the middle permanently submerged portions are in a marine-submerged zone, and the bottom regions of the piles are in a buried zone.

The barriers and railing are also within the tidal zone.

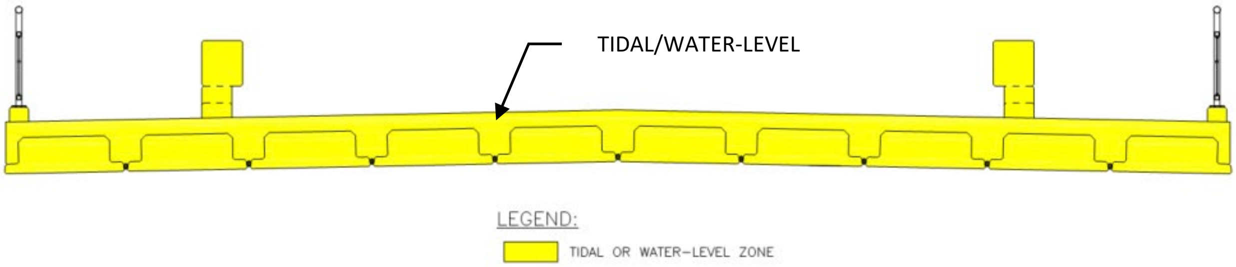
Table 28. Summary of micro exposure zones.

Exposure Zone	Elements	Description
Tidal or Water- Level zone	• Top of Deck, • Soffit of Girders, • Bents, • Piles, • Barriers • Railing • Approaches	Not permanently submerged in the water, subject to wet-dry cycles, 20 ft above the tidal zone
Marine-Submerged	• Piles	Permanently submerged in sea water
Buried	• Piles • Abutments	Permanently buried in soil



Source: adapted from Pease et al. (2019)

Figure 16. Typical exposure conditions of the bridge elements.



Source: adapted from Pease et al. (2019)

Figure 17. Typical exposure conditions on the superstructure.

Exposure Classes

Based on Guide Specification Table 2.2.2.1-1, a list of concrete exposure classes and potential deterioration mechanisms is summarized in Table 29.

Table 29. Concrete exposure classes.

Exposure Zone	Elements	Exposure conditions	AAR	DEF(1)	Sulfate	Carbonation-induced Corrosion	Chloride-induced Corrosion	Abrasion	Preservice Cracking
Tidal or Water-Level zone	• Top of Deck • Soffit of Girders • Bent • Piles • Barriers	Alternating wetting and drying with sea water. High Cl^- in the water. SO_4 in the sea water. Atmospheric O_2 and CO_2, in marine environment	x(2)	x	S1	C-M1	C-M3	x	x
Marine-Submerged	• Piles	Exposure to sea water. High Cl^- in the sea water. SO_4 in sea water.	-	-	S1	-	C-M2	-	-
Buried	• Piles • Bent	Chlorides and sulfates exposure in soil.	x	x	S1	-	C-B	-	-

(1) Applicable to mass concrete with least dimension > 48 inches
(2) x means the deterioration mechanism is applicable to the exposure zone and associated elements, however, no specific Exposure Class is described in the nonbinding Guide Specification with respect to that mechanism.

Design of Concrete Components

For this project, the Owner confirmed only fly ash and silica fume is available as supplementary cementitious material. Black steel is used everywhere except the piles where stainless steel is used.

Material Parameters

Initial design assumptions for each concrete component are summarized in Table 30 based on the *AASHTO LRFD Bridge Design Specifications* (2017b) (23 CFR 625.4(d)(1)(v)) Table 5.10.1-1. To ensure the desired service life can be met and provide protection at the material-level, the concrete mix, concrete cover and reinforcement type are investigated in this section according to Article 4.2 of the nonbinding Guide Specification. In this section, the sufficiency of the preliminary design assumptions will be investigated to determine if the design needs to be improved to meet the desired service life.

It should be noted that the cover values specified by the *AASHTO LRFD Bridge Design Specifications* (2017b) (23 CFR 625.4(d)(1)(v)) are defined as the minimum cover to main bars and cover to ties and stirrups may be 0.5 inch less. However, cover thickness as per the Guide Specification is the distance from the concrete surface to the outermost steel reinforcement, whether that is the main bars or ties and stirrups.

Table 30. Initial design assumptions for concrete cover and reinforcement type.

Elements	Cover to the main bar (in)	Reinforcement Type
Top of the Deck	2.5	Black steel
Soffit of the deck	2	Black steel
Top of the Girders	1	Black steel
Soffit of the Girders	2	Black steel
Bents	4.5	Black steel
Piles	3	Stainless Steel
Barriers	2	Black steel

General

Based on the Guide Specification Table 4.2.1-1, the maximum W/CM for all concrete members is 0.45 since the structure is over saltwater.

Sulfate Attack

Table 31 summarizes the minimum design for exposure class S1 based on the Guide Specification Tables 4.2.1-1 and 4.2.3-1.

Table 31. Cementitious materials parameters for concrete exposed to sulfate attack.

Exposure Class	Exposure zone/condition	Element	Maximum W/CM	Minimum f'c (ksi)	Min Cementitious Materials Type	Minimum class of concrete
S1	Moderate sulfate exposure; sea water exposure	• Deck, • Girders, • Bent, • Piles, • Barriers	0.45	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS	A*

*As specified in the *AASHTO LRFD Bridge Construction Specifications* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv))

Alkali Aggregate Reaction

All concrete mixes are subject to a risk of alkali-aggregate reaction. As stated in Article C4.1.4 of the Guide Specification, there are two types of AAR: alkali silica reaction and alkali-carbonate reaction. Provisions from AASHTO R-80 should be implemented to mitigate the risk of AAR and should be implemented into the project construction specifications.

Abrasion

As stated in Guide Specification Article 4.1.6, the abrasion resistance of concrete is related to the hardness of the concrete. A high strength concrete (Class A) is used for this project to improve the abrasion resistance of the deck. In addition, the minimum cover is increased by 0.25 inch as indicated by Article 4.2.4.2.2 for the top of the deck which is susceptible to abrasion.

Preservice Cracking

Referencing Article 4.1.7, the use of fly ash as a pozzolanic admixtures in the concrete mix will reduce the heat of hydration and reduce the potential for early age cracking. Proper curing techniques will be applied according to the *AASHTO LRFD Bridge Construction Specifications* (2017a) (23 CFR 625.4(d)(1)(iv)).

Corrosion

Referencing Guide Specification Table 4.2.4.2.2-1 and the available cementitious materials, design options for each component in terms of concrete type, reinforcement class, and cover are listed in Table 32.

An OPCFA mix contains ordinary portland cement with 20-50 percent fly ash type F by mass of total cementitious materials. In addition to fly ash, an OPCFA+SF mix contains 5-8 percent silica fume by mass of cementitious materials.

Table 32. Cover associated with the corrosion exposure class.

Exposure Class	Exposure zone/condition	Element	Concrete Type	Cover (in)	Reinforcement Type
C-B	Buried	• Bent	OPCFA+SF, OPCFA OPC Any	1 2.5 1	A A D
C-M2	Marine-Submerged	• Piles	OPCFA+SF OPCFA Any	2.5 3 1	A A D
C-M3	Marine-Tidal or Splash/Spray	• Top of Deck • Soffit of Girder • Bent • Pile • Barrier	OPCFA+SF OPCFA Any	3 3.5 1	A A D
C-NA1	Interior exposure	• Soffit of Deck • Top of Girders	OPCFA+SF, OPCFA Any	1 1	A D

Where possible, the same concrete type should be selected for similar elements for practicality purposes and ease of construction. For example, rather than specifying two different concrete types for the deck, the cover can be adjusted, and one concrete type should be specified.

Concrete type, reinforcement class, and cover values based on the governing corrosion Exposure Class are given in Table 33.

Table 33. Selected Exposure Class C designs.

Component	Controlling Exposure Class	Concrete Type	Reinforcement Class	Cover (in)
Deck	C-M3	OPCFA	A	3.5 (Top) 1 (Bottom)
Girders	C-M3	OPCFA	A	1 (Top) 3.5 (Bottom)
Bent	C-M3	OPCFA	A	3.5
Piles	C-M3	OPCFA	D	1

Summary

The final mix design is summarized in Table 34. The maximum water-cementitious materials ratio is reduced to 0.40 in order to meet the assumption in the Article 4.2.2.2.2 for determining the concrete cover.

Table 34. Summary of design values for each element.

Component	Concrete Type	Concrete Class	Reinforcement Class	Cover (in)	W/CM	Minimum f'_c (ksi)	Min Cementitious Materials Type
Deck	OPCFA	A	A	3.5 (Top) 1 (Bottom)	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Girders	OPCFA	A	A	1 (Top) 3.5 (Bottom)	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Bent	OPCFA	A	A	3.5	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Piles	OPCFA	A	D	1	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS

Consideration of Other Items

Mass Concrete

Based on Guide Specification Article C4.2.4.2.1c, a project specific Thermal Control Plan is suggested for all mass concrete in this example (components with least dimension of 48 in for this example). The plan will include provisions to limit the maximum temperature of curing concrete to 158°F. in accordance with ACI 201.2R (2016).

Design of Foundations

Concrete Piles

Referencing Article 6.2.2.1, a Protection Index of 2 is selected in accordance with Table 6.2.2.1-1 to design the foundations since the piles are located in an aggressive environment with chloride content greater than 500 ppm and exposure to wet/dry cycles. Based on Guide Specification Table 6.2.2.2.2-1, a dense impermeable concrete will be used in addition to using corrosion resistant reinforcement (stainless steel).

Design of Renewable Elements

Bearings

Elastomeric bearing pads support the girders at each bent. Referencing Article 7.2.2, these bearings must be properly designed in accordance with the *AASHTO LRFD Bridge Design Specifications*, (2017b) (23 CFR 625.4(d)(i)(v)). Conservatively, it is assumed their service life is 40 years. Bearing maintenance activities will address regular cleaning, inspection and evaluation of the bearing systems, housings, and anchors. This will be detailed in the Maintenance and Inspection Manual.

Expansion Joints

There are two finger joints on the deck at the abutments. Referencing Table 7.1.1-1 of the Guide Specification, the expected service life of these joints varies between 20 to 50 years. It is assumed that the joints will be replaced every 25 years. Regular maintenance including clearing of debris in the joints will be specified in the Maintenance and Inspection Manual.

Barriers

An OPCFA concrete mix with a maximum W/CM ratio of 0.4 is used for the barriers to provide necessary durability for marine-tidal exposure. However, the concrete covers specified by the Guide Specification Article 4.2.4.2.2 are not meant to be used for the barriers with shorter target service life. Therefore, the standard cover specified by the Owner is used while the concrete mix parameters are more stringent and results in a higher quality concrete with longer expected service life.

Referencing Article 7.3, the barriers should be continuously monitored for damages and promptly repaired.

Pedestrian Handrailing

Aluminum pedestrian handrailing is used for this bridge. It is assumed the railing gets replaced at the time of the barrier's replacement. Referencing to Article 7.3, railing and barriers should be continuously monitored for damages and promptly repaired.

Design Summary

A summary of the service life design for each element is provided in Table 35.

Table 35. Summary of the service life design by component.

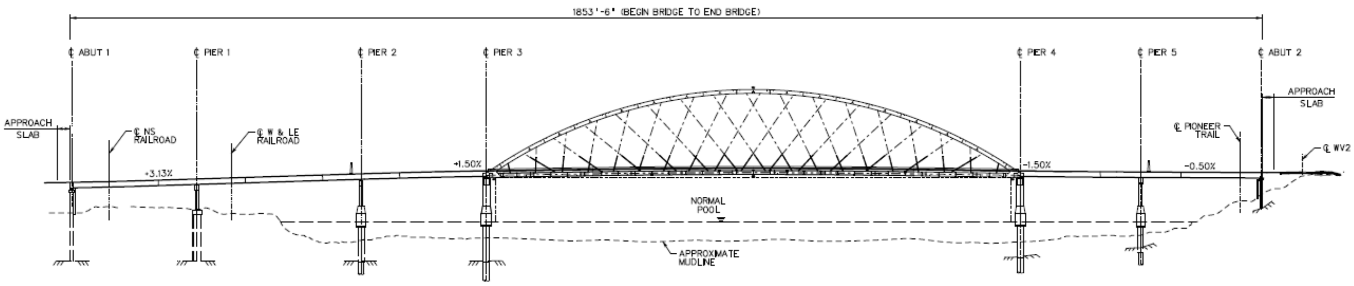
Component	Target Service Life (years)	Concrete Type	Reinforcement Class	Cover (in)	W/C	Minimum f'c (ksi)	Min Cementitious Materials Type
Deck	75	OPCFA*	A	3.75* (Top) 1 (Bottom)	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Girders	75	OPCFA	A	1 (Top) 3.5 (Bottom)	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Bent	75	OPCFA	A	3.5	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Piles	75	OPCFA	D	1	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Barriers	40	OPCFA	A	2	0.40	4	ASTM C150 Type II, ASTM C595 Type IP, IS, or IT with (MS) designation, ASTM C1157 Type MS
Bearings	40	OPCFA	-	NA	NA	NA	NA
Expansion Joint	25	NA	NA	NA	NA	NA	NA
Bearings	40	NA	NA	NA	NA	NA	NA
Handrailing	40	NA	NA	NA	NA	NA	NA

*0.25 is added to account for abrasion

EXAMPLE 3: SIGNATURE ARCH BRIDGE

Introduction

The signature tied arch bridge in this example is in West Virginia. Figure 18 shows the general elevation of the bridge.



Source: FHWA

Figure 18. Bridge general elevation.

Location

The bridge has the following location information:

- Crossing the Ohio river
- Urban environment with periods of snow and freeze-thaw cycles.
- Use of deicing salts but only as necessary during the winter months.

Components and Features

The structure consists of two approach units and a main span. There are four piers in the river, one pier on land adjacent to a railway, and two semi-integral abutments. There are expansion joints at Piers 3 and 4.

The main span unit is a basket-handle, network tied arch structure with a 830 ft span length. The superstructure depth is 10 ft (steel girders with a cast-in-place (CIP) deck). The deck width is constant along the main span and measures 59ft-6 ½ inches to the exterior faces of the barriers. The composite cast-in-place concrete deck within the tied arch span is post-tensioned transversely and longitudinally. The arch ribs feature a circular profile in their inclined plane. The hangers radiate at a constant angle about the center of the arch, forming a “bicycle wheel” pattern.

The approach structures consist of steel plate girders with intermediate cross frames and a composite CIP concrete deck with a total length of 645 ft. on the West side and 375 ft. on the East side.

The foundations of Piers 2, 3, 4, and 5 consist of a single row of concrete drilled shafts with permanent steel casings, with diameters of 8 ft or 10 ft socketed into bedrock, while Pier 1 is founded on steel piles. Abutment units are semi-integral founded on steel piles.

Environmental Data

Historical data show that the sulfate content of the Ohio River water near the bridge site has a mean concentration of 71 mg/L with values ranging from 24 to 130 mg/L (ppm). Sulfate content in the soil at the bridge location varies from 0 to 240 ppm. Field data show that the chloride concentration in the Ohio River and its tributaries averaged 30 mg/L. The chloride content is relatively small and the water is considered fresh water and would not qualify as brackish water (more than 0.05percent chloride). A maximum chloride content in the soil of 60 ppm (0.0060 percent) was measured. The resistivity of the soil was measured to be lower than 2000 ohm-cm.

Classification

The bridge is classified, in terms of target service life and the environment, according to Section 2 of the Guide Specification.

Target Service Life

The target service life for nonreplaceable and renewable bridge components is determined using Guide Specification Article 2.1.1.

Nonreplaceable

The structure is classified as Maximum according to Article 2.1.1 and Table 2.1.1-1 of the Guide Specification since it is a major signature bridge. Therefore, the Best level of qualitative practice is assigned to nonreplaceable components as summarized in Table 36.

Table 36. Service life category selection for nonreplaceable components.

Category	Component	Bridge Description	Level of Qualitative Practice
Maximum	• Foundations • Abutments • Piers • Pier caps • Deck • Superstructure	Signature Bridge	Best

Renewable

The target service lives for renewable components would be selected by the Engineer. As described in Article 3.2.2 of the Guide Specification, the design should account for the interaction of bridge components, replacement of renewable elements, and the service life of individual components in relation to the total bridge service life.

Based on the ranges provided in Table 7.1.1-1 of the Guide Specification, a service life range between 8 and 50 years appears to be achievable for joints. The minimum service life for the expansion joints in this example is 30 years as per the Project Criteria.

Certain bearing types have the potential to last at least 75 years based on Article 7.2.2 of the Guide Specification. The minimum service life for the bearings is 50 years as per the Project Criteria.

A target service life of 25 years is desired for paint and coating systems by the Project Criteria.

Barriers are made of concrete to have a 60-year service life. They are designed to be replaceable.

Hangers should have a 60-year service life. They are designed to be replaceable.

Target service lives for renewable components are summarized in Table 37.

Table 37. Target service lives for renewable components.

Replaceable Components	Minimum Service Life (years)
Bridge bearings	50
Expansion joints	30
Bridge barriers	60
Paint/Coating System	25
Hangers	60

Environmental Classification

Exposure Zones

<u>Macro</u>
Based on the provided location and climate features and referencing Article 2.2.1.1 of the Guide Specification, the structure is placed in Industrial/Moderate zone. The bridge is not located in a heavy deicing salt environment due to milder winters but will have occasional exposure to airborne salts and deicing salt runoff. The area has moderate to high humidity.

<u>Micro</u>
Referencing the criteria from Article 2.2.1.2 of the Guide Specification, applicable micro exposure zones for each component are listed in Table 38.

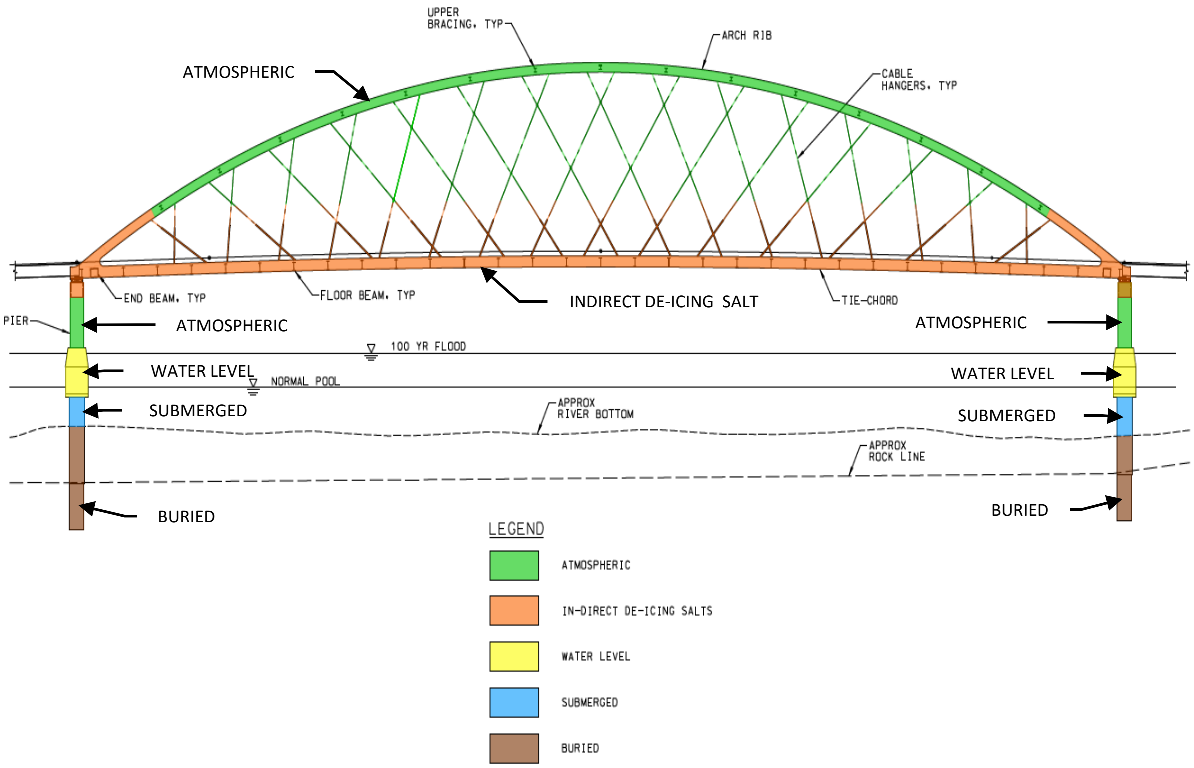
Note that since the concrete drilled shafts are covered with permanent steel casing the concrete is not exposed to the environment. Concrete in rock sockets is similarly not exposed to the environment.

At the abutment, it is assumed that any leakage of potentially chloride contaminated water will not collected on the backside of the abutment due to proper drainage detailing. In addition, the concrete mix design for the abutment will be controlled by the buried exposure condition which considers exposure to chlorides.

Table 38. Exposure zones.

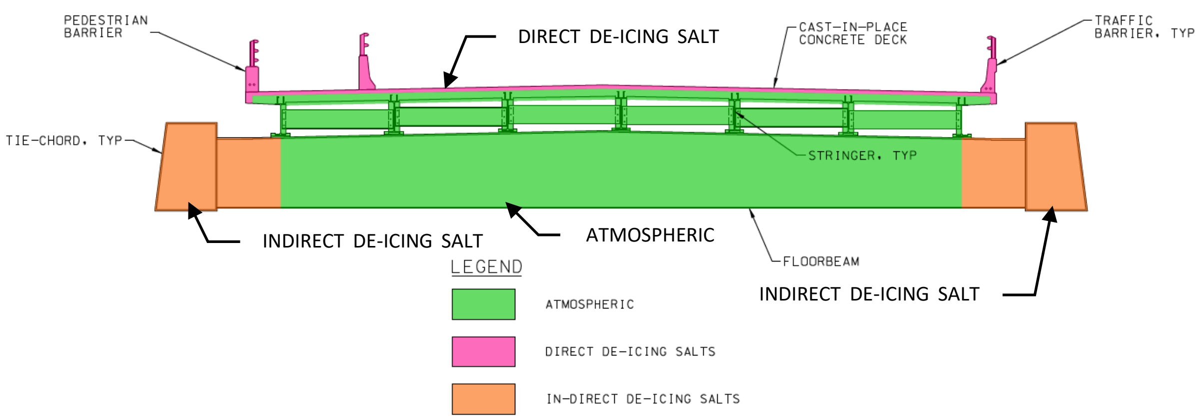
Exposure Zone	Component	Description
Buried	• Abutment • Wing wall • Steel casing for drilled shafts • Steel piles	• Permanently buried in soil.
Atmospheric	• Arch Rib • Upper part of cable hanger • Upper bracing • Stringer • Girder (approach spans and main span, except under expansion joints) • Floor beam (except under expansion joints) • Intermediate cross-frame (approach spans) • Deck soffit • Pier column • Pier cap • Abutment • Wing wall	• Not exposed to soil, water, or deicing salts. • Solid traffic barriers and drip grooves are considered to prevent indirect deicing exposure to exterior girders.
Indirect Deicing	• Crossbeam • Top of pier column • Arch rib • Lower part of cable hangers • End beam • End beam diaphragm • Stringer • End cross-frame • Girder under expansion joint • Floor beam under expansion joint • Tie chord	• Indirect exposure to deicing salts due to runoff and joint failure. • Indirect exposure to deicing salts within the roadway splash/spray zone.
Direct Deicing	• Top surface of deck and approach slab • Traffic barrier • Pedestrian barrier • Deck fascia and soffit to drip groove detail	• Directly exposed to deicing salts.
Water Level	• Pedestal • Pile caps	• Not permanently submerged in water, subject to wet-dry cycles.
Submerged	• Steel casing for drilled shafts	• Permanently submerged in water

Figure 19 through Figure 24 visually show the exposure zones assigned to the bridge components.



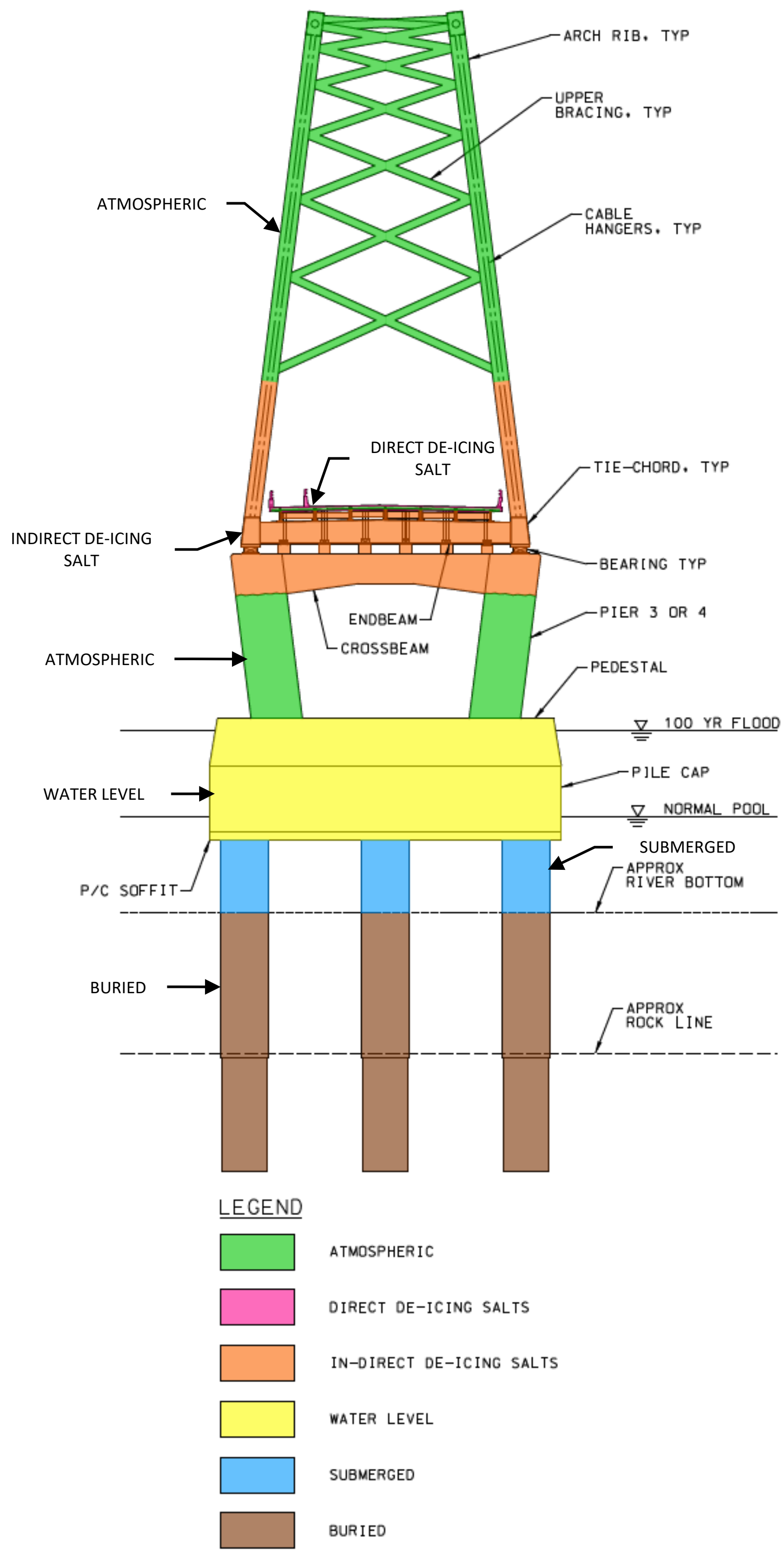
Source: FHWA

Figure 19. Exposure zones shown in bridge general elevation view.



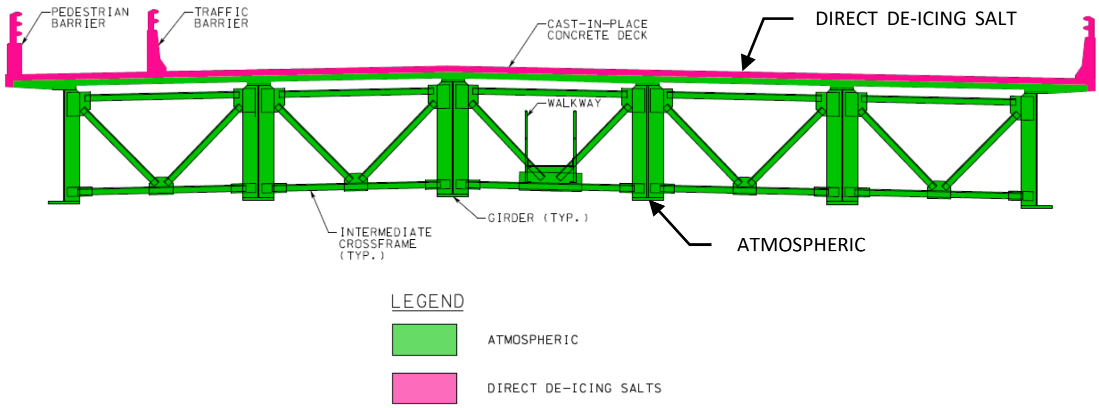
Source: FHWA

Figure 20. Exposure zones for superstructure components at tie chord.



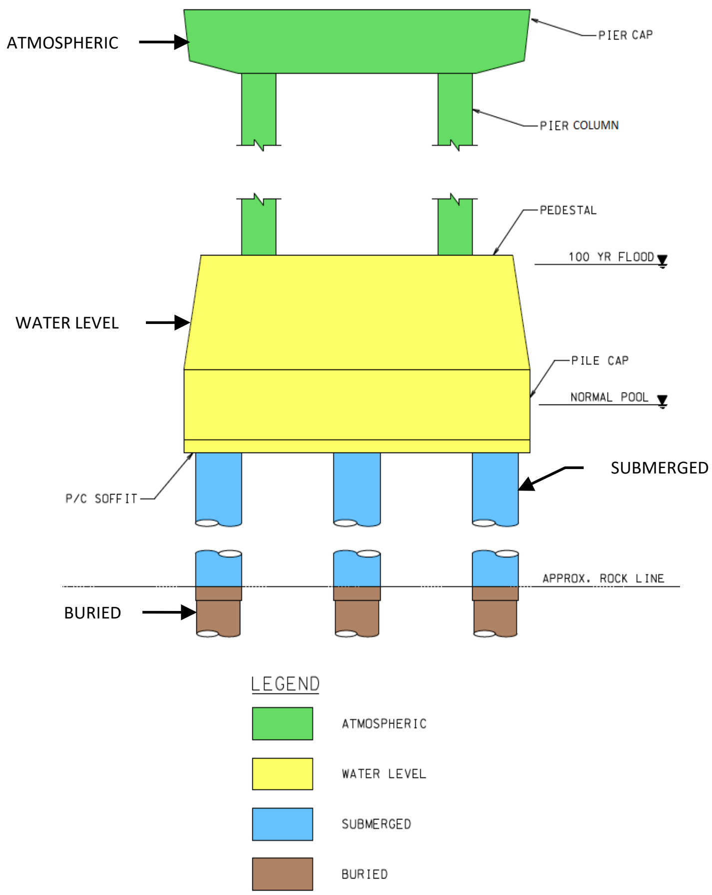
Source: FHWA

Figure 21. Typical exposure zones for Piers 3 and 4 at the expansion joints.



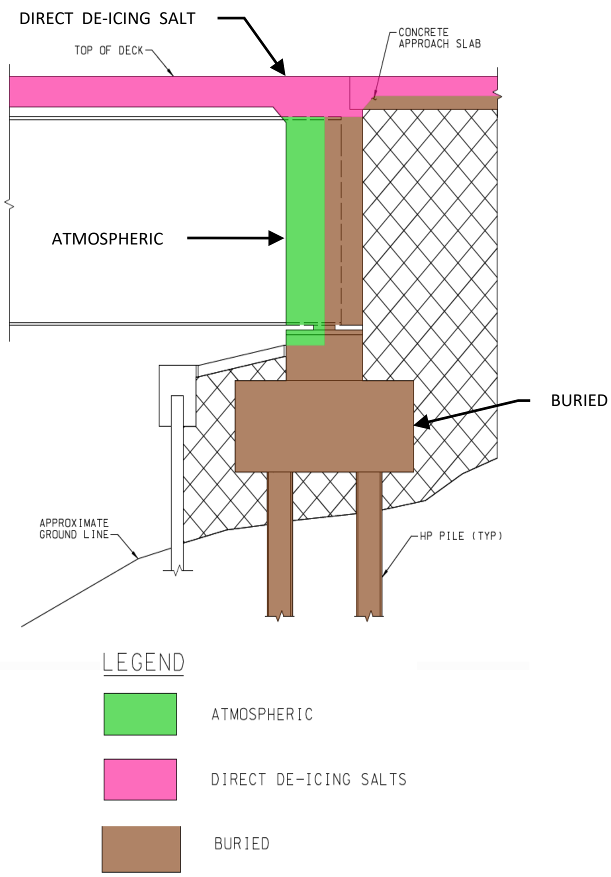
Source: FHWA

Figure 22. Exposure zones for approach spans components.



Source: FHWA

Figure 23. Typical exposure zones for pier located in water (Piers 2 and 5).



Source: FHWA

Figure 24. Typical exposure zones at the semi-integral abutments.

Exposure Classes

Exposure classes are assigned following Guide Specification Table 2.2.2.1-1 and 2.2.2.2-1 for concrete and steel components based on previously determined exposure zones and applicable deterioration mechanisms. Assigned exposure classes for concrete components are shown in Table 39, and those for steel components are given in Table 40.

Abrasion is not considered a significant deterioration mechanism for the piers at the water level given that nearby bridges do not exhibit section loss due to abrasion by water. A new combined exposure zone named C-NA2 + W1 was created for water level exposure as this exposure is not considered in the Guide Specification.

Table 39. Concrete exposure class assignments.

Exposure Zone	Components	Exposure Conditions	Corrosion	Freeze-thaw	AAR	Sulfate Attack	Abrasion	Preservice Cracking[1]
Buried	• Abutment • Wing wall • Footing	• Freeze-thaw above the frost line • Sulfates in the soil	C-B	FT1	X	S0	-	-
Atmospheric	• Pier column • Pier cap • Abutment • Wing wall • Deck soffit[2)]	• Some airborne chlorides and carbon dioxide • Freeze-thaw potential	C-D1	FT2	X	-	-	X
Indirect Deicing	• Crossbeam • Top of pier column	• Cycles of wetting and drying due to roadway splash/ spray and runoff • Freeze-thaw potential with frequent exposure to water and chlorides	C-D2	FT3	X	-	-	X
Direct Deicing	• Top surface of deck and approach slab • Traffic barrier • Pedestrian barrier • Deck fascia and soffit to drip groove detail	• Direct exposure to deicing salts • Freeze-thaw potential with frequent exposure to water and chlorides	C-D3	FT3	X	-	X (top of the deck only)	X
Water Level	• Pedestal • Pile caps	• Alternating wetting and drying with river water.	C-NA2 + W1	FT2	X	S0	-	X

[1]"X" indicates applicable deterioration mechanism but without a specific Exposure Class in the nonbinding Guide Specification for that deterioration mechanism
[2]FT1 could be used for deck soffit; however, this freeze thaw exposure class will not govern the deck concrete mix design.

Table 40. Steel exposure class assignments.

Exposure Zone	Components	Exposure Conditions	Corrosion	Fatigue
Atmospheric	• Arch rib • Upper part of cable hanger • Upper bracing • Stringer • Girder (approach spans and main span, except under expansion joints) • Floor beam (except under expansion joints) • Intermediate cross-frame	• Some airborne chlorides and carbon dioxide	C3	F
Buried	• Steel casing • Steel piles	• Moisture, sulfate, and chloride in soil	See Design of Foundations Section	-
Indirect Deicing	• Arch rib • Lower part of cable hangers • End beam • End beam diaphragm • Stringer • End cross-frame • Girder under expansion joint • Floor beam under expansion joint • Tie chord	• High salinity area	C4	F
Submerged	• Steel casing for drilled shafts	• Sulfate and chloride in water	See Design of Foundations Section	-

Design of Concrete Components

Materials

The design for the concrete components is determined according to Section 4 of the Guide Specification. Based on material availability and Owner standard mixes, the following concrete classes can be specified for the project, as defined in Section 8 of the *AASHTO LRFD Bridge Construction Specifications* (2017a) (23 CFR 625.4(d)(1)(iv)):

- An ordinary portland cement (OPC) mix with air entrainment (AE) for general structural applications: Class A(AE)
- An OPC mix with AE typically reserved for substructures: Class B(AE)
- An OPC mix with AE for thin elements: Class C(AE)
- A high-performance concrete (HPC) mix with 20-35percent fly ash or 36-65 percent ground granulated blast furnace slag (GGBS) replacement of portland cement: Class A(HPC)

The minimum material properties of available mix are shown in Table 41.

Table 41. Available concrete mixes.

Concrete Class[1]	W/CM	Min f'_c (ksi)	Size of Coarse Aggregate[3]	Air Content Range (%)	SCM Substitutions (% mass of total cementitious material)
A(AE)	0.45	4.0	1.0 in. to No. 4	4.5 – 7.5	-
B(AE)	0.55	2.5	2.0 in. to 1.0 in. and 1.0 in. to No. 4	3.5 – 6.5	-
C(AE)	0.45	4.0	0.5 in. to No. 4	5.5 – 8.5	-
A(HPC)	0.45	5.0[2]	1.5 in. to 0.25 in.	5.0 – 8.0[2]	20-35% Class F Fly Ash or 36-65% GGBS

[1]As defined in the *AASHTO LRFD Bridge Construction Specifications* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv))
[2]As specified in the contract documents or Owner standard specifications
[3]Per AASHTO M43 (ASTM D448). Not a Federal requirement

General

The general materials are determined according to Guide Specification Article 4.2.1. Discussion for each relevant exposure class is contained in later sections.

Sulfate Attack

Sulfate contents reported above in the water and soil are not sufficient to cause a chemical attack to the concrete. Based on Guide Specification Table 2.2.2.1-1 and the amount of sulfate in soil and water, sulfate concentration in contact with concrete is represented by the S0 class and injurious sulfate attack is not a concern.

Freeze-Thaw Attack and Elements in Contact with Water

Using Guide Specification Table 4.2.1-1, Exposure Class FT and W concrete material parameters and the governing condition for each element are summarized in Table 42. Drilled shaft concrete is not included as the concrete is not in contact with the environment. The deck soffit is not included as this side of the deck is sheltered and this exposure condition will not govern the design. Article 4.2.4.2.2 of the Guide Specification assumes a 0.40 maximum W/CM which therefore will control over the freeze/thaw parameters in Table 42.

Target air content and corresponding available concrete classes are determined according to Guide Specification Table 4.2.2-1 and are presented in Table 43.

Table 42. Concrete material parameters for freeze-thaw attack and in contact with water.

Exposure Class	Component	Max W/CM	Min f'_c (ksi)	Additional Parameters	Min Concrete Class[1]
FT1	• Footing	0.55	3.5	See Table 43	B (AE)
FT2	• Wing Walls • Abutments • Pile Cap • Pedestal • Pier Cap	0.45	4.5	See Table 43	A (AE)
FT3	• Pier Columns • Deck and Approach Slab • Traffic and Pedestrian Barriers • Crossbeam	0.40	5.0	See Table 43	A (HPC)

[1]As defined in the *AASHTO LRFD Bridge Construction Specifications* (2017a) (incorporated by reference at 23 CFR 625.4(d)(1)(iv))

Table 43. Additional parameters for freeze-thaw attack.

Nominal Maximum Aggregate Size (in)	Target Air Content for FT1 (%)	Available Concrete Class	Target Air Content for FT2, FT3 (%)	Available Concrete Class
0.5	5.5	C(AE)	7.0	C(AE)
1.0	4.5	A(AE)	6.0	A(AE)
1.5	4.5	A(HPC)	5.5	A(HPC)

Alkali Aggregate Reaction

From Table 39, all concrete components are potentially susceptible to alkali aggregate reaction (AAR). Based on Guide Specification Article 4.1.4, general approaches to reduce the risk for AAR that are applicable to this bridge include the use of low-alkali cement, nonreactive aggregates, and SCMs. Additional provisions from AASHTO R80 should be followed to mitigate AAR risks.

Abrasion

The top surface of the deck is susceptible to abrasion, particularly for this deicing environment where studded tires and plows may be used. Per Article 4.1.6 of the Guide Specification, the abrasion resistance is proportional to the concrete strength. The concrete to be used for the deck has a specified minimum compressive strength of 5 ksi which should provide sufficient abrasion resistance. In addition, the minimum cover for the top of the deck is increased by 1/4-inch to be consistent with Article 4.2.4.2.2 of the Guide Specification.

Preservice Cracking

For components susceptible to preservice cracking listed in Table 39, the implementation of proper curing techniques is a primary method to reduce cracking potential, as specified in Article 4.1.7 of the Guide Specification. The Owner should request the concrete to be wet cured for 7 days minimum, which should reduce the risk of early age cracking.

Corrosion

Initial design assumptions for each concrete component are summarized in Table 44 based on common practice and Owner standard specifications. The covers used in the structural design and shown on RFC drawings can be greater than the cover used in the durability analysis or have a smaller construction tolerance.

Table 44. Summary of concrete covers.

Structural Element	Description	Cover	
		Mean (in)	Construction Tolerance (in)
Drilled shaft concrete	With permanent steel casings	3	-[(1)]
Pier 3 and Pier 4	Pile caps and pedestals	3	0.50
	Column and crossbeam	3	0.50
Pier 1	Footing	3	0.50
	Column and pier cap	2	0.50
Pier 2 and Pier 5	Pile cap and pedestal	3	0.50
	Column and pier cap	2	0.50
Abutment and wing walls	Pile cap and diaphragm	3	0.50
Cast-In-Place Deck	Top of the deck	2.75	0.25
	Deck fascia up to the drip groove	2.5	0.25
	Underside of the deck	1.5	0.25
Approach Slab	Top of the approach slab	2.75	0.25
	Underside of the approach slab	3	0.50
Barrier		2	0.25

[1]No construction tolerance for the drilled shafts encased by the steel casing and rock socket.

Referencing Guide Specification Table 4.2.4.2.2-1 and the available concrete mixes from Table 41, design options for each component in terms of concrete type, reinforcement class, and cover are listed in Table 45. Note that improved reinforcement classes were not considered for this example. Article 4.2.4.2.2 of the Guide Specification assumes a 0.40 maximum W/CM. Corrosion of barriers is discussed in the Design of Renewable Elements.

Guide Specification Table 4.2.4.2.2-1 does not explicitly address elements at the water level or those exposed to fresh water such as Piers 2, 3, 4 and 5 pile cap and pedestal. Therefore, provisions included in NCHRP 269 (Murphy et al., 2020) are used to determine the parameters for these elements following a full probabilistic approach based on fib Bulletin 34 (fib 2006). Table 46 summarizes the service life modeling input parameters and Table 47 shows the design options based on the analysis. It is noted that inputs in Table 46 are generally equivalent to C-NA2 exposure except the convection zone is applied in recognition of the wetting and drying in the water level zone (thus creating a C-NA2 + W1 exposure classification). Use of these resources is not required by Federal law.

Table 45. Exposure Class C design options.

Component	Exposure Zone	Exposure Class	Concrete Type	Reinforcement Class	Cover (in)
Pier 3 and Pier 4: column and crossbeam	Indirect deicing salts	C-D2	OPCFA+SF OPCFA GGBS+SF GGBS	A A A A	2.5 2.5 3.0 4.0
Pier 1: footing	Buried	C-B	OPCFA+SF OPCFA GGBS+SF GGBS OPC	A A A A A	1.0 1.0 1.5 1.5 3.5
Pier 1: column and pier cap	Atmospheric	C-D1	OPCFA+SF GGBS GGBS+SF OPCFA OPC	A A A A A	1.5 1.5 1.5 2.0 2.5
Pier 2 and Pier 5: column and pier cap. Abutment and wing walls	Atmospheric	C-D1	OPCFA+SF GGBS OPCFA GGBS+SF OPC	A A A A A	1.5 1.5 2.0 1.5 2.5
Abutment: Pile cap and diaphragm	Buried	C-B	OPCFA+SF OPCFA GGBS+SF GGBS OPC	A A A A A	1.0 1.0 1.5 1.5 3.5
Top of the deck	Direct deicing salts	C-D3	OPCFA+SF OPCFA GGBS+SF	A A A	2.5 3.0 3.5
Deck fascia up to the drip groove	Direct deicing salts	C-D3	OPCFA+SF OPCFA GGBS+SF	A A A	2.5 3.0 3.0
Underside of the deck	Atmospheric	C-D1	OPCFA+SF GGBS+SF GGBS OPCFA OPC	A A A A A	1.0 1.0 1.0 1.5 1.5
Top of the approach slab	Direct deicing salts	C-D3	OPCFA+SF OPCFA GGBS+SF	A A A	2.5 3.0 3.5
Underside of the approach slab	Buried	C-B	OPCFA+SF OPCFA GGBS+SF GGBS OPC	A A A A A	1.0 1.0 1.5 1.5 3.5

Table 46. Input parameters for service life modeling for elements at water level.

Exposure Zone	Temperature (F)			Chloride Surface Concentration			Depth of the Convection Zone, Δx				
	dist.	Mean μ (F)	Stdev σ (F)	dist.	Mean μ (%)	Stdev σ (%)	dist.	Mean μ (in)	Stdev σ (in)	a (in)	b (in)
Water Level	Normal	64	3.6	Lognormal	1	0.5	Beta	0.35	0.22	0	2

Table 47. Exposure Class C water level exposure zone design options.

Component	Exposure Zone	Exposure Class	Concrete Type	Reinforcement Class	Cover (in)
Pier 2, 3, 4 and 5: Pile cap and pedestal	Water Level	C-NA2 + W1	OPCFA+SF OPCFA GGBS+SF GGBS	A A A A	2 3 3 4

Based on the design life category of Maximum for this bridge, it is decided to use supplementary cementitious materials in all concrete mixes to enhance the quality of concrete and maximize the service life. Therefore, OPC only mixes will not be used for this project. Hence, additional concrete mixes with up to 8 percent silica fume were added to Table 45 as additional design options.

It is impractical to vary the concrete type within one element (e.g., pier columns, deck). Rather, the concrete type was selected to meet the most stringent Exposure Class for each element. It is decided to use slag mixes for all substructure elements as it is locally available and would reduce the cost of materials transportation. However, for the deck and approaches, higher concrete cover is suggested if slag mixes are used which cannot be accommodated due to constraints imposed by the structural design. Therefore, the use of fly ash mixes with reduced cover are necessary for the deck and approaches.

The parameters for concrete type, reinforcement class, and cover values based on the governing corrosion Exposure Class are given in Table 48.

Table 48. Selected Exposure Class C designs.

Component	Elements	Controlling Exposure Class	Concrete Type	Reinforcement Class	Cover (in)
Pier 3 and Pier 4	Column and crossbeam	C-D2	GGBS+SF	A	3.0
	Pile Cap and pedestal	C-D2	GGBS+SF	A	3.0
Pier 1	Footing	C-B	GGBS	A	1.5
	Column and pier cap	C-D1	GGBS	A	1.5
Pier 2 and Pier 5	Pile Cap and Pedestal	C-NA2 + W1	GGBS+SF	A	3.0
	Column and pier cap	C-D1	GGBS+SF	A	1.5
Abutment	Pile cap and diaphragm	C-B	GGBS	A	1.5
Deck	Top of the deck	C-D3	OPCFA	A	3.0
	Deck fascia up to the drip groove	C-D3	OPCFA	A	3.0
	Underside of the deck	C-D1	OPCFA	A	1.5
Approach slab	Top of the approach slab	C-D3	OPCFA	A	3.0
	Underside of approach slab	C-B	OPCFA	A	1.0

Summary

The final mix designs are shown in Table 49. The final concrete type, concrete class, reinforcement class, and cover to be used for each component are listed in Table 50. The following adjustments were made as a result of the design:

- Class A(HPC) will be used to meet the OPCFA, GGBS, and GGBS+SF concrete types from Table 48, with a separate mix design for each. The W/CM of Class A(HPC) will be reduced to 0.40 to meet the parameters for Exposure Class FT3 and Article 4.2.4.2.2 of the Guide Specification for concrete cover.

- Some covers are smaller than typically used and to ensure satisfactory concrete quality, the NT Build 492 should be performed to validate the chloride migration coefficient of the concrete and that it meets the intent of the specification.

Table 49. Final concrete mixes.

Concrete Class[1]	W/CM	Min f'_c (ksi)	Size of Coarse Aggregate[3]	Target Air Content (%)	SCM Substitutions (% mass of total cementitious material)
A(HPC)	0.40[4]	5.0[2]	1.5 in. to 0.25 in.	5.5[2]	20-25% Class F Fly Ash or 35-65% Slag and/or up to 8% Silica Fume

[1]As defined in the *AASHTO LRFD Bridge Construction Specifications* (2017a) (23 CFR 625.4(d)(1)(iv))
[2]As specified in the contract documents or Owner standard specifications
[3]Per AASHTO M43 (ASTM D448). Not a Federal requirement
[4]Adjusted to meet design parameters

Table 50. Concrete material design summary.

Component	Elements	Concrete Type	Concrete Class	Reinforcement Class	Cover (in)
Pier 3 and Pier 4	Column and crossbeam	GGBS+SF	A (HPC)	A	3.0
	Pile Cap and pedestal	GGBS+SF	A (HPC)	A	3.0
Pier 1	Footing	GGBS	A (HPC)	A	1.5
	Column and pier cap	GGBS	A (HPC)	A	1.5
Pier 2 and Pier 5	Pile Cap and Pedestal	GGBS+SF	A (HPC)	A	3.0
	Column and pier cap	GGBS+SF	A (HPC)	A	1.5
Abutment	Pile cap and diaphragm	GGBS	A (HPC)	A	1.5
Deck	Top of the deck	OPCFA	A (HPC)	A	3.0
	Deck fascia up to the drip groove	OPCFA	A (HPC)	A	3.0
	Underside of the deck	OPCFA	A (HPC)	A	1.5
Approach slab	Top of the approach slab	OPCFA	A (HPC)	A	3.0
	Underside of approach slab	OPCFA	A (HPC)	A	3.0

Other Protection Strategies

In addition to the material design determined previously, other strategies to consider for the protection of the concrete elements include sealers, proper detailing, and deck-specific strategies.

Surface Sealers

Due to the use of deicing salt application anticipated for this bridge, there is a high chance of chloride exposure for many of the exposed concrete surfaces. Surface sealers, as described in Article 4.3.1 of the Guide Specification, are an added layer of protection when used in conjunction with the previously designed material properties and cover parameters.

A state-approved coating-type sealer is available for use on the project. Therefore, this sealer will be specified to be applied to certain concrete surfaces. In general, these surfaces will be those assigned Exposure Class C-D2 or worse.

Detailing

Detailing measures will be employed to enhance durability, with a focus on drainage and conveying moisture off of the structure, as outlined in Article 4.3.2 of the Guide Specification. This will be accomplished by:

- Providing sufficient scupper and downspout capacity at the abutments and piers, particularly at the lowest elevation abutment.
- Sloping the abutment seats, which are directly under joints (See Guide Specification Figure C4.3.2-5).
- Detailing drip grooves in the deck overhang to limit moisture from reaching the girder top flanges (see Guide Specification Figure C4.3.2-5).

- Specifying water stops at all cold joints (see Guide Specification Figures C4.3.2-8 and C4.3.2-9).

Deck-Specific Design

The deck will be exposed to a combination of freeze-thaw cycles and application of deicing salts. As noted in the nonbinding Guide Specification Article C4.2.2, the fly ash content will be limited and testing for scaling resistance will be implemented. This means the following additional parameters apply to deck concrete:

- Scaling test as per ASTM C672 should be performed.

- The fly ash content should be limited to 25 percent by mass of total cementitious materials to limit risk of scaling damage.

- The NT Build 492 test should be performed to validate the chloride migration coefficient of the concrete and that it meets the intent of the specification.

A secondary protection measure is to use a deck-specific sealer. The governing standard specifications include a penetrating sealer on all new bridge decks. Therefore, an approved silane or siloxane penetrating sealer will be specified in the contract documents.

In addition, the Guide Specification Article 4.3.3.3 concrete curing could be considered for this project. Specific to this design:

- Make the cure period longer than the 7 days (preferably 14 days).

- Deck loads should be restricted until sufficient strength has been verified.

- Local environmental and weather conditions should be accounted for during construction, and appropriate limitations should be adhered to (e.g., temperature, wind).

Design of Steel Components

The steel components, which include the steel casing for drilled shafts, driven steel H-piles, girder, stringer, floor beam, intermediate cross-frame, and arch rib, should be designed according to Section 5 of the Guide Specification. As previously indicated, the applicable deterioration mechanisms include corrosion and fatigue.

Corrosion Protection

Article 5.2.1 of the Guide Specification should be used to develop corrosion protection strategies for superstructure. The protection strategy for the buried elements/piles will be discussed in the Design of Foundations section.

Steel Type Selection

For a Maximum service life category and considering the exposure classes summarized in Table 40 for steel elements, the suggested protection strategy based on Table 5.2.1-1 of the Guide Specification is summarized in Table 51. Each component is designed per its most critical exposure condition and the controlling protection strategy is provided in the table.

Table 51. Controlling steel corrosion protection strategy selection.

Component	Service Life Category	Exposure Class	Corrosion Protection Strategy
• Upper bracing • Intermediate cross-frame • Hangers (upper)	Maximum	C3	Uncoated Weathering Grade
• Arch rib • End beam • End beam diaphragm • Stringer • Plinth • End cross-frame • Girder • Floor beam • Tie chord • Hangers (lower)	Maximum	C4	Coated Non-Weathering Grade

The governing exposure class for the superstructure elements is C4 and so coated non-weathering grade will be used (except for intermediate cross-frame where they could remain as uncoated weathering steel). Based on the considerations listed in Article 5.2.1.2 of the Guide Specification, uncoated weathering steel does not seem to be suitable for all superstructure elements where there is a continuing presence of chloride and moisture holding debris that prevents alternate wet-dry cycles at some locations. For simplicity, all superstructure elements will be coated non-weathering steel.

Unless otherwise dictated by the structural design, the commonly available ASTM A709 Grade 50 will be specified for the superstructure elements.

Coating System Design

Options for coating systems include galvanization, metallization, and paint. The Owner should specify a paint system to be used for non-weathering steel superstructure elements. Governing standard specifications include options for a three-coat system with a zinc primer. The available paint systems are listed in Table 52. Using the information provided in Guide Specification Appendix A5 (Helsel and Lanterman, 2018), the practical maintenance time, *P*, for these paint systems is estimated based on exposure class.

Table 52. Available paint coating systems and their practical maintenance times.

Type	Coating System	Surface Preparation	Number of Coats	Min. DFT (mils)	C3 (years)	C4[1] (years)	C5M
Epoxy Zinc	Epoxy Zinc/ Epoxy/ Polyurethane	Blast	3	10	20	17	14
Organic Zinc	Organic Zinc/ Epoxy/ Polyurethane	Blast	3	10	18	15	12

[1]Practical maintenance time for C4 is interpolated linearly.

The chosen paint system will not last for the entire service life of the components they are applied to; therefore, the systems will have to be maintained at regular intervals. The maintenance intervals will vary depending on the exposure class and the type of coating system. Table 53 shows the maintenance painting sequence options for each combination of exposure class from Table 40 and available coating type from Table 52. The timing for initial touch-up, maintenance repaint (*M*), and full repaint (*R*) given in Table 53 follows the information provided in Appendix A5 of the Guide Specification. Based on Table 37, the service life for painting is 25 years for this project which is the time until full repaint as indicated by project agreement.

Table 53. Maintenance painting sequence options.

Exposure Class	Coating Type	Initial Touch-Up (Year)[1]	Maintenance Repaint, *M* (Year)[2]	Full Repaint, *R* (Year)[3]
C4	Epoxy Zinc	17	26	35
C4	Organic Zinc	15	23	31

[1]Equal to Practical Maintenance Time, P
[2]Equal to $1.5P$
[3]Equal to $M+0.5P$

There are several factors that should be considered when selecting a coating system, including service life, cost, and difficulty of application and re-application. The choice of coating system should be based on a life cycle cost analysis (LCCA), as outline in Section 8 of the Guide Specification. For brevity, a LCCA is not performed for this example; Epoxy zinc was used because the time to initial touch-up is longer than the organic zinc system.

Using the information from Table 53, the timing of painting for superstructure elements and exposure class is calculated as shown in Table 54. The timing should be designed such that the total life of the coating system (including repainting multiple times) exceeds the target service life of the component that the coating system is applied to, which is approximately 150 years considering the Maximum target service life category for nonreplaceable components.

A copy of Table 54 should be included in the Service Life Design Manual for bridge management to timely schedule maintenance painting activities.

Table 54. Design maintenance painting timing for superstructure elements, exposure class C4, epoxy zinc coating type.

Action	Timing
Initial Painting (Year)	0
P_1 (Year)	17
M_1 (Year)	26
R_1 (Year)	35
P_2 (Year)	52
M_2 (Year)	61
R_2 (Year)	70
P_3 (Year)	87
M_3 (Year)	96
R_3 (Year)	105
P_4 (Year)	122
M_4 (Year)	131
R_4 (Year)	140

Fatigue Design

The designer must refer to Section 6 of the *AASHTO LRFD Bridge Design Specifications* (2017b) (23 CFR 625.4(d)(1)(v)) to determine the design for each fatigue detail.

Detailing

Detailing measures will be employed to enhance durability, with a focus on drainage and conveying moisture off of the structure, as outlined in Article 4.3.2 of the Guide Specification. This will be accomplished in the example by:

- Detailing to avoid debris traps, such as limiting the number of transverse stiffeners if possible and by providing adequate clip sizes and clearance where they are needed (See Guide Specification Figure C5.3.1.1-1).

- Providing drip bars on the girder bottom flanges near support locations such that moisture is shed from the girder prior to reaching the piers and abutments (see Guide Specification Figure C5.3.1.1-2).

- Extending any downspouts below the girder bottom flange (see Guide Specification Figure C5.3.1.1-8).

Design of Foundations

Drilled shafts with permanent steel casings and reinforced concrete infills will be used at Pier 2, 3, 4 and 5. Driven steel H-piles are used at the abutments and Pier 1.

H-Piles

As previously assigned in Table 40, corrosion is a concern for the H-piles. Although the sulfate and chloride concentration in the soil is low, the resistivity of the soil was measured to be lower than 2000 ohm-cm. Therefore, a P_I of 1 was assigned to the local deterioration environment from Tables 6.2.2.1-1 of the Guide Specification.

From Table 6.2.2.2.1-1 of the Guide Specification, a $P_I = 1$ asks the Owner to specify a protection based on local past experience. In this project, the Owner would specify that the steel area of the piles be increased. Using information provided in C6.2.2.2.1 of the Guide Specification and conservatively assuming the 95 percent maximum probable corrosion rate for piles in the Buried Zone in Marine environment, the corrosion loss is calculated as 0.3 inches (i.e., 0.0020 inches/year times 150 years). This thickness should be added to each side of each pile and a new section size should be selected accordingly.

Drilled Shafts Steel Casing

A similar approach that was used for the H-Piles is applied to the steel casings. A corrosion allowance of 0.3 inches is added to the exterior face only as concrete will be poured inside the casing.

Design of Renewable Elements

Expansion Joints

Expansion joint design minimums are outlined in the Project Criteria:

- The minimum service life of expansion joints should be 30 years.
- All joints should be sealed from bridge deck surface drainage. Open-type joints that accept bridge deck surface drainage, such as finger joints, should include drainage troughs to collect runoff and protect superstructure components, including bearing assemblies.
- Bridge expansion joints should be located at substructure units only. Joints are not allowed at bridge end abutments.
- No expansion joints or stress relief joints are allowed between the beginning and end of the continuous arch.

Based on Table 7.1.1-1 of Guide Specification, finger expansion joints are expected to have service life between 20 to 50 years. Therefore, with adequate inspection and maintenance, it is reasonable to assume 30 years of service life for these joints. They will be provided at Pier 3, 4, and at the end of the approach slabs of Abutment 1 and Abutment 2.

Use of the information in section 7.1.4 of Guide Specification is suggested for regular maintenance and inspection of all joints during the service life.

Bearings

Design the bearings for service life in accordance with Article 7.2 of the Guide Specification. The bridge Owner commonly uses steel-reinforced elastomeric pads for this bridge type. Based on Table 7.2.1-1 and Article 7.2.2 of the Guide Specification, these types of bearings have a low cost, typically involve low maintenance, and have the potential to last 75 to 100 years. Therefore, steel-reinforced elastomeric bearings will be specified for both the fixed and expansion bearings. The use of these bearings must be confirmed with a structural design in accordance with the AASHTO LRFD Bridge Design Specifications (2017b) (23 CFR 625.4(d)(1)(v)). Since the bearings will not last the entire service life of the structure, the superstructure should be designed and detailed to accommodate bearing replacement.

Article 7.2.3 of the Guide Specification indicates that the steel components of the bearings should be protected from corrosion using a type of coating. Anchor bolts for the bearings are commonly available galvanized; therefore, galvanization will be specified for all other steel components of the bearings (e.g., sole plates) for consistency.

Proper and routine maintenance activities, including inspection, cleaning, and evaluation of the bearing assemblies, should be specified in the Service Life Design Manual.

Barriers (Traffic and Pedestrian)

An OPCFA concrete mix with a maximum W/CM ratio of 0.4 is used for the barriers to provide necessary durability to direct deicing salt exposure. Per Table 42, a minimum concrete class of A(HPC) and 5.0 ksi minimum compressive strength should be specified. The concrete covers specified by the Guide Specification Article 4.2.4.2.2 are not meant to be used for the barriers with a shorter target service life (60 years for this example). Therefore, the standard cover specified by the Owner, 2 in of cover and 0.25 in of construction tolerance, is used. Conversely, the concrete mix parameters are more stringent and, therefore, are expected to result in a higher quality concrete with a longer service life.

Testing for resistance to scaling for the barrier concrete mix design should be considered to enhance the durability of the concrete.

Referencing Article 7.3, the barriers should be continuously monitored for damages and promptly repaired.

Hangers

A proprietary corrosion protection system will be provided by the supplier.

Design Summary

Summaries of the service life design for the concrete and steel components are provided in Table 50 and Table 51 (including the use of coating systems presented in Table 53). Foundation design will rely on a corrosion allowance where 0.3 inches is added to the face exposed to soil.

Design approaches for renewable elements include:

- Expansion Joints: Finger expansion joints with an expected service life of 30 years or more.

- Bearings: Steel-reinforced elastomeric pads will be used.

- Barriers: An OPCFA concrete mix with a maximum W/CM ratio of 0.4 combined with the standard concrete cover thickness.

- Hangers: A proprietary system will be provided by the supplier.

DISCUSSION ON DIFFERENCES BETWEEN EXAMPLES USING A FULL PROBABILISTIC APPROACH AND THE AASHTO GUIDE SPECIFICATION (I.E. SHRP2 VS. NCHRP 12-108)

The AASHTO Guide Specification and the SHRP2 R19A project share many similarities: they use a deemed-to-satisfy approach or avoidance approach for many deterioration mechanisms for steel and concrete components. In addition, they share a common key reference: the fib Bulletin 34. The service life design method implemented in the AASHTO Guide Specification uses principles from the fib Bulletin 34. The implementation part of the SHRP2 R19A project was focused on the application of fib Bulletin 34 including the use of a full probabilistic approach for calculations of corrosion initiation time for reinforced concrete. A significant component of the SHRP2 R19A project was to produce calculation tools for the time to corrosion modeling and input data specific to the United States. Use of these resources is not required under Federal law or regulations.

A major difference between the AASHTO Guide Specification and SHRP2 R19A is that the AASHTO Guide Specification methodology proposes calibrated design tables for concrete components where the designer does not need to perform modeling to determine time to corrosion initiation for certain combinations of environmental exposure conditions, concrete mixes, reinforcement chloride thresholds, and target service lives. This means that the bridge designer does not need to go through the time to corrosion modeling as proposed by the SHRP2 R19A project. The design examples shown in this section illustrate the simpler process developed by the AASHTO Guide Specification for service life design.

The development of calibrated design tables inherently means that the design process has to be simplified. The AASHTO Guide Specification tables for concrete cover are based on a number of assumptions to bound the model for time to corrosion. For example, a key assumption is the ambient temperature: in order to keep the cover table to a manageable size, only two temperatures were used for the calculation of time to corrosion, one for deicing salt environments and one for marine environments (hot environments are considered through a footnote in the table). In general, this means that the designer has less flexibility and is limited to the design options within the table, and the assumptions used to develop the table. This is an expected cost for using a simplified approach.

The worked design examples presented in this document are very similar to the worked design examples presented in the SHRP2 R19A Service Life Design of Bridges Summary Guide. In fact, the first example of each document use the same bridge: a conventional multiple span composite-deck highway overpass bridge located in the Northeast region of the US subjected to heavy de-icing salt use and freeze-thaw cycles and having a target service life of 75 years for non-replaceable components.

A comparison of both designs, one following the full probabilistic approach as given in SHRP2 R19A and one following the simplified AASHTO Guide Specification approach shows that the two approaches produce different results. The primary mitigation method for corrosion of concrete components is to provide sufficient concrete cover with adequate concrete quality, characterized by the chloride migration coefficient (D_{RCM}). A low chloride migration coefficient generally means a higher concrete quality. The AASHTO Guide Specification made the assumption that the chloride migration coefficient was fixed for each concrete type considered (i.e. D_{RCM} is not a value that the designer can modify without switching to a different concrete type). This limits the flexibility of the designer but also greatly simplifies the design. The maximum allowable migration coefficients used by the AASHTO Guide Specification for OPC and OPCFA concrete types are as follows:

OPC: 12.5×10^{-12} m^2/s

OPCFA: 8.0×10^{-12} m^2/s

Table 55 presents for each component the combination of concrete cover and concrete quality desired. Both examples assumed similar concrete types (OPC and OPCFA) and similar exposure conditions, and many underlying assumptions are also similar making a comparison between the two examples possible. One can see that, for the same component, the combination of concrete quality and concrete covers differ between the two methods: generally, SHRP2 shows a higher cover and a higher allowable chloride migration coefficient (lower concrete quality). This means that the design would allow more lenient concrete design at the cost of higher covers. There are some exceptions to this observation:

- The piles have the same cover for both SHRP2 and AASHTO, however the maximum allowable migration coefficient in SHRP2 is higher than AASHTO. This shows that the flexibility provided by the full probabilistic approach allowed the designer to remove some conservatism that is embedded in the AASHTO tables.

- The bottom pier column has similar covers, but the AASHTO allowable migration coefficients are greater than that calculated by SHRP2. The main factor is the exposure conditions assumed: SHRP2 assumed the equivalent to a CD-3 exposure (a 3 percent surface chloride concentration) whereas AASHTO assumed a higher exposure of CD-4 (4 percent surface chloride). Another factor that contributes to a stricter migration coefficient for SHRP2 is that the construction tolerance for cover (+/- 1 in) is double the value in AASHTO (+/- 0.5 in).

The comparison of this design example shows that both methods lead to a durable design, albeit using slightly different strategies: SHRP2 opted for larger covers and more lenient concrete designs while AASHTO opted for smaller covers and stricter concrete design parameters. The difference however is not significant as both options are constructible. In fact, the majority of the cover values are within 0.5 in of each other when comparing the examples. The AASHTO method is simpler to implement than SHRP2 and both lead to a design meeting the target service life.

Table 55. Comparison of AASHTO and SHRP2 design example.

Component	Concrete Type	AASHTO			SHRP2		
		Exposure Class	D_{RCM} (x10^{-12} m^2/s)	Cover (in)	Exposure Class	D_{RCM} (x10^{-12} m^2/s)	Cover (in)
Tangent Piles	OPC	C-B	12.5	2.5	Buried / Atmospheric	15	2.5
Pile Caps	OPC	C-B	12.5	2.5	Buried	15	3
Abutments	OPCFA	C-D2	8.0	2.5	Indirect De-icing	10	3
Pier Columns (bottom)	OPCFA	C-D4	8.0	3	Direct De-icing	7	3
Pier Caps	OPCFA	C-D2	8.0	2.5	Indirect De-icing	10	3
Deck (Top)	OPCFA	C-D4	8.0	3.25	Direct De-icing	7	2.75
Deck (Soffit)	OPCFA	C-D1	8.0	1.5	Atmospheric	Not calculated.	1.75

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Publication No. FHWA-HIF-22-052