Service Life Design for Bridges

Summary Guide

April 29, 2019

The second Strategic Highway Research Program (SHRP2) is a national partnership of key transportation organizations: the Federal Highway Administration, American Association of State Highway and Transportation Officials, and Transportation Research Board. Together, these partners are deploying products that will help the transportation community enhance the productivity, boost the efficiency, increase the safety, and improve the reliability of the nation's highway system.

This report is a work product of the SHRP2 Solution, Service Life Design for Bridges (R19A). The product leads are Raj Ailaney, Federal Highway Administration, and Patricia Bush, American Association of State Highway and Transportation Officials. This report was authored by Brad J. Pease, Ph.D., Mia Lund, Ph.D., Fatemeh Alapour, and Anne-Marie Langlois, P.Eng., P.E., from COWI North America Ltd., with support from Mike Bartholomew, P.E., from Jacobs Engineering Group Inc.

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Abbreviations

°C	degrees Celsius
AAR	alkali-aggregate reactions
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
CO_2	carbon dioxide
COV	Coefficient of Variation
CSA	CSA Group
DEF	Delayed Ettringite Formation
FHWA	Federal Highway Administration
fib	International Federation for Structural Concrete
ISO	International Standards Organization
kg/m^2	Kilograms per square meters
LCCA	Life Cycle Cost Analysis
LRFD	Load and Resistance Factor Design
m^2/s	square meter per second
mm	millimeter(s)
mm^2/yr	square millimeters per year
MPa	megapascal
NACE	National Association of Corrosion Engineers
P3	Public-Private Partnership
psf	per square foot
psi	pounds per square inch
QA	quality assurance
QC	quality control
R19A	Project number of the FHWA and AASHTO project *Service Life Design for Bridges*

RFP	Request for Proposal
SHRP2	Strategic Highway Research Program
w/c ratio	water/cement ratio

1.0 Introduction

The assessment of service life design or durability of building materials is increasingly applied to major bridge and other infrastructure projects, to ensure the condition of the bridge components and materials are kept above a minimum acceptable level throughout the lifespan of the structure. New major structures are being designed for service lives of 100 years or more, with the Osman Gazi (Izmit Bay) Bridge in Turkey, the Gov. Mario M. Cuomo (Tappan Zee) Bridge, and Abraham Lincoln Bridges in the United States as three recent examples of major infrastructure projects with a 100-year service life requirement for non-replaceable components.

North American structural design codes do not explicitly consider the durability and service life of structures. While 75 years is a common 'design life' considered in design codes (for example, American Association of State Highway and Transportation Officials [AASHTO] Load and Resistance Factor Design [LRFD] [1].) as described herein, design life and service life are not synonymous. Further, the traditional prescriptive requirements included in design codes do not consider project-specific situations and may result in an unreliable long-term durability performance of structures.

The Federal Highway Administration (FHWA), in partnership with AASHTO, is responsible for implementing *Service Life Design for Bridges* (also referred to as R19A) through the second Strategic Highway Research Program (SHRP2). Multiple tools, products, and training materials aimed at practitioners and state bridge engineers were developed as part of the implementation effort and can be found at the AASHTO website at:

http://shrp2.transportation.org/Pages/ServiceLifeDesignforBridges.aspx

This Summary Guide is a key component of R19A as it provides a framework for the integration of service life design with typical design processes for new bridges, starting from the inclusion of appropriate provisions and requirements in Requests for Proposal (RFPs), through design considerations, construction requirements, and handover and operation of bridges with enhanced service life requirements. The Summary Guide serves as guidance to specifying asset owners, design engineers conducting the service life design, contractors constructing a bridge with an extended service life requirement, and operators charged with inspecting and maintaining bridges, and aims at aligning the understanding of Service Life Design amongst these parties.

The organization of the Summary Guide follows the framework introduced in Section 2.0 and covers the following topics:

- Section 3.0 – Guidance on Requiring Service Life Design
- Section 4.0 – Guidance on Implementing Service Life Design during Structural Design

- Section 5.0 – Guidance on Implementing Service Life Design during Construction and Documentation of the As-Built Service Life
- Section 6.0 – Guidance for the Operation (Monitoring, Inspection, and Maintenance)

Within these sections, target advice and information are provided to these individual parties. The document presents the current practices for developing technically feasible service life requirements and should aid in aligning expectations of all parties involved in a service life design.

The Summary Guide presents effective durability strategies and deterioration mitigation measures available and provides a useful shelf reference, compiling pertinent test data collected by Implementation Assistance Program Agencies through the SHRP2 R19A project, as well as examples of documents that are composed of key steps in the Service Life Design process. This includes RFP examples in Appendix A, supplementary material information compiled as part of the SHRP2 R19A project in Appendices B and C, example service life design reports in Appendix D, an example Supplementary Concrete Specification in Appendix E, and an example Birth Certificate in Appendix F.

2.0 Framework for Service Life Design

2.1 Objective

The objective of service life design is to complete a rational assessment of the potential deterioration mechanisms affecting structural elements to achieve a target service life duration. This approach goes beyond sole reliance on code-based prescriptive requirements that may not sufficiently consider project-specific environmental exposure conditions and/or performance requirements. To best implement service life design in a new bridge project, aspects of the service life design process should be appropriately implemented from project outset (that is, during the planning and inception phase) and during all project stages. The framework described in Section 2.3 provides a step-by-step process flowchart and provides typical service life-related inputs to the project during the planning, design, construction, and operation phases.

2.2 Terminology

Table 2-1 provides definitions to key terms related to service life design, while Table 2-2 provides definitions and examples of various maintenance-related terms. Definitions for the following terms commonly differ between design codes and literature sources with many terms used interchangeably. The following lists are introduced to aid in comprehension and to maintain consistency within this Summary Guide. Several definitions of 'service life' exist from various literature sources, as provided in Table 2-1. As explained in Sections 2.3 and 2.6, the definition of service life can vary on a project-specific basis and selecting a definition of service life is a key aspect of the service life design process. In general, maintenance-related terms and definitions are based on definitions provided in the FHWA Bridge Preservation Guide [2]. The term 'restoration' is outside the scope of service life design and its definition, from American Concrete Institute (ACI) CT13 [3], is included for information only.

Table 2-1: Service Life-related Terminology.

Term	Definition
Design life* [1]	Period of time on which the statistical derivation of transient loads is based: 75 years for AASHTO LRFD [1].
Design service life [4]	The assumed period for which a structure or a part of it is to be used for its intended purpose.
Service life [1]	The period of time that the bridge is expected to be in operation.
Service life [2]	The period for which a component, element, or bridge provides the desired function and remains in service with appropriate preservation activities.
Service life [3]	Service life is the period after installation and placement during which all the properties exceed the minimum acceptable values when routinely maintained.
Service life [5],[6]	Service life is the time during which the structure performs its design function without unforeseen maintenance or repair.
Service life design	A rational engineering design approach to resist deterioration mechanisms caused by the prevailing environmental actions, by considering the durability performance of materials and component details, with the goal of achieving a desired lifetime (service life).
Limit states [4]	States beyond which the structure no longer fulfills the relevant design. criteria.
Design criteria [4]	Quantitative formulations that describe, for each limit state, the conditions to be fulfilled.

* Load and resistance factors for structural design in AASHTO LRFD were calibrated to the 75-year period, but deterioration of the structure over time was not explicitly considered. Unfortunately, there is no direct relationship between the AASHTO LRFD definitions for Service Life and Design Life.

Table 2-2: Maintenance-related Terminology.

Term	Definition	Informative Examples
Condition-based Maintenance [2]	Condition-based maintenance activities are performed on bridge components or elements in response to known defects. Condition-based maintenance improves the condition of that portion of the element but may or may not result in an increase in the component condition rating. Replacement of replaceable components (see Section 2.4) is part of condition-based maintenance.	• Repair/replacement of crash barrier(s) and steel coatings, in response to reaching end of service life • Repair or replacement of strip seals in expansion joints • Bearing restoration (cleaning, lubrication, re-setting, replacement) • Spot/zone/full painting of steel elements
Cyclical Maintenance [2]	Maintenance activities performed on pre-determined intervals that aim to preserve and delay deterioration of bridge elements or component conditions.	• Cleaning of dirt, debris, bird droppings from structural steel • Flush drains • Cleaning of expansion joints and bearings • Periodic application of grease, lubricants where appropriate • Repair of concrete surfaces due to mechanical damages or local surface spalls • Apply sealers to concrete surfaces

Table 2-2: Maintenance-related Terminology.

Term	Definition	Informative Examples
Maintenance (based on [2])	Work performed to maintain the condition of the bridge or respond to specific conditions or events that restore the bridge to a functional state of operations. Maintenance is a general term and encompasses routine, preventative, cyclical, and condition-based maintenance activities as well as preservation, rehabilitation and replacement. Rehabilitation, prior to reaching the required service life, is commonly referred to as 'major maintenance'.	See other definitions for examples of individual maintenance types
Preservation [2]	Actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; and keep bridges in good or fair condition. Preservation actions include cyclical maintenance, condition-based maintenance, and preventative maintenance, the latter being a form of preservation intended to extend service life.	See definitions of cyclical maintenance, condition-based maintenance, and preventative maintenance for informative examples
Preventative Maintenance [2]	A proactive and cost-effective approach to extend the service life of a bridge. Preventative maintenance is typically not foreseen as part of the service life design of new structures.	• Application of cathodic protection to arrest ongoing reinforcement corrosion • Electrochemical chloride extraction from concrete • Mechanical strengthening of the bridge or component

Table 2-2: Maintenance-related Terminology.

Term	Definition	Informative Examples
Rehabilitation [2]	Major work required to restore the structural integrity of a bridge, as well as work necessary to correct major safety defects.	• Partial or complete deck replacement • Superstructure replacement • Substructure/culvert strengthening or partial/full replacement
Replacement [2]	Total replacement of an existing bridge with a new facility in the same general traffic corridor.	Not applicable for service life design of new structures
Routine Maintenance [2]	Work performed in reaction to an event, season, or activities that are done for short-term operational need that do not have preservation value. This work requires regular reoccurring attention.	• Trash, litter, and dead animal removal • Snow removal/application of salt/de-icing chemicals • Graffiti removal • Hazardous material removal • Asphalt patch with no membrane on concrete deck • Accident damage to bridge and its appurtenances • Storm damage

2.3 Step-by-Step Service Life Design Process

Figure 2-1 provides a step-by-step process flowchart of the service life design process. The individual steps in the flowchart can be more generally divided into the following project phases as also indicated in Figure 2-1:

- Inception and initial planning
- Design
- Construction
- Operation

During project inception and initial planning, basic targets for service life requirements shall be established. A definition for service life and the duration of the service life for replaceable and non-replaceable components should be established at this stage. As explained in Section 2.4, not all components of the structure need have the same service life requirement because components identified as replaceable may have a shorter service life requirement than the overall structure. The following two steps in the flowchart (materials selection and assess environmental exposure conditions) may be considered as either part of the inception and initial planning phase, the design phase, or both as shown in Figure 2-1.

The design phase consists of selection of durability strategies to address the individual applicable deterioration mechanisms and verification of the service life through establishment of requirements to materials, design, and construction. Additional introduction of the basic durability strategies presented in Figure 2-1 is given in Section 2.5.

The construction phase consists of development of Construction Specifications and plans for inspection and maintenance of the structure. Final inspection of the executed works marks the end of the construction phase. During the operation phase, inspection, monitoring, and maintenance works are carried out in accordance with the associated developed plans.

Similar to the structural design process, different parties can potentially complete the different steps of the service life design. For example, the asset owners can potentially complete the entire design phase presented in Figure 2-1, or the design phase could be completed by the designer/engineer.

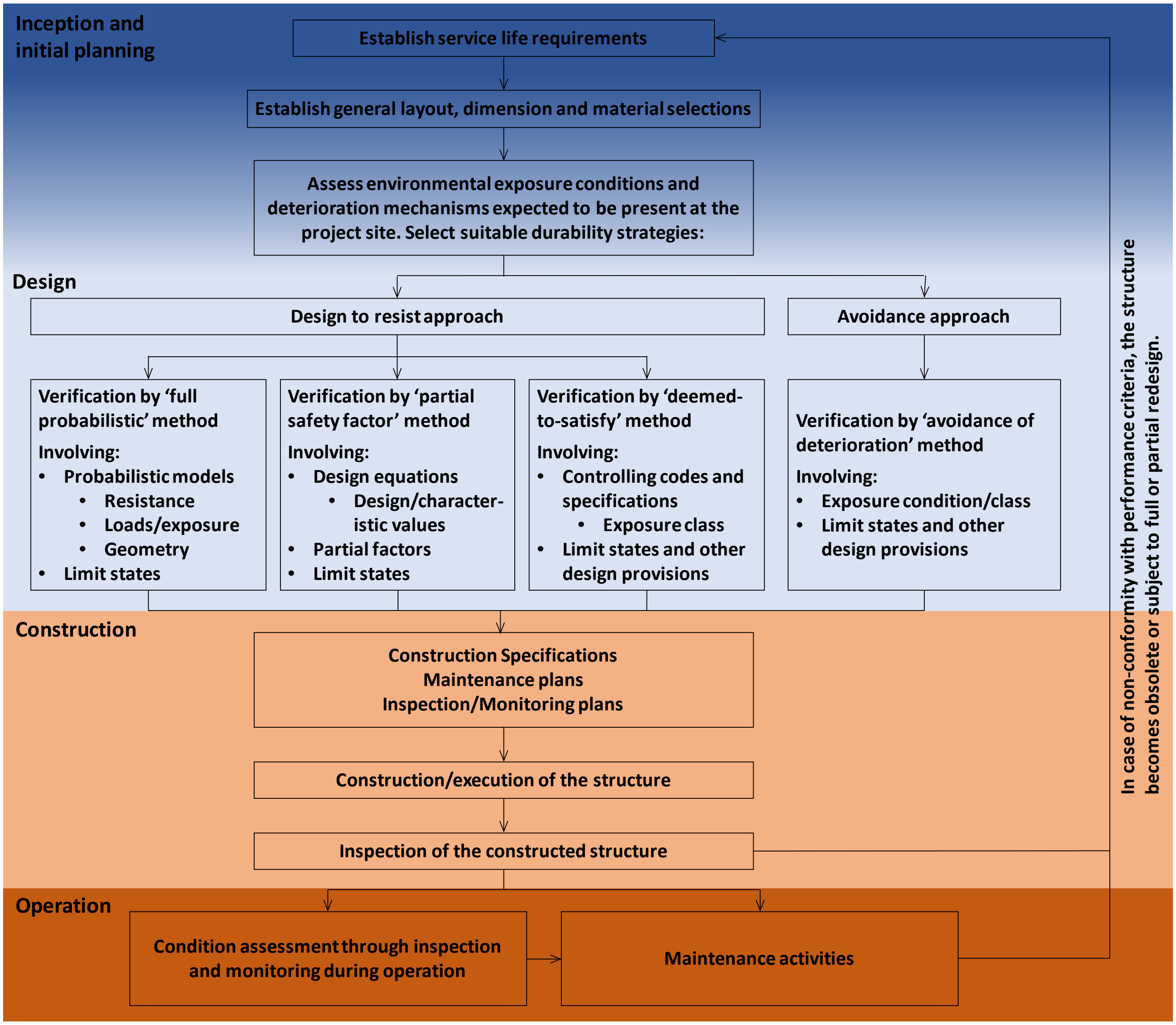

Figure 2-1: Flowchart for Service Life Design Process, after [4] and [7]

Table 2-3 provides an overview of elements or engineering tools that may be considered through the service life design process. The aim of the individual elements and tools and the phase during which the tool may be implemented are indicated in Table 3. Details and guidance on these elements are provided in the following sections.

As mentioned in Section 1.0, the appendices provide examples of the key 'Elements or tools' listed in Table 2-3, with Appendix A providing RFP examples, Appendix D providing example service life design reports, Appendix E providing example Construction Specification (Supplementary Concrete Specification), and Appendix F an example Birth Certificate. The example Supplementary Concrete Specifications given in Appendix E builds from the 'Example 2' Service Life Design

Report in Appendix D, thereby providing a direct example of how conclusions from a Service Life Design report might be effectively conveyed to the construction site.

Table 2-3: Overview of Typical Elements or Tools Utilized in the Service Life Design Process.

<table>
<tr><th>Project Phase/ Service Life Design Step</th><th>Elements or Tools</th><th>Aim and Goals</th></tr>
<tr><td rowspan="2">Inception and Initial Planning/ Establish Service Life Requirements</td><td rowspan="2">RFP, Employer's Requirements, and similar</td><td>The Owner shall establish achievable preferences and requirements for the service life of the bridge and clearly state these. Requirements may include:
• Definition of Service Life
• Service life durations for non-replaceable and replaceable components
• Desired level and frequency of maintenance
• Criteria (limit states) beyond which the structure or structural component no longer fulfils an acceptable condition
• Applicable codes and standards
• Requirements of as-built documentation (e.g., Birth Certificate)</td></tr>
<tr><td>• Any restrictions or desires by the Owner with durability implications shall be set, for example:
– Material selections and constraints (reinforcement type, steel or concrete superstructure, and basic requirements to steel coatings)
– Allow/prohibit use of specific durability measures (e.g., cathodic protection, membranes/coatings, and corrosion resistant steel types)</td></tr>
</table>

Table 2-3: Overview of Typical Elements or Tools Utilized in the Service Life Design Process.

Project Phase/ Service Life Design Step	Elements or Tools	Aim and Goals
Assess Environmental Exposure Conditions 1)	Collection of project-specific exposure information including e.g.: • Historic meteorological data • Site history • Geotechnical and geochemical investigations • Investigation of chloride profiles and surface concentrations from similar, existing structures • Water analyses	• Environmental exposure conditions in service life design are analogous to loads in structural design. The applicability and severity of potential deterioration mechanisms are controlled by the environmental exposure conditions and thus this information is a vital component in the service life design process.
Design Phase	Service Life Design Report	The aim of this step is to identify relevant deterioration mechanisms for individual components of the structure based on prevailing environmental exposure conditions, and to define suitable durability requirements to comply with associated service life requirements. Durability requirements should be concluded using one of the available durability strategies shown in Figure 2-1 and described in Section 2.5. The design phase is concluded with a Service Life Design Report that documents the implementation of the flowchart in Figure 2-1 to the given project.
Construction	Construction Specifications	Construction Specifications include project drawings, technical specification, and other documents with the aim of transferring conclusions of the service life design process to the construction site.

Table 2-3: Overview of Typical Elements or Tools Utilized in the Service Life Design Process.

Project Phase/ Service Life Design Step	Elements or Tools	Aim and Goals
	Maintenance, Inspection and Monitoring Plans	To ensure assumptions of the service life design of the structure and structural components are continually fulfilled; inspection, monitoring and maintenance is typically necessary. Plans shall be developed with consideration of project-specific details and components. The plans shall describe types of inspection/maintenance and associated intervals and procedures. These plans may be considered as part of the design phase; however, plans shall reflect the as-built structure and construction phase process (e.g., non-conformances, and changes/variations).
	Inspection of the Construction	The aim of inspection of the construction is to verify the quality of the materials and construction methods are capable of achieving the durability-related and other requirements and to assure that requirements are continually achieved during construction.
	Birth Certificate	The aim of a Birth Certificate is to compile the achieved quality/condition of the in-situ structure based on collected observations during the construction phase (e.g. mean chloride migration coefficient of concrete, and as-built concrete cover thickness). The Birth Certificate may be considered an initial inspection report, subject to updates based on future inspections.
Operation	Inspection and Monitoring Maintenance	Inspection and preservation activities aim to prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; and keep bridges in good or fair condition.

1) This service life design step may be completed as part of the inception and initial planning or design phase of the project.

2.4 Replaceable and Non-Replaceable Components

While the service life duration of a bridge is typically thought of as a single value, not all components of a bridge will have the same service life. Commonly, non-replaceable components (e.g., foundations, substructure, and decks) and replaceable components (e.g., bearings and joints) have varying service life requirements. Non-replaceable components should be designed to achieve the target service life (e.g., 100 years), while replaceable components should be designed to be exchangeable after a set period (e.g., 20, 30, and 50 years) with limited effort and limited impact on traffic. The selection of service life requirements (for replaceable and non-replaceable components) shall consider that simply maximizing the service life requirement for individual components may have detrimental impact on items such as cost. The technically feasibility of the service life requirements must also be considered, particularly for replaceable components.

Service life requirements for replaceable components may be set such that the service life of non-replaceable components is reached after one or more replacements. For example, requiring a 25- or 50-year service life for replaceable components for a bridge with an overall 100-year service life requirement would be logical. Further, it may be considered to schedule the replacement of various components simultaneously to minimize the overall impact that maintenance has on the operation and life cycle cost of the bridge.

To achieve a limited impact on traffic, it is important to consider the accessibility of replaceable and non-replacement elements and foreseen maintenance practices necessary to meet the target service life as part of the structural design process.

Table 2-4, taken from the first example RFP for Alternative Delivery Projects in Appendix A, demonstrates one possible difference in service life requirements from replaceable and non-replaceable components. Values in Table 2-4 are strictly exemplary and the required service life for individual bridge components (replaceable and non-replaceable) should be set on a project-specific basis.

Table 2-4: An Example of Minimum Service Life Requirements for Replaceable and Non-Replaceable Components of a Cable Stay Bridge, Taken from Example 1 – Signature Bridge Found in Appendix A.

Non-Replaceable Components	Minimum Service Life (years)*
Towers, foundations, abutments, piers, pier caps, deck, superstructure, approach slabs, Mechanically Stabilized Earth walls	100
Replaceable Components	**Minimum Service Life (years)***
Stay cables	60
Stay cable vibration suppression system	30
Bridge bearings	50
Expansion joints	30
Bridge traffic and pedestrian/bicycle barriers	40
Separate bridge deck wearing surface	25
Drainage system piping	75
Access systems (tower and piers): Internal access ladders, platforms, and lifts (galvanized and painted)	60

Stay cable dampers	40
Stay cable dehumidification system**	30
Painting and coatings	25
Electrical and mechanical parts of access systems (travelers, cat-walks, inspection platforms, and other access structures/sys-tems)	30
(List other project-specific components and minimum service life not cited herein.)	_____

* Durations shown are for informative purpose only and should be revised based on the characteristics of the specific project.
** If required by Service Life Design Report.

2.5 Available Durability Strategies

Different durability strategies are available to mitigate the different forms of deterioration applicable to various bridge elements. Figure 2-1 illustrates in the 'Design' section, two fundamental durability strategies available to address deterioration of bridge components:

- Avoidance approach
- Design to resist approach

The design to resist approach may be further subdivided into the following approaches:

- Full probabilistic method
- Partial safety factor method
- Deemed-to-satisfy method

2.5.1 Avoidance Approach

In the avoidance approach, the deterioration is avoided up-front by evaluating the exposure conditions or other factors impacting deterioration and establishing suitable limit states and other design provisions. Examples of avoidance approaches include use of stainless steel (with appropriate grade for exposure conditions) for reinforcement or structural steel components, to avoid corrosion and use of non-reactive aggregate in concrete to avoid alkali-aggregate reactions.

2.5.2 Design to Resist Approach

In the design to resist approach, deterioration is allowed to a certain level within the defined service life. As discussed in Section 3.0, an acceptable limit state should be included as part of the definition of service life. An example of an acceptable level of deterioration for chloride-induced corrosion of reinforcement, is the initiation of corrosion.

2.5.2.1 Full Probabilistic Method

Verification of the design to resist approach using a probabilistic model is considered the most sophisticated method [4], which is similar in nature to the LRFD employed by many structural design codes. In this approach, the environmental exposure conditions (analogous to load) and the material resistance are treated as probabilistic distributions to account for the inherent scatter of these input parameters. The probabilistic modeling process is used to establish performance-based requirements needed to provide the established acceptable limit state. 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, as discussed in Section 3.0, knowledge of the available tools is a prerequisite for the development of requirements to service life.

2.5.2.2 Partial Safety Factor Method

As described in *fib* Bulletin 34 [4], the partial safety factor method is a deterministic approach wherein the probabilistic nature of the environmental exposure conditions and material resistances are accounted for using partial safety factors. The factors are calibrated against the full probabilistic design to provide, as a minimum, the same reliability of the design. As is done with LRFD structural design, partial safety factors are applied to design input parameters to verify that the required limit state is achieved. Examples of the partial safety factor method are available in *fib* Bulletin 34 [4] and International Standards Organization (ISO) 16204 [7] for carbonation-induced reinforcement corrosion, while Design Example 4 presented in Appendix D develops a theoretical partial safety factor design methodology for chloride-induced reinforcement corrosion under certain exposure conditions.

2.5.2.3 Deemed-to-satisfy Method

Finally, the deemed-to-satisfy method utilizes descriptive rules and provisions from the codes and specifications for the given project. These provisions are used together, in some cases, with the exposure conditions, to verify the design to resist approach by applying limit states and other design provisions. Typically, deemed-to-satisfy methods in codes are not based on sound physical and/or chemical models, but on practical experience [4]. For certain deterioration mechanisms, deemed-to-satisfy rules have been shown to provide a durable performance. However, in other cases these deemed-to-satisfy rules are not capable of sufficiently considering the actual project-specific exposure situation and can ultimately fail to provide a sufficiently durable structure.

An example of a deterioration mechanism that is reliably addressed by deemed-to-satisfy rules is sulfate attack of concrete. For sulfate attack, design codes (e.g., ACI 318 [9]) commonly require 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.

The effectiveness of AASHTO LRFD [1] provisions to address chloride-induced reinforcement corrosion can vary widely, depending on environmental exposure conditions and selected design provisions [8]. AASHTO LRFD [1] and ACI 318-14 [9] (among other design codes) do not quantify exposure to chloride ions from de-icing chemicals, sea/brackish water, and/or other sources. AASHTO LRFD [1] describes situations of 'Direct exposure to salt water', 'Coastal', and 'Exposure to deicing salts' with corresponding cover requirements of 4.0 inches, 3.0 inches, and 2.5 inches, respectively (ignoring the modification factors for water/cement [w/c] ratio). However, the specific chloride exposure situation is not quantified. The reliability of deemed-to-satisfy provisions from various codes, including AASHTO LRFD [1], to address chloride-induced reinforcement corrosion were benchmarked in *fib* Bulletin 76 [8]. The benchmarking approach used the full probabilistic methodology described in Section 4.3.1.6 and in the SHRP2 R19A Academic Toolbox with an assumed limit state of reinforcement depassivation, and a 50-year service life. 'Favorable' and 'unfavorable' combinations of exposure conditions and concrete requirements that are permitted by AASHTO LRFD [1] were investigated. Favorable combinations consisted of less onerous exposure together with higher chloride resistant concrete permissible by AASHTO LRFD [1], and vice versa for unfavorable combinations. Calculations in *fib* Bulletin 76 [8] conclude that after 50 years, the use of AASHTO LRFD [1] deemed-to-satisfy provisions achieved reliability indices as low as approximately -1.0 for unfavorable combination and up to greater than 5.0 for favorable combinations. For unfavorable combinations, a reliability index of -1.0 equates to a greater than 80% probability of failure in achieving the limit state (i.e., reinforcement depassivation) after 50 years, while favorable combinations can exceed the typical reliability index (i.e., 1.3).

Further examples of both avoidance and design-to-resist approaches are described in Section 4.0.

2.6 Consideration of Life Cycle Costs and Other Factors

As part of the service life design process, it is vital to consider life cycle costs and other influential factors. The optimal design from a durability and service life perspective (for example, limit state, service life duration, material selections, and design solution) may not always provide the optimal design regarding life cycle costs or other considerations.

The life cycle cost of a bridge includes not only the initial design and construction costs, but also the cost of ownership of the bridge including the direct costs of inspections and maintenance activities and indirect costs including road user costs. Life Cycle Cost Analysis (LCCA) is an engineering economic analysis tool that allows the quantification of differential costs associated with alternatives for a given project. At the project level, LCCA of alternative new bridge designs seeks to quantify the differential costs associated with differing design features to allow optimization of costs. Therefore, consideration of life cycle costs can support the service life design process to assess optimal service life design alternatives, based on minimum cost or maximum service life durations.

To highlight the importance of LCCA on service life design and how it may be considered in the process, a briefing on the topic has been prepared as part of SHRP2 R19A [10]. The briefing provides a review of life cycle cost strategies and activities, presents a step-by-step approach to complete life cycle cost calculation for an entire bridge, and provides worked examples of how LCCA can be used as a decision-making tool for the selection of materials and details for durability.

In addition to life cycle costs, other factors including anticipated future traffic load, urban/master plans, plans for operation and future handover of the bridge (e.g., P3-projects) should also be considered during the service life design process. Additional explanation on other potentially influential factors to consider are provided in the following sections.

3.0 Guidance on Requiring Service Life Design

The first step in the service life design process, as shown in Figure 2-1, is to establish service life design requirements. Establishment of the service life requirement is typically completed by the Owner and as such the guidance in this section is likely aimed to the Owner, specifying agencies, or similar.

As described in Section 2.2, multiple viable definitions for Service Life exist and the term should be clearly defined from the outset. Therefore, the Owner's or project-specific requirements should provide a clear definition for Service Life and associated requirements to the service life design. As outlined in Table 2-3, the Owner's service life requirements may be expressed through a RFP, Employer's Requirements, or similar.

The main aim of the included service life-related text in RFPs or similar is to provide:

1) A clear definition of Service Life for the project.
2) The required service life duration for all permanent components of the bridge, including replaceable and non-replaceable components.
3) Clear definitions of the acceptable level of maintenance permitted during the service life.
4) Where appropriate, the minimum acceptable condition (i.e., limit state) to coincide with the end of service life.

Additionally, requirements to the preferred codes and standards to apply, requirements for as-built documentation (e.g., Birth Certificate), along with any other specific requirements of the Owner should be set in RFPs.

The level of specificity of service life requirements may be tailored to suit the needs of the individual project and the desires of the Owner. Changing the specificity of an RFP alters the extent of flexibility for designers to consider different solutions and adjusts the responsible party for the given aspect of the service life design process.

Two example RFPs for alternative delivery projects, given in Appendix A, provide suitable service life design requirements with varying levels of specificity:

- The first example RFP, for a signature bridge project, provides only basic service life design requirements including a project-specific definition of Service Life, durations of service life for replaceable and non-replaceable components, and required reliability index for the full probabilistic method. In this example, the assessment of exposure conditions and selection of materials is left to the designer.
- The second example RFP, for a design-build corridor project with multiple highway bridges, provides a more prescriptive RFP with specific environmental exposure conditions and material selections included.

Both example RFPs provided in Appendix A follow the four basic steps previously described, with additional specific requirements included that allow, prohibit, or require use of specific durability strategy or measures (e.g., use of a specific full probabilistic approach to address a specific deterioration mechanisms, use of cathodic protection systems, reliance on membranes/coatings, and use of corrosion resistant steel types). It is noted that the provided examples in Appendix A may be considered for guidance; however, these examples are not complete RFPs directly appropriate for use.

3.1 Option to Specify Environmental Exposure Details

As shown in Figure 2-1, material selections and assessment of environmental exposure conditions (i.e., second and third steps of the Service Life Design process) may be considered as part of either the inception/initial planning phase or the design phase. Commonly, there are opportunities to begin compiling information on the environmental exposure conditions during initial planning stages. For example, as part of a wider geotechnical testing campaign(s) of the bridge alignment, chemical evaluation of the soil and groundwater should be included. The Owner may already possess or be able to obtain environmental exposure details from similar existing structures under their authority (e.g., surface chloride concentrations in concrete structure), to inform input parameters in the service life design process.

The Owner may therefore include specific exposure condition descriptions in RFPs, if so desired and when sufficient data is available. Including specific environmental exposure conditions in an RFP would limit the responsibility and flexibility for designers and contractors and may result in more consistent proposals. Providing specific requirements for environmental exposure may be more desirable for certain projects, for example projects with a small scope and time schedule where the time needed to assess the environmental exposure conditions may be excessive compared to the overall project. A further alternative for Owners is to provide environmental exposure details to designers as information only through Reference Design documentation and require that further assessment be completed by the designer to confirm.

The second RFP in Appendix A gives an example wherein details on the environmental exposure was provided. Additional discussion on obtaining data on environmental exposure conditions is provided in Section 4.2.

3.2 Non-Service Life Considerations in Service Life Design Requirements

When establishing service life design requirements, the impact of other influential factors should not be overlooked. As discussed in Section 2.6, other questions may be considered during this phase, including for example:

- Do the Service Life definition and other controlling requirements achieve an optimal life cycle cost?
- Do traffic models or urban/master plans indicate that the structure will become functionally obsolete, due to situations such as increased congestion and planned re-purposing, long before the targeted service life duration is reached?
- Will the bridge be operated for an initial duration by a third party and, if so, what is the desired condition of the bridge at the time of handover?

In establishing service life requirements, it is also important that Owners educate themselves in the available and viable service life design tools available. As presented in the SHRP2 R19A Academic Toolbox, and for example *fib* Bulletin 34 [4] and ISO 16204 [7], full probabilistic methods are available for carbonation- and chloride-induced reinforcements corrosion. However, such models do not exist for all deterioration mechanisms affecting concrete, steel, and other construction materials.

It is also noted that the Owner may opt to complete the entire process in Figure 2-1. This may be done for a single, specific structural element, exposure condition, material, deterioration mechanism, or for the entire bridge. In case the entire service life design process is completed by the Owner, the guidance provided in the following sections on the design, construction, and operation phases should also be considered by Owners.

3.3 List of Applicable Codes, Guidelines, Specifications, and Standards

Owners may consider requiring specific codes, guidelines, specification, and standards to be applied during the design, construction, and/or operation phases. The following list provides typically referenced codes, guidelines, and specifications in RFPs or similar documents. The listed references are limited to documents with a primary focus on the basis of the service life design process. Additional detailed lists of standard material and testing specifications are provided in Sections 4.0 and 5.0, as well as in the SHRP2 R19A Academic Toolbox.

- *fib* Bulletin 34, Model Code for Service Life Design (MCSLD), 2006, International Federation for Structural Concrete
- *fib* Model Code for Concrete Structures 2010, International Federation for Structural Concrete
- ISO 16204, Durability – Service Life Design of Concrete Structures, 2012
- ISO 12944, Paints and varnishes – Corrosion protection of steel structures by protective paint systems – Part 2: Classifications of environments
- ISO 2394, General Principles on Reliability of Structures
- National Association of Corrosion Engineers (NACE) Paper 7422, Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work, 2016
- FHWA Design and Construction of Driven Piles Foundations – Volume 1
- German Concrete and Construction Association – DBV, Technical Report, Concrete Cover and Reinforcement per Eurocode 2
- Relevant, topic specific titles from the ACI Collection of Concrete Codes, Specifications, and Practices (e.g., ACI 201.2R-16 Guide to Durable Concrete, ACI 221.1R-98 Report on Alkali-Aggregate Reactivity, among others)
- Azizinamini A., Ozyildirim, H.C., Power, E.H., Kline, E.S., Mertz, D.R., Myers, G.F., and Whitemore, D.W. 2013. Design Guide for Bridges for Service Life, SHRP2, National Academy of Sciences, Washington, D.C., 744 pp.
- FHWA. 2018. Bridge Preservation Guide – Maintaining a Resilient Infrastructure to Preserve Mobility, Publication No. FHWA-HIF-18-022, 28 pp.
- AASHTO. 2017. Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction. AASHTO R 80
- DuraCrete. 2000. General Guidelines for Durability Design and Redesign. Report BE95-1347/R15. European Union, Luxembourg. 109 pp. Part of the Brite-EuRam III Project BE95-1347, DuraCrete, Probabilistic Performance Based Durability Design of Concrete Structures

- FHWA. 2010. Laboratory and Field Evaluations of External Sulfate Attack in Concrete. Technical Report. FHWA/TX-11/0-4889-1 http://ctr.utexas.edu/wp-content/uploads/pubs/0_4889_1.pdf

4.0 Guidance on Implementing Service Life Design during Structural Design

As illustrated in Figure 2-1, following the establishment of service life design requirements, steps that should be completed are as follows:

- Establish general layout, dimension and material selections.
- Assess environmental exposure conditions and deterioration mechanisms expected to be present at the project site.
- Select suitable durability strategies to mitigate relevant deterioration mechanisms.
- Verify service life requirements using one of the four available durability strategies.

As discussed in Sections 2.3 and 3.1, the first two items listed may be included in the Owner's service life design requirements or determined through the design process. Regardless of how the service life design process is put into action for a project, the following pieces of guidance for service life design, during the structural design process, are considered important:

- Initiate the service life design process early during the design process.
- Clearly document the assumptions and conclusions of the service life design process in a Service Life Design Report (or similar).
- Verify that the key conclusions of the Service Life Design Report are appropriately transferred to structural designers, operations and maintenance experts, and to the construction site.

While more specific guidance is provided of the service life design process in Sections 4.2 to 4.4, the importance of the above list should not be overlooked. The service life design process will conclude fundamental parameters, influencing the structural design (e.g., concrete cover thicknesses, corrosion allowances, and material selections). Assessment of the environmental exposure conditions may also be required, if details are not provided in an RFP or other documents from the Owner. Delaying the service life design may therefore result in changes in basic design parameters, which can be avoided by simply initiating the task in a timely manner.

4.1 Service Life Design Report

A vital tool in the service life design process, as shown in Table 2-3, is the Service Life Design Report. Other names are commonly used for this report, including Durability Assessment Report, Concrete/Steel Durability Report, and Corrosion Protection Plan[1]. The report is a basis document with the fundamental aim of describing how the flowchart in Figure 2-1 is to be implemented on a given project. Typically, the Service Life Design Report focuses primarily on the design phase items in Figure 2-1. Specifically, the report should summarize details on the environmental exposure conditions and provide an assessment of relevant deterioration mechanisms for individual components and structural materials. Furthermore, suitable durability requirements should be determined to mitigate possible deterioration in compliance with the established service life design requirements.

It is a common misconception that all aspects of the design, construction, and operation phases of service life design are to be completely addressed by the Service Life Design Report. However, as shown in Table 2-3 additional tools are developed and used during these phases. The Service Life Design Report should be treated as a basis document that is considered, along with other inputs, in the development of Construction Specifications; Maintenance, Inspection and Monitoring Plans; and other service life design tools listed in Table 2-3. To clarify the aim of a Service Life Design Report and to provide worked examples, Appendix D includes several exemplary Service Life Design Reports for bridges of varying type, service life duration, and exposure conditions.

Conclusions of the service life design process typically impact fundamental parameters in the structural design. These may include:

- For concrete structures:
  - Required cover thickness
  - Materials selections, including potential need for supplementary cementitious materials (e.g., slag, and fly ash)
  - Possible need for (selective) use of alternative reinforcement material (e.g., stainless steel)
- For structural steel:
  - Required corrosion allowances
  - Possible need for (selective) use of alternative material (e.g., stainless steel)
  - Possible need for coating system(s)

As a result, the majority of the service life design work should be completed early in the design phase. In certain cases, particularly for concrete structures with an extended service life requirement and severe exposure conditions, concrete covers specified in codes may not be sufficient.

[1] The name Corrosion Protection Plan is not a preferred name for this type of report, particularly for concrete, as the name under-emphasizes the importance of other deterioration mechanisms.

Discovery of a need for increased cover thickness late in the detailed design process may necessitate a complete redesign of certain elements. Similarly, the required corrosion allowance for structural steel elements embedded in corrosive soil (e.g., steel piles) should be determined early in the project to avoid the need for redesign of the foundations.

Concluded durability-related provision to the structural design necessary to fulfill the service life requirements should be effectively transferred through the entire structural design process by inclusion of such provision in the structural design basis/criteria documents.

4.2 Environmental Exposure Conditions, Exposure Zones and Deterioration Mechanisms

This section provides guidance on how a structure may be subdivided into exposure zones based on the prevailing environmental exposure conditions, and on the assessment of corresponding potential deterioration mechanisms.

As described in Table 2-3, environmental exposure conditions in service life design are analogous to load in structural design. The applicability and severity of potential deterioration mechanisms are controlled by environmental exposure and thus this information is a vital component in the service life design process. Location-dependent (both geographic location and location on a given bridge) variations in the prevailing environmental conditions will impact the potential deterioration mechanisms affecting a bridge. For example, portions of a concrete structure permanently submerged in water will not be exposed to carbon dioxide (CO_2); therefore, carbonation-induced reinforcement corrosion can be excluded in these areas. Similarly, from a geographic perspective, structures in southern states may not be subjected to freeze/thaw cycles, de-icing salt exposure and associated attacks. The descriptions provided in Sections 4.3 and 4.4 (also see SHRP2 R19A Academic Toolbox, ACI 201.2R-16 [23]) can be used as guidance in the assessment of relevant deterioration mechanisms based on the prevailing environmental exposure conditions.

Characteristics of the environmental exposure conditions should be investigated and described, as these characteristics control which of the potential deterioration mechanisms are relevant. As shown in Table 2-3, the project-specific details on the environmental exposure conditions can be compiled from:

- Historic meteorological data
- Site history
- Geotechnical and geochemical investigations
- Investigation of chloride profiles and surface concentrations from similar, existing structures
- Water analyses

Any additional investigation(s) of soil and water needed to determine the prevailing environmental exposure conditions should typically be considered a critical path item during the design phase as the severity of the exposure will impact the fundamental parameters listed in Section 4.1.

It is a common approach to subdivide a structure into exposure zones, based on the locally prevailing exposure conditions. These exposure zones need not be constant for different materials and commonly reinforced concrete and structural steel elements rely on different exposure zone descriptions and classifications. As such, guidance is provided in the Section 4.2.1, Reinforced Concrete and Section 4.2.2, Structural Steel. These sections consider typically encountered environmental exposure situations for bridges, which may potentially lead to the following deterioration mechanisms:

- For reinforced concrete described in Section 4.3:
  - Freeze-thaw attack and scaling
  - Alkali-aggregate reactions (AAR)
  - Delayed ettringite formation (DEF)
  - Sulfate attack
  - Chloride- and carbonation-induced reinforcement corrosion
- For structural steel described in Section 4.4:
  - Corrosion

Other deterioration mechanisms may possibly occur beyond those listed above and covered in Sections 4.3 and 4.4 for a given structure. These additional deterioration mechanisms may include ice abrasion, microbial-induced deterioration of concrete or steel, stray-current induced corrosion of reinforcement or structural steel, among others. However, guidance on such deteriorations and corresponding environmental exposure conditions is considered outside the scope of this document as specialist guidance would be needed based on project-specific evaluations. If there are concerns of deterioration mechanisms beyond those listed above, expert support should be sought.

Sections 4.2.1 and 4.2.2 provide additional guidance on the assessment of environmental exposure conditions for reinforced concrete and structural steel, respectively. Sections 4.3 and 4.4 provide additional detail on potential mitigation measures to address possible deterioration mechanisms.

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4.2.1 Reinforced Concrete

Figure 4-1 and Figure 4-2, taken from Example 1 in Appendix D, provide examples of reinforced concrete components of a bridge being subdivided into exposure zones. In that case, the exposure zones identified were:

- Atmospheric Zone
- Buried Zone
- Indirect de-icing salt zone
- Direct de-icing salt zone

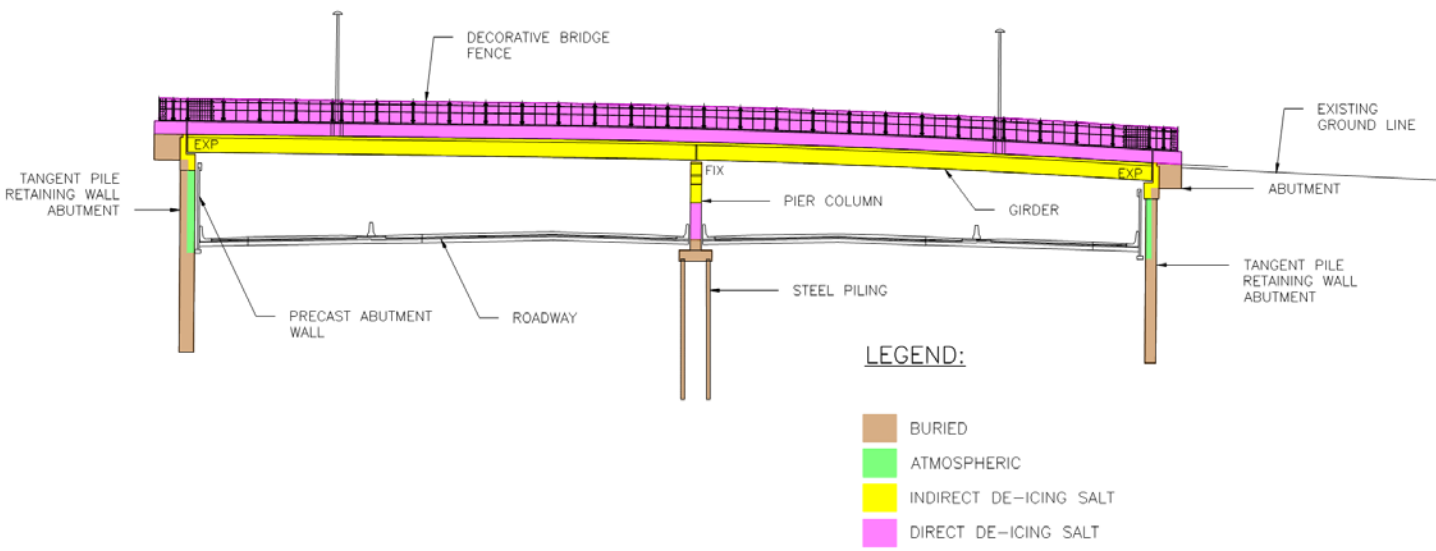


Figure 4-1: Example of Exposure Zones for a Composite Bridge Deck Sub- and Superstructure, from Example 1 in Appendix D.

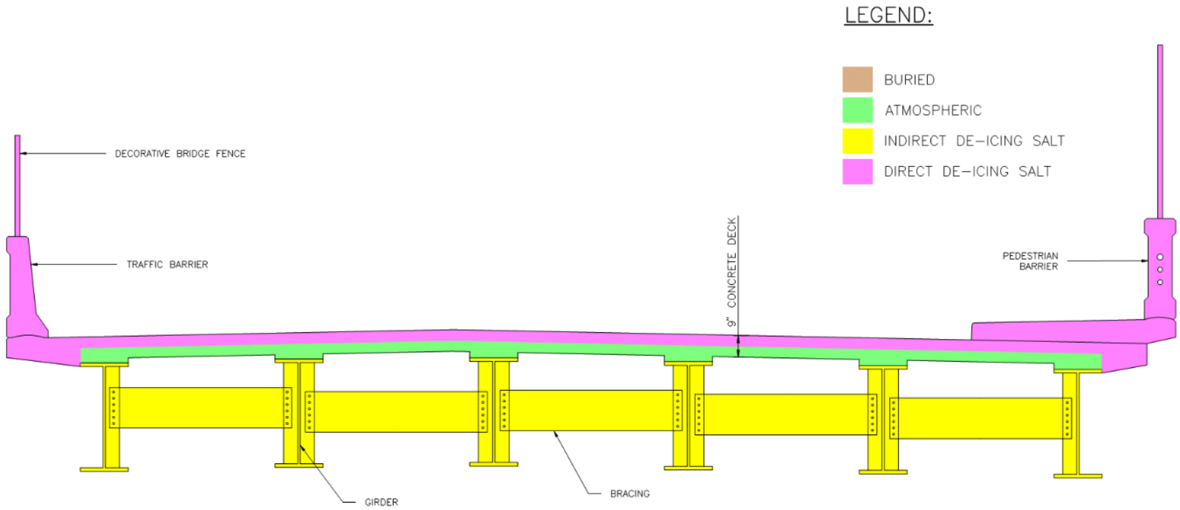


Figure 4-2: Example of Exposure Zones for a Composite Bridge Deck Superstructure, from Example 1 in Appendix D

Other zones may exist for reinforced concrete structures based on project-specific considerations, including splash, spray and/or tidal and submerged (in water) zones. The example Service Life Design Reports in Appendix D cover these exposure zones.

In certain cases, the boundary line between individual exposure zones for reinforced concrete is obvious. For example, the final ground level would distinguish the boundary between the Buried Zone and the Atmospheric Zone. However, in other cases these boundary lines may be more difficult to distinguish. Literature sources (e.g., *fib* Bulletin 34 [4] and BS 8500-01 [22]) may be considered in this determination and in certain cases local codes may also provide appropriate definitions. For example, the Example 3 Service Life Design Report in Appendix D utilizes the definition for 'Splash, Spray and Tidal Zone' from a controlling code applicable in that example (i.e., the Florida DOT Structures Design Guidelines).

For each exposure zone, basic descriptions on the environmental exposure conditions should be developed including consideration of access to moisture, oxygen (O_2) and/or CO_2, and/or other potentially harmful substances including sulfate and chloride ions. The majority of deterioration mechanisms affecting reinforced concrete listed in Section 4.3 require one or more of these substances to occur. Details on the environmental exposure characteristics for individual exposure zones can be determined through databases (e.g., temperature and humidity) or literature sources, while other parameters will require sampling and testing (e.g., sulfate and chloride content of groundwater, soil, and waterways). Information on the site history may also provide pertinent information on possible contamination and should also be considered. Site history is important for structural steel, with certain exposure classifications systems from standards directly accounting for the site history.

Online resources are available for historic data on temperature, relative humidity, freeze-thaw cycles, precipitation, and other parameters regarding the environmental conditions in the atmosphere. For example, the National Oceanic and Atmospheric Administration's National Centers for Environmental Information has extensive climate datasets online (https://www.ncdc.noaa.gov/cdo-web/datasets).

Certain characteristics of the exposure zones can be based upon classifications from codes and standards. For example, ACI 318 provides a classification system (and corresponding mitigation measures, see Section 4.3) for sulfate exposure. This classification system relies on the following test methods:

- American Society for Testing and Materials (ASTM) C1580 for the water-soluble sulfate content of soil
- ASTM D516 or ASTM D4130 for the concentration of dissolved sulfates in water

Sulfate exposure conditions should be reported as SO_4^{2-}, and results reported in terms of SO_3 shall be appropriately converted (in case of results reported as mg/l SO_3 should be multiplied by a factor 1.2 [21]).

The severity of exposure of reinforced concrete elements to chloride from marine sources, deicing salt exposure, or other sources (i.e., surface chloride concentration) should also be assessed. Obtaining reliable and representative surface chloride concentrations for the design of new structures is a common hurdle in the design process. Surface chloride concentrations may be measured from existing structures (e.g., according to either AASHTO T-260 or ASTM C1152), and often Owners possess such data from existing structures after years of exposure. As part of the SHRP2 R19A project, surface chloride concentrations were therefore measured from samples extracted from the bridge decks in Iowa, Oregon and Virginia as compiled below. Surface chloride concentration data from existing structure subjected to similar exposure conditions to that for a new structure is highly valuable in informing the design of new structures.

Summaries of measured surface chloride concentrations from Iowa are provided in Table 4-1 and Table 4-2, while data from Oregon is provided in Table 4-3 and Table 4-4. Table 4-5 compiles historic data on surface chloride concentration from bridge decks in Virginia, with the locations of individual regions provided in Figure 4-7. As shown in the tables provided, the surface chloride concentration may vary widely based on the location of the bridge deck, likely due to variations in exposure to chloride from de-icing salt application or exposure to marine conditions.

Figure 4-3 identifies the locations of the bridge corridors investigated in Iowa. The locations of the IA-9 corridor bridges are provided in Figure 4-4, with the summary of measured surface chloride concentrations given in Table 4-1. Locations and measured surface chloride concentrations from the US-18 corridor bridges are given in Figure 4-5 and Table 4-1, respectively.

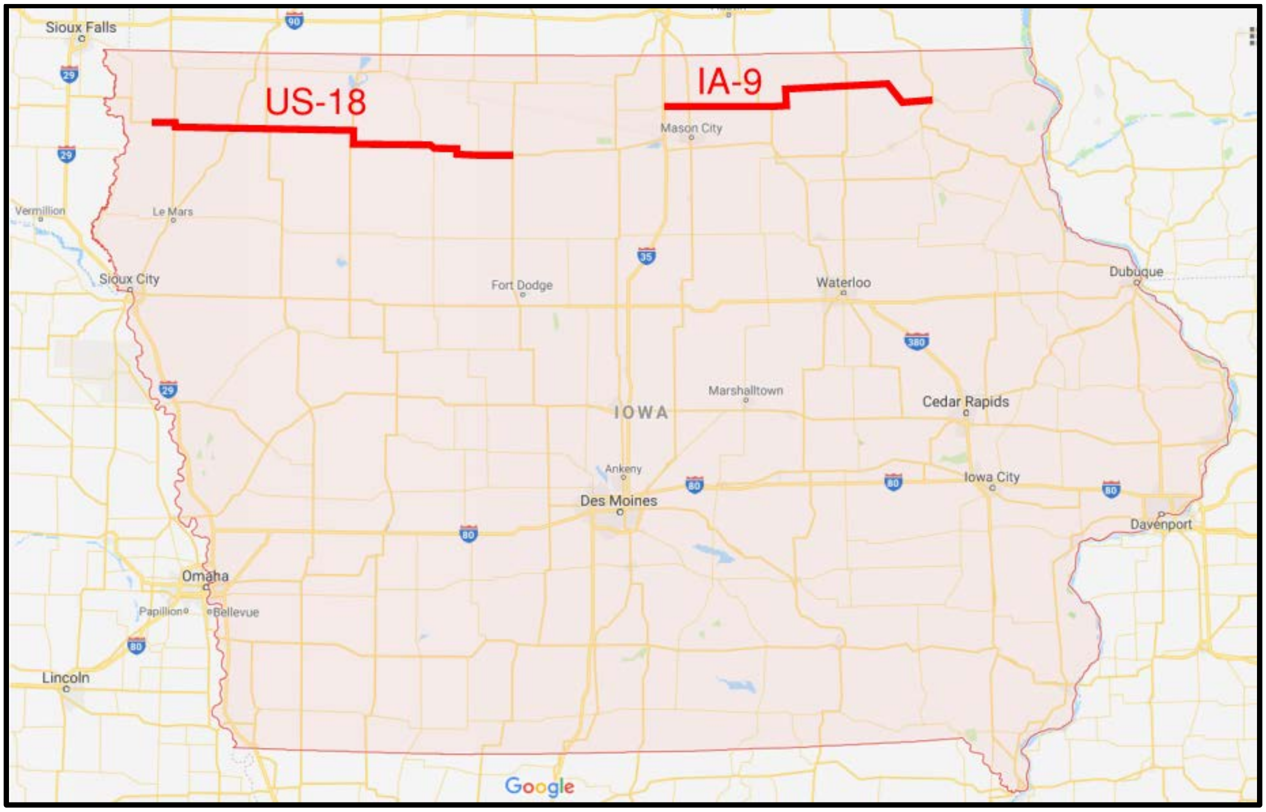


Figure 4-3: Iowa Highway Corridors for Chloride Profile Study

Table 4-1: Summary of Measured Surface Chloride Concentrations on the Deck Surface in the IA-9 Corridor Bridges in Iowa

Bridge #	54021	36501	36511	36531	36541	36561	28721	28731	52590
Test Method	AASHTO T-260								
Year constructed	2004	1992	1992	1979	1978	1979	1976	1976	1969
Surface Chloride Concentration (%mass of binder)	4.75	2.35	2.97	3.49	3.38	3.74	4.18	3.91	3.29
	3.92	2.63	2.73	4.10	2.70	3.72	3.26	2.93	2.09
	6.20	3.81	3.37	3.50	3.62	2.88	3.60	3.44	2.85
	4.09	4.36	3.78	4.33	3.40	4.92	4.72	3.53	2.52
			2.95		3.82			3.95	
			2.30		3.45			2.40	
			3.81		2.83			3.32	
			3.32		2.50			2.74	

Table 4-1: Summary of Measured Surface Chloride Concentrations on the Deck Surface in the IA-9 Corridor Bridges in Iowa

Bridge #	54021	36501	36511	36531	36541	36561	28721	28731	52590
			2.37		3.81			3.04	
			2.73		4.14			3.45	
			4.69		3.10			4.63	
			2.78		3.43			4.38	
			3.83						
			2.87						
			2.03						
			2.44						
Average (%mass of binder	4.74	3.29	3.07	3.85	3.35	3.81	3.94	3.44	2.69

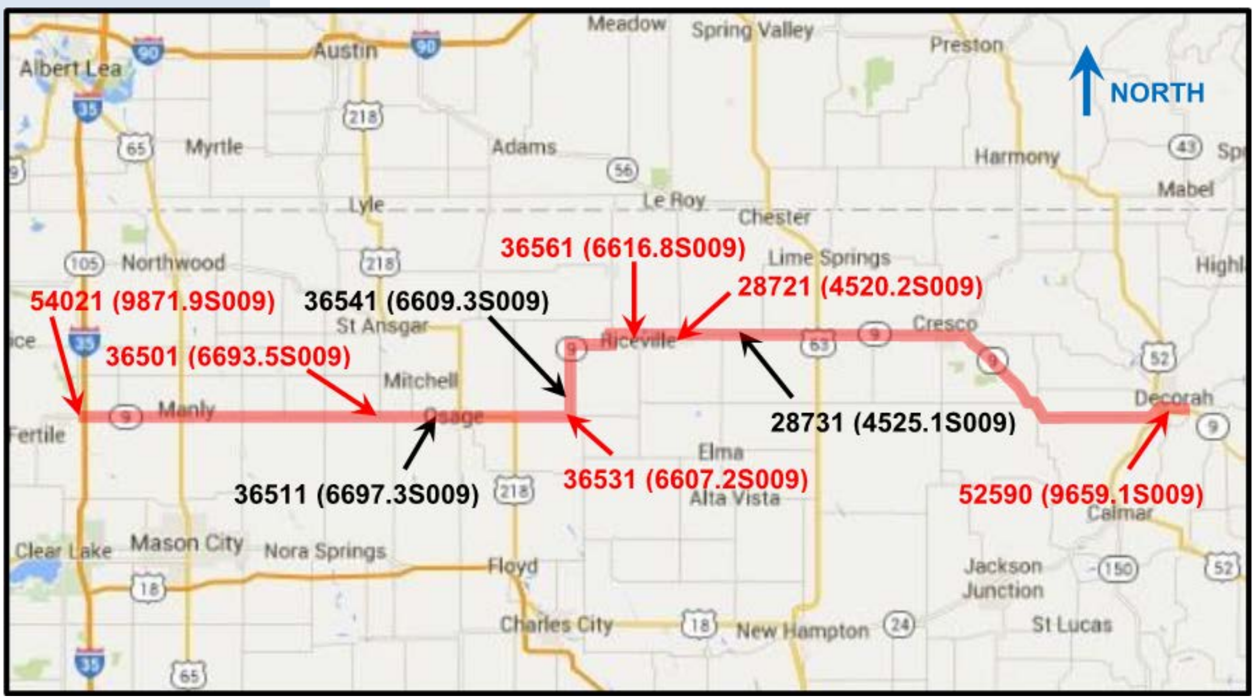


Figure 4-4: IA-9 Bridge Locations

Table 4-2: Summary of Measured Surface Chloride Concentrations on the Deck Surface in the US-18 Corridor in Iowa

Bridge #	48211	48221	48231	48281	48351	20331	32821	32831	32841
Test Method	AASHTO T-260								
Year constructed	2007	1989	2002	1985	1992	2006	1993	1993	1984
Surface Chloride Concentration (%mass of binder)	2.11 1.45 2.46 2.36	3.13 3.26 2.91 2.55 4.03 2.97 3.03 3.18 3.25 2.53 2.24 2.98	2.88 3.16 3.43 3.32 2.99 3.19 3.67 3.67	2.81 2.89 2.73 1.86	2.34 2.02 2.59 3.05	4.16 3.26 3.65 2.68 2.64 3.01 3.37 2.95	3.46 2.42 3.95 3.94	3.97 3.36 3.08 3.19	3.81 3.07 3.07 3.47 3.89 3.45 2.00 2.77
Average (%mass of binder	2.09	2.99	3.29	2.50	2.50	3.22	3.44	3.21	3.10

Figure 4-5: US-18 Bridge Locations

Figure 4-6 identifies the locations of the bridge investigated in Oregon. As shown in the figure, Oregon is divided into the following three regions:

- Pacific Coast
- Willamette Valley
- Cascade Mountains and East Regions of Oregon

Data from bridges located in the Pacific Coast and Cascade Mountains and East Regions of Oregon are provided in Table 4-3, while data from the Willamette Valley are provided in Table 4-4.

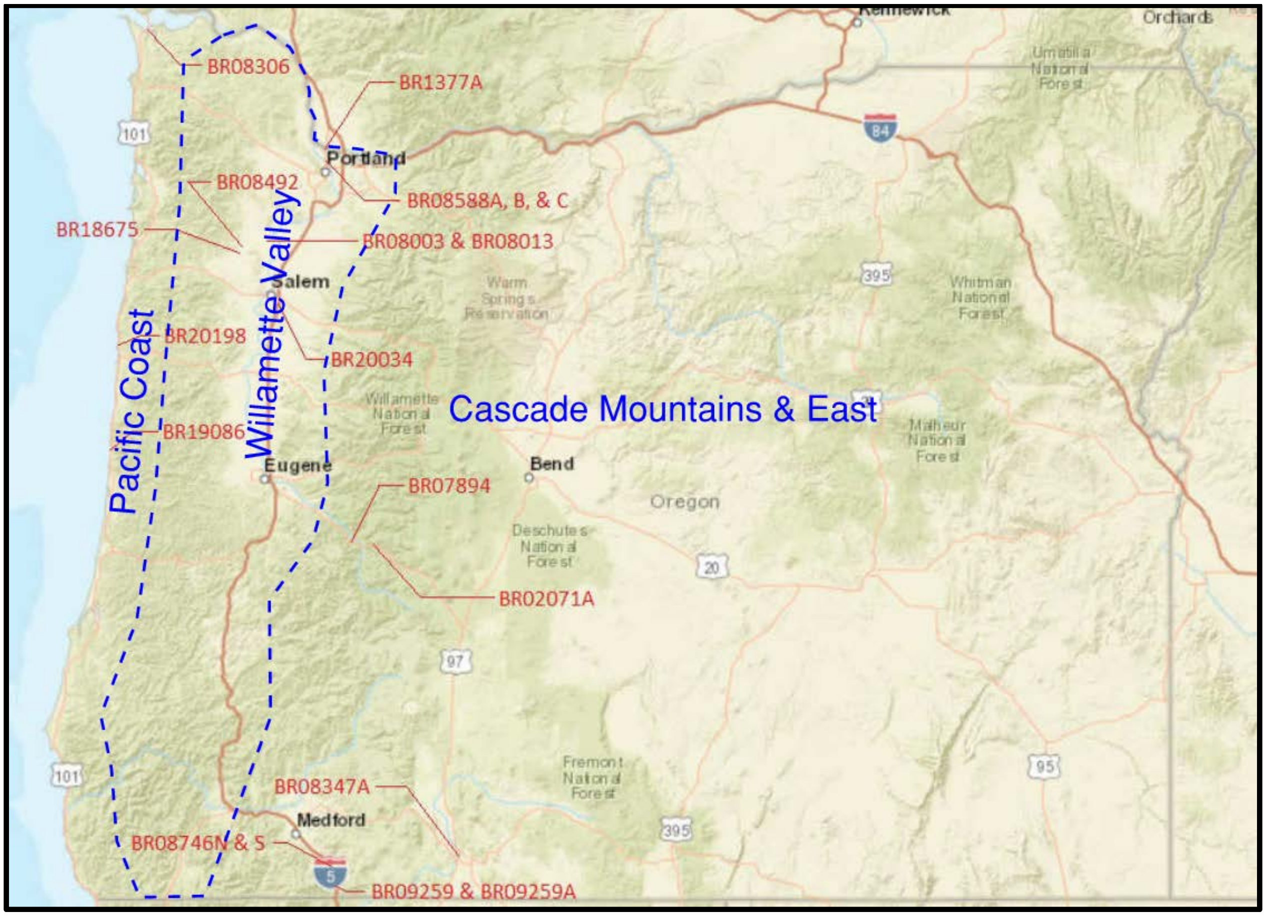


Figure 4-6: Location of Oregon In-Service Bridge Deck Testing and Exposure Zones

Table 4-3: Summary of Measured Surface Chloride Concentrations in the Pacific Coast and Cascade Mountains and East Regions of Oregon

Region	Pacific Coast			Cascade Mountain & East						
Bridge #	Spencer Creek BR20198	Bob Creek BR19086	Youngs Bay BR08306	Willamette River (Barnard) BR07894	Salt Creek BR2071A	Link River, Hwy 4 NB Conn BR08347A	Hwy 1 Over Crow-son Rd. NB BR08746N	Hwy 1 Over Crow-son Rd. SB BR08746S	Hwy 1 Over Hwy 273 SB BR09259	Hwy 1 Over Hwy 273 NB BR0925 9A
Test Method	ASTM C1152		AASHTO T-260	ASTM C1152						
Year con-structed	2008	2003	1964	1955	1965	1968	1963	1963	1965	1965
Surface Chloride Concentra-tion (%mass of binder)	1.37 1.57	1.99 2.32	0.33 0.79 0.33 0.97	0.96 1.09 2.40 1.62	1.21 1.18 1.16 1.53 1.39 0.65	1.01 1.06 0.97	2.01 1.71 2.25	2.74 2.38 2.79 2.13	2.63 5.42 4.84	5.01 4.84
Average (%mass of binder)	1.47	2.16	0.60	1.52	1.19	1.01	1.99	2.51	4.30	4.93

Table 4-4: Summary of Measured Surface Chloride Concentrations in the Willamette Valley Region of Oregon.

Bridge #	Yamhill River Overflow, Hwy 39 BR08492	South Yamhill River (White-son) BR18675	Interstate Bridge NB BR01377A	Banfield Inter-change BR08588A	Banfield Inter-change BR08588B	Banfield Inter-change BR08588C	Yamhill River (Dayton) BR08003	Hwy 39 Over Hwy 150 BR08013	Mill Creek (I5) NB BR20034
Test Method	AASHTO T-260	ASTM C1152	AASHTO T-260						ASTM C1152
Year con-structed	1963	2002	1916	1963	1963	1963	1955	1957	2007
Surface Chlo-ride Concen-tration (%mass of binder)	0.37 0.22 0.32 0.31	0.74 0.17 0.08 0.05	0.16 0.12 0.21 0.13 0.16 0.10 0.42 0.13 0.16 0.21 0.27 0.13	0.21 0.20 0.16 0.18	0.13 0.15 0.13 0.11	0.13 0.13 0.18 0.20	0.47 0.45	0.06 0.08	0.27 0.07 0.18 0.22
Average (%mass of binder)	0.30	0.26	0.18	0.19	0.13	0.16	0.46	0.07	0.18

Table 4-5 compiles historic data from bridge decks in various regions of Virginia. Figure 4-7 illustrates the regions and corresponding surface chloride concentration.

Table 4-5: Historical Data on Virginia Bridge Deck Surface Chloride Concentration.

Region	Surface Chloride Concentration (%mass of binder)
Tidewater	0.42
Eastern Piedmont	0.78
Western Piedmont	1.33
Northern	0.99
Central Mountain	0.72
Southwestern Mountain	1.57

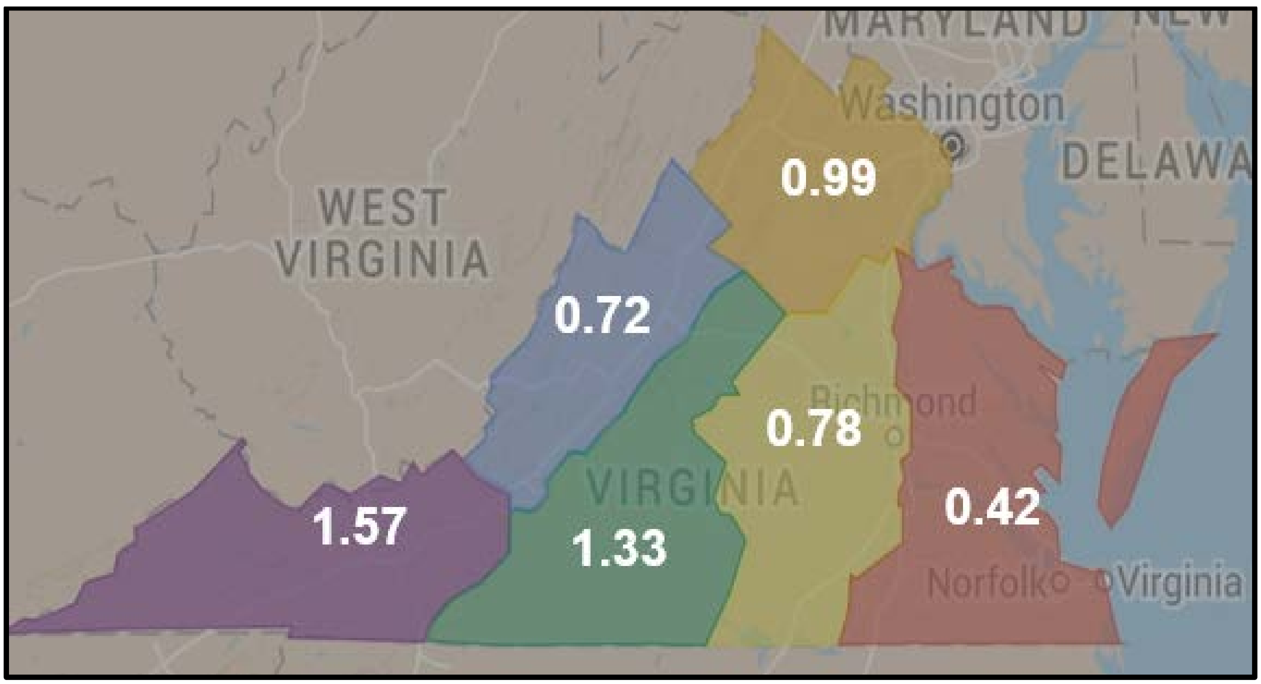


Figure 4-7: Virginia Surface Chloride Concentration by Region

Where data or access for testing of similar structures in proximity of the new structures is unavailable, surface chloride concentrations from literature sources are commonly used as input in the service life design. In such cases, values from literature should be taken from structures that are similarly exposed to the structure under consideration.

4.2.2 Structural Steel

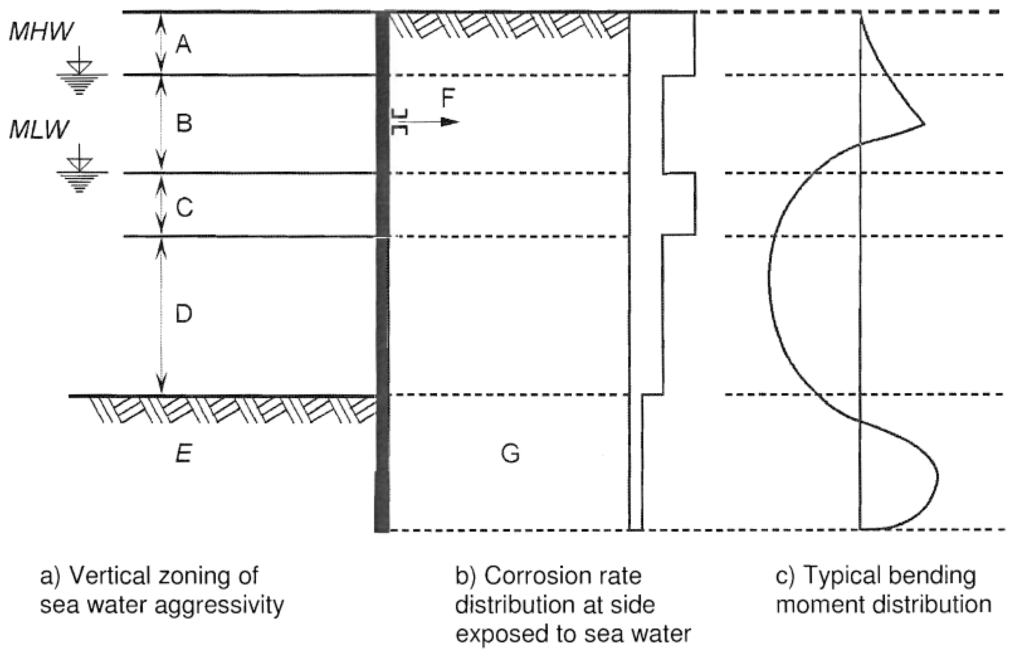
For structural steel, environmental exposure is generally classified as atmospheric, soil or water, with varying levels of corrosion potential existing within these broad classifications. The exposure classification selected should also consider the selected mitigation method (see Section 4.4).

Figure 4-8, from EN 1993-5 [17], illustrates an example exposure zones classification for structural steel element located in marine environments. The figure includes indicative corrosion rate and bending moment distribution profiles, illustrating the importance of selecting the controlling corrosion allowance on the structural integrity of the element. The FWHA Manual [18] also provides test methods and associated criteria for classification of the aggressivity of soil. The FHWA Manual [18] includes the following electrochemical tests for soil and groundwater for aggressiveness of the buried zone with regard to the potential for pile deterioration:

- pH (AASHTO T289 / ASTM D4972)
- Resistivity (AASHTO T288 / ASTM D1125)
- Sulfate ion content (AASHTO T290 / ASTM D4230)
- Chloride ion content (AASHTO T291 / ASTM D512)

Other references are available in defining exposure zones for structural steel protected by coatings. ISO 12944-2 [20] provides an exposure classification convention for structural steel in a temperate climate and includes descriptive examples of interior and exterior situations that can be categorized in the various exposure zones.

Care of the applicability of a given code to the precise situation of a given project should be considered. The selected code should reflect the selected mitigation method and be based on similar exposure details (for example, ISO 12944-2 considers structural steel in temperate climate). Where available, local experience, practice, guidelines, and/or codes should be considered, particularly when evaluating corrosion rates for structural steel.



A	Zone of high attack (splash zone);	B	Intertidal zone;
C	Zone of high attack (Low water zone);	D	Permanent immersion zone;
E	Buried zone (Water side);	F	Anchor;
G	Buried zone (Soil side)		
MHW	Mean high water;	MLW	Mean low water

Figure 4-8: Example of Exposure Zones for Structural Steel in a Marine Environment, from EN 1993-5 [17].

4.3 Concrete Components

The most common concrete deterioration mechanisms are briefly summarized herein. A more detailed description of the different types of deterioration can be found in the SHRP2 R19A Academic Toolbox. Table 4-6 in Section 4.3.2 summarizes potential durability design strategies that may be considered to mitigate the various potential deterioration mechanisms and to verify the service life requirements can be achieved.

4.3.1 Concrete Deterioration Mechanisms

4.3.1.1 Freeze-Thaw Cycles

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.

4.3.1.2 Scaling

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.

4.3.1.3 Alkali-Aggregate Reactions

Aggregates containing reactive minerals react with alkalis from the cement and/or from external sources, such as de-icing salts, under the presence of water and high pH-value to form an expansive gel.

4.3.1.4 Delayed Ettringite Formation

Form of internal sulfate attack that can occur in concrete cured at elevated temperatures such as in precast units or mass concrete placements.

4.3.1.5 Sulfate Attack

Expansive sulfate reactions occur when Portland cement with a moderate- to-high C_3A-content is used in concrete in contact with sulfate-bearing water or soil containing dissolved sulfates.

4.3.1.6 Reinforcement Corrosion

Carbon dioxide from the surrounding air reacts with calcium hydroxide in the cement paste, which decreases the pH-value of the concrete pore solution. The alkaline protective reinforcement environment breaks down, which can initiate reinforcement corrosion.

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.

4.3.1.7 Influence of Concrete Cracking on Deterioration Mechanisms

Cracks, which may develop in plastic, hardening or hardened concrete, can provide a pathway for accelerated ingress of moisture, chloride, CO_2, and other substances potentially aggressive to the embedded reinforcement or the concrete itself. Commonly, the primary focus of concrete cracking is related to its impact on the reinforcement corrosion process, which is discussed below. However, the impact of cracking on deterioration process affecting the concrete itself should not be overlooked. Excessive concrete cracking can influence concrete resistance against freeze-thaw attack, sulfate attack (external and internal, i.e., DEF), and AAR [23], [40] as well as resistance to abrasion [39]. To minimize the impact of cracks on these deterioration processes, in addition to the mitigations described in Table 4-6, the structural design and construction processes including concrete placement, consolidation, evaporation protection, finishing, and curing should follow industrial standards to minimize cracking and to control the widths of cracks, for example through application of design provision for reinforcement distribution in AASHTO LRFD [1].

Figure 4-9 illustrates the basic electrochemical and physical processes taking place during corrosion (on the left) and the fundamental impact cracking has on these processes (on the right). The concrete provides a diffusion barrier against chloride and carbon dioxide, which influence the anodic process, and against oxygen, which is required in the cathodic process. It is important to note that both the anodic and cathodic process are required to sustain active corrosion. In uncracked concrete, chloride ions or carbon dioxide gradually penetrate into the concrete and may eventually lead to depassivation of the reinforcement. Mitigation measures in Table 4-8 describe means to address this process for the duration of the service life. However, cracking of the concrete can provide a path of easy access for chloride or carbon dioxide to the reinforcement. Nevertheless, as illustrated in the schematic on the right of Figure 4-9, the anodic process takes place at the steel intersecting the cracked and carbonated concrete, while the steel contained in the undamaged concrete participates in the cathodic process, as has been observed from several investigations, see e.g., [42], [43], [44]. The significance of crack width is of lesser importance in this case as the cathodic process in the uncracked concrete will depend on the diffusion of oxygen through the concrete. Therefore, the primary measures to address chloride-induced reinforcement corrosion (i.e., concrete cover thickness, low permeability concrete, etc., see Table 4-8) remain the primary measures in the event of cracks.

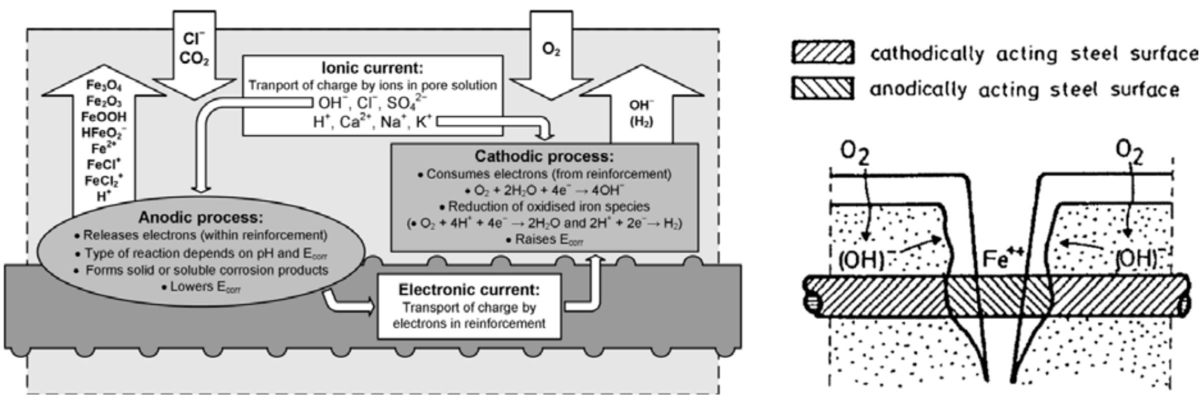


Figure 4-9: (Left) Illustration of electrochemical and physical processes occurring in reinforcement corrosion process, from [41], and (Right) illustration of the basic impact concrete cracks have on reinforcement corrosion with the white cover zone indicating a layer of carbonated concrete, from [42].

The fib Bulletin 34 permits crack width up to 0.30 mm (approximately 0.012") without additional protection for a 'high quality of concrete cover', described as combination of a concrete with w/c-ratio ≤0.50 and a minimum cover thickness of 50 mm (approximately 2"). Additional caution should however be applied to situations where chloride-laden water can pond and lead to accumulation of chlorides in cracks due to evaporation of water on e.g., top surface of decks. It is also noted that AASHTO LRFD [1] seems to recognize the limited impact of crack width on durability and states in Section 5.6.7 commentary that "Previous research indicates that there appears to be little or no connection between crack width and corrosion."

4.3.2 Concrete Mitigation Methods

There are several strategies available to perform service life design for concrete structures as discussed in Section 2.5 and shown in Figure 2-1. The most typically used durability strategies for the different concrete deterioration mechanisms described in the last section are summarized in Table 4-6. The table also summarizes general mitigation methods and potential code and standard provision to address the deterioration mechanism in question. Table 4-7 provides recommendations on the input parameters for the full probabilistic *fib* Bulletin 34 calculations.

Appendix D provides three worked design examples wherein the prevailing exposure conditions are described and evaluated. Relevant deterioration mechanisms are identified and mitigated using similar measures as those described in Table 4-6.

Table 4-6: Mitigation Methods used for Deterioration Mechanisms affecting Reinforced Concrete.

Deterioration Mechanism	Available Durability Strategy	Contributing Exposure Situation(s)	General Mitigation Methods	Potential Code and Standard Provisions	Potential Test Methods
Freeze-Thaw and Scaling	Avoidance of deterioration	Concrete exposed to freeze-thaw cycles with or without de-icing salts or other chloride sources.	- Using freeze-thaw resistant aggregates - Providing air-entrainment, ACI 318-14 provides required fresh air contents based on various factors. - Designing provisions to avoid ponding on horizontal surfaces (drainage slopes) - Evaluating resistance to freeze-thaw and/or scaling	Requirements from ACI 318-14: - For concrete classified as F1: a maximum water-cement ratio of 0.55 and a minimum compressive strength of 3,500 psi (24 MPa). Plastic air content of 4.5% for maximum aggregate size of 1''. - For concrete classified as F2: a maximum water-cement ratio of 0.45 and a minimum compressive strength of 4,500 psi (31 MPa). Plastic air content of 6% for maximum aggregate size of 1''. - For concrete classified as F3: a maximum water-cement ratio of 0.40 and a minimum compressive strength of 5,000 psi (35 MPa). Plastic air content of 6% for maximum aggregate size of 1''.	- Plastic air content of freshly mixed concrete tested for suitability of air void content and network, e.g., ASTM C 138 [24], ASTM C 231 [25], ASTM C 457 [12]. - Air-void system of hardened concrete in accordance with ASTM C457. ACI guideline: maximum spacing factor of 0.008 inches. - Freeze-thaw resistance in accordance with ASTM C666 Procedure A. Recommendation: minimum durability factor of 90 after 300 cycles. - Resistance to scaling for deck and barrier concrete in accordance with ASTM C672 with an appropriate project-specific requirement to the visual rating after a number of freeze/thaw cycles. Alternatively, CSA A23.2-22C testing can be used with the associated requirement of a maximum mass loss of 0.16 psf (0.8 kg/m^2).
AAR	Avoidance of deterioration or Deemed-to-satisfy	Exposure to water or moisture during service together with chemical composition of aggregate	- Using non-reactive aggregate (Avoidance) or avoid exposure to moisture or, in case of use of a reactive aggregate, selection of deemed-to-satisfy provisions to mitigate deleterious expansions, which may include: - Limiting the equivalent alkali content of the concrete. - Using of a sufficient amount of supplementary cementitious materials based on prescriptive or performance-based requirements, for example, from AASHTO R80-17 [11]. - Combining the limiting the equivalent alkali content and using supplementary cementitious materials.	Guidance from AASHTO R80-17 [11] can be used.	The testing paradigm in AASHTO R80-17 may be used to evaluate the potential reactivity of the aggregate. If aggregates are shown to be reactive, additional mitigation measures as per AASHTO R80-17 may be considered for implementation.
DEF	Avoidance of deterioration	Exposure to water or moisture during service together with exceeding a maximum allowable concrete temperature during hardening	- Implementing a suitable maximum temperature (based on details of the concrete mix design) during hardening of the concrete.	ACI 301 [26] includes requirements to the maximum temperature during placement for mass placements and precast concrete. However, it is noted that DEF may also affect non-massive placements if the maximum allowable temperature is exceeded. ACI 201.2R-16 provides recommendations on measures to reduce the potential for DEF in concrete exposed to elevated temperatures at early ages.	The maximum temperature during hardening may be subject to measurements during casting by means of cast-in thermocouples. Thermocouples should be located at a depth where the anticipated maximum temperature is to occur.

Table 4-6: Mitigation Methods used for Deterioration Mechanisms affecting Reinforced Concrete.

Deterioration Mechanism	Available Durability Strategy	Contributing Exposure Situation(s)	General Mitigation Methods	Potential Code and Standard Provisions	Potential Test Methods
Sulfate Attack	Avoidance of deterioration or Deemed-to-satisfy	Exposure to sulfate (SO_4^{2-}) in soil, ground-water, or other water sources	- Using Portland cement with a low alkali content and C_3A-content (sulfate resistant cement, Type II or V) - Providing a concrete with low permeability and a low water-cement ratio - Using supplementary cementitious materials.	Requirements according to ACI 318-14: - For concrete classified as S1: Maximum water-cement ratio of 0.50 and a minimum compressive strength of 4,000 psi (28 MPa). ASTM C150 Type II cement is allowed. Types I and III are also allowed if the C_3A-content is less than 8%. - For concrete classified as S2: Maximum water-cement ratio of 0.45 and a minimum compressive strength of 4,500 psi (31 MPa). ASTM C150 Type V cement is allowed. Types I and III are also allowed if the C_3A-content is less than 5%. - For concrete classified as S3: Maximum water-cement ratio of 0.45 and a minimum compressive strength of 4,500 psi (31 MPa). ASTM C150 Type V cement plus pozzolan or slag cement is allowed.	Limits on cementitious materials as per ACI 318-14 may be considered. Alternative combinations of cementitious materials are permitted per ACI 318-14 when sulfate resistance testing per ASTM C 1012 [27] meet given exposure-specific criteria in ACI 318-14.
Carbonation-induced Corrosion	Full probabilistic, Partial safety factor, or Deemed-to-satisfy	Exposure to atmospheric CO_2	Using low concrete permeability and adequate concrete cover. Typically, it is assumed that measures (i.e. cover and concrete requirements) to resist chloride-induced reinforcement corrosion are sufficient to address carbonation-induced corrosion	Concrete cover thickness requirements and requirements to concrete mix design (e.g., w/c ratio) indirectly address carbonation-induced reinforcement corrosion in a deemed-to-satisfy approach. Full probabilistic and partial safety factor method models for carbonation-induced reinforcement corrosion are also available in *fib* Bulletin 34.	Tests for evaluation of carbonation depth include: - NT Build 357 [28] - RILEM CPC-18 [29] - An accelerated method for the inverse effective carbonation resistance of concrete as described in *fib* Bulletin 34.
Chloride-induced Corrosion	Full probabilistic approach following *fib* Bulletin 34, or Deemed-to-satisfy	Exposure to chloride and O_2	- Using low permeability concrete - Using adequate concrete cover - Using corrosion-resistant reinforcing - Using proper control of cracking per applicable structural design code and Construction Specifications.	A full probabilistic model for chloride-induced reinforcement corrosion are available in *fib* Bulletin 34. Table 4-7 provides recommendations on the input parameters for the full probabilistic *fib* Bulletin 34 calculations. Appendix B and C provide overviews of obtainable chloride migration coefficient values and experimentally measured critical chloride contents for various reinforcement steel types. Deemed-to-satisfy cover requirements from design code provisions may also be considered; however as discussed in Section 2.5.2.3, this approach may not be reliable.	Performance testing of the concrete is required using the full probabilistic approach including: - NT Build 492 [13] at a concrete age of 28 days. - Water-soluble chloride (ASTM C1218 [14]) or acid-soluble chloride (ASTM C1152 [15]). Test criteria will be determined by the modeling.

Table 4-7: Recommendations to Input Parameters for Full Probabilistic Modeling of Chloride-induced Reinforcement Corrosion.

Variable	Symbol	Short description	Recommendations to input parameters	Statistical Distribution	Unit
Cover	a	Concrete thickness measured from concrete surface to the surface of the outermost steel reinforcement.	*fib* Bulletin 34 recommends that the distribution function for large cover depths be typically chosen as a normal distribution whereas for small cover depths, distributions excluding negative values should be chosen, such as the lognormal function. Starting point for cover thicknesses would typically be taken from requirements from controlling design codes (e.g., AASHTO LRFD or local code). The standard deviation of cover, σ is typically selected as a function of the permissible cover tolerance, Δa, as follows: $\sigma = \Delta a / 1.645$ meaning that approximately 95% of the model-considered cover thicknesses would be greater than the specified cover thickness minus the tolerance.	Normal (larger covers) Lognormal (lower cover to avoid negative values)	mm
Temperature	T_{real}	Temperature of the structural element or the ambient air.	*fib* Bulletin 34 recommends that T_{real} can be determined by using available data from a weather station nearby the structure. See Section 4.2.1 for additional information on this topic.	Normal	°C
Initial chloride concentration	C_o	Initial chloride content in concrete at time t = 0.	*fib* Bulletin 34 states that the initial chloride content in the concrete is not only caused by chloride ingress from the surface, but can also be due to chloride contaminated aggregates, cements or water used for the concrete production.	Deterministic	Mass-% of total cementitious materials
Surface concentration	$C_{s,\Delta x}$	Chloride content at the depth Δx.	*fib* Bulletin 34 states that it depends on material properties and on geometrical and environmental conditions. Ideally, data is gathered from similar structures. See Section 4.2.1 for additional information on this topic.	Lognormal	Mass-% of total cementitious materials
Chloride migration coefficient	$D_{RCM,0}$	Chloride migration coefficient measured from NT Build 492 at t = 28 days.	*fib* Bulletin 34 recommends the standard deviation of the chloride migration coefficient to be 0.2 times the mean value. The mean value is assumed in the model such that the desired reliability index is obtained.	Normal	x 10^{-12} m^2/s
Ageing factor	α	The age factor describes the time-dependent change of the migration coefficient as concrete matures.	*fib* Bulletin 34 and *fib* Bulletin 76 recommend ageing factors for concrete with an equivalent water-cement ratio between 0.40-0.60.	Beta	-
Transfer function	Δx	Capillary action leads to a rapid transport of chlorides into the concrete up to a depth Δx from the surface. Beyond this depth, chloride ingress is controlled by diffusion.	*fib* Bulletin 34 recommends the following values for the transfer function: - For water level, direct and indirect de-icing salts zones: beta distribution with a mean value of 8.9 mm, standard deviation of 5.6 mm with parameter a = 0.0 and b = 50.0. - For buried, submerged, and atmospheric zones: deterministic value of 0.	Beta	mm

Table 4-7: Recommendations to Input Parameters for Full Probabilistic Modeling of Chloride-induced Reinforcement Corrosion.

Variable	Symbol	Short description	Recommendations to input parameters	Statistical Distribution	Unit
Critical chloride concentration	C_{crit}	Concentration required to break down the passive layer protecting the steel reinforcement.	*fib* Bulletin 34 recommends using a beta distribution with a mean value of 0.6% by mass of cementitious materials (based on uncoated carbon steel reinforcement), a standard deviation of 0.15, a lower bound of 0.2, and an upper bound of 2.0. Additional literature on critical chloride thresholds are presented in Appendix C.	Beta	Mass-% of total cementitious materials
Transfer parameter	k_t	-	*fib* Bulletin 34 assumes k_t as a constant value equal to 1.	Deterministic	-
Regression variable	b_e	-	*fib* Bulletin 34 recommends using a normal distribution with a mean value of 4,800K and a standard deviation of 700K.	Normal	K
Reference time	t_0	-	*fib* Bulletin 34 assumes t_0 as a constant value equal to 28 days = 0.0767 years.	Deterministic	years
Standard test temperature	T_{ref}	-	*fib* Bulletin 34 defines T_{ref} to be constant with a value of 293K = 20°C.	Deterministic	°C

4.4 Steel Components

The primary deterioration mechanism for steel and corrosion may occur in various forms including pitting, crevice, and galvanic corrosion. Further detail on the fundamentals of these processes is provided in the SHRP2 R19A Academic Toolbox. Table 4-8 provides guidance on mitigation of corrosion in structural steel using the following basic mitigation methods:

- Protective coatings
- Corrosion allowance
- Use of special steel alloys
- Cathodic protection
- Encasement in concrete

Example 1 in Appendix D provides an example Service Life Design Report that includes considerations of structural steel.

Table 4-8: Mitigation Methods used for Deterioration Mechanisms affecting Steel Components.

Deterioration Mechanism	General Mitigation Methods	Considerations
Steel corrosion	Protective coatings	Coatings provide a barrier to structural steel to protect the member against moisture and chloride. Coatings may include paint systems, epoxy, and galvanization. Corrosion of the steelwork will be eliminated if the coating is properly maintained. The service life verification of elements with protective coatings is therefore driven by the service life of the paint system, epoxy and/or galvanization. Reference with estimated service life of protective coatings include the NACE paper 7422 [16] and ISO 12944-2 [20]. Anticipated frequency of inspection and maintenance of coatings shall be considered as part of the design process.
	Corrosion allowance (increased steel area)	For uncoated weathering steel a certain amount of surface metal will be lost due to corrosion over time. A reasonably conservative corrosion allowance should be determined, based on the exposure conditions as describe in Section 4.2.2.

Table 4-8: Mitigation Methods used for Deterioration Mechanisms affecting Steel Components.

Deterioration Mechanism	General Mitigation Methods	Considerations
		Numerous references are available for the estimation of corrosion allowances including EN 1993-5 [17], FWHA Manual [18], ASTM G101, ISO 9223 [30], and FDOT Structures Design Guidelines – Section 3.1 [19].
	Use of special steel alloys	For particular environmental exposure conditions (e.g., presence of chloride ions from de-icing salts or marine environments) or specific element where corrosion allowances cannot be permitted, alternative steel alloys may be used with sufficient resistance to the prevailing exposure. One such example is a martensitic stainless steel as described in ASTM A1010 [31], which was utilized in an example structure in Iowa as part of the SHRP2 R19A project. Details on this application are provided on the SHRP2 R19A website: http://shrp2.transportation.org/Pages/Service-LifeDesignforBridges.aspx
	Cathodic protection	Standards are available on the use of either sacrificial anode or impressed current cathodic protection systems for structural steel including: - ISO 12473 - ISO 13174 - EN 12495 - DNVGL-RP-B401 It is noted that cathodic protection systems are typically more maintenance intensive than the other described mitigation methods, and the anticipated needs for inspection and maintenance should be covered as part of the design phase. Further, consideration of stray-current induced corrosion of reinforcement of steel items not subject to the protection of the cathodic protection system should be subject to project-specific consideration by an expert in the topic of stray-current induced corrosion.
	Concrete encasement	Encasement of structural steel in concrete may be considered. The minimum cover thickness of concrete and performance requirements to the

Table 4-8: Mitigation Methods used for Deterioration Mechanisms affecting Steel Components.

Deterioration Mechanism	General Mitigation Methods	Considerations
		concrete should be determined as described in Section 4.3 with additional consideration of the potential deterioration mechanisms to the concrete itself (e.g., sulfate attack, DEF, and AAR).

5.0 Guidance on Implementing Service Life Design During Construction and Documentation of the As-Built Service Life

As shown in Figure 2-1 and Table 2-3, specific elements of the service life design process developed and/or used during the construction process include:

- Construction specifications
- Maintenance plans
- Inspection/Monitoring plans
- Inspection of the construction, resulting in the (optional) Birth Certificate

Table 2-3 provides the basic aims and goals of the specific tools during this phase and further guidance on Construction Specifications is provided in this section.

Construction Specifications, and the quality control (QC) measures contained therein, are the tools to transfer key conclusions of a service life design to the jobsite. Key details including the test methods, acceptance limits, minimum testing frequencies, and procedures in case of non-conforming results should be clearly stipulated in the Construction Specifications. It is recommended that engineers with relevant specialist knowledge in concrete materials and steel coating systems prepare the associated Construction Specifications.

QC and quality assurance (QA) during construction are essential to achieve the service life requirements. QC is all activity carried out by the Contractor to ensure that the work complies with the Construction Specifications, while QA is all activity carried out by the Owner to verify that the Contractor's QC process is effective. For concrete structures, this process typically consists of two phases:

- Prequalification phase
- Production and construction phase.

Proactive QC by the Contractor is vital to limit non-conformances. It is a good practice to assign a designated person on the QC team to promptly review key service life-related parameters, like cover measurements and chloride migration coefficient results. This review should involve observing trends in the test results of a particular concrete mix design (in the case of chloride migration coefficient), to identify adverse trends or non-conformances. Early corrective action, when adverse trends or non-conformances are observed, helps to prevent further non-conformances.

Other factors that influence service life of reinforced concrete and structural steel should be subjected to rigorous QC, including for example:

- Placement, consolidation, finishing, and curing procedures for concrete
- Surface preparation, application procedures, weather conditions and monitoring procedures for coatings of structural steel

The QC and QA of these operations should be described in Construction Specifications and, if applicable, a project Quality Management Plan. These documents should also describe responsible parties for the review of inspection and test reports during production, the tracking of test results, and procedures for acceptance.

In case of a non-conformance in a material, procedure or other topic covered by a Construction Specification, remedial action to address the specific non-conformance should be proposed and agreed. It should be considered whether the non-conformance amounts to a local anomaly (e.g., one rebar with a minor misalignment) versus a situation that reduces the service life (e.g., an entire reinforcement cage being out of specification for cover). Remedial actions to consider when addressing a specific non-conformance related to service life may include:

- Re-evaluation of the actual exposure condition and achieved material properties
- Consideration if the intent of the service life requirements is still achieved despite the non-conformance (e.g. local anomaly versus situation that reduces service life)
- Remediation required (if any) and what are the controlling parameters for the remediation to reinstate the service life requirements

Further, corrective action(s) should be taken to prevent repetition of non-conformances. Further consequence of the non-conformance should be considered and appropriately documented. For example, if non-conformance impacts the expectations for maintenance and repair of the given component, this should be reflected in the Maintenance Plans.

The Owner should also implement an effective QA program to verify that the contractor's QC measures are effective in identifying and addressing non-conformances in materials and procedures.

In addition to the above general guidance, additional guidance is provided for concrete and steel components in Sections 5.1 and 5.2, respectively.

5.1 Concrete Components

An example 'Construction Specification' for concrete components is provided in Appendix E. The example provided therein is a supplementary specification, in that it is not a complete standalone specification. Rather, the example relies on general requirements from AASHTO’s LRFD Bridge Construction Specifications, 4th Edition (2017). The example supplementary specification builds from the Example 2 Service Life Design Report in Appendix D.

For projects with extended service life requirements, common standard concrete mix designs may not be sufficiently optimized for consideration of service life-related requirements. The chloride migration coefficient results compiled in Appendix B illustrate this concern, wherein the tested concrete mix design by various states involved in the SHRP2 R19A project appeared to under-perform compared to similar concrete mix design (i.e., mixes with similar binder composition and w/c ratio) from other data sources. Due, in part, to the need for utilizing special concrete mix designs specifically developed for a project, a two-step approach is typically implemented. In this approach, potentially suitable concrete mix designs are evaluated in a prequalification step, followed by requirements to the production/construction phase. This process may present a new challenge during the implementation of service life design due to the additional requirements to the concrete, which may not yet be part of the local common practice. Experience should be gained on the performance of the locally available concrete mix designs through the described prequalification step and only concrete mix designs that can simultaneously achieve all requirements should be considered for eventual use on the project.

During the prequalification step, the following steps may be taken:

- Material properties of the concrete mix constituents (aggregates, cementitious materials, admixtures, mix designs) are reviewed to verify that what is proposed will meet the requirements of the project,
- Review of data sheets, mill reports, aggregates source reports, etc. to verify the materials comply with the Construction Specification,
- Conduct testing of constituent material properties, if test data is missing,
- Upon confirmation that the constituent materials satisfy the requirements, a series of laboratory trial mixtures can be completed, using one or more of the proposed cementitious material combinations and appropriate testing is done to demonstrate that all requirements are met, including for example:
  - Compressive strength requirements,
  - Chloride migration coefficient requirements – NT Build 492,

- Water-soluble chloride (ASTM C1218) or acid-soluble chloride (ASTM C1152) content of concrete,
- Air void spacing factor – ASTM C457,
- Freeze-thaw and scaling resistance – ASTM C666 and C672

Further, it is noted that certain test requirements can take several months, and potentially up to a year for the evaluation of alkali-aggregate reactivity of aggregates if no prior test data is available. Due to the time demand, it would be preferable to qualify existing concrete mix designs and constituent materials for use on a 'standard' bridge. Qualification of existing mixes, in terms of durability-related properties and practicality of placement, should be started as soon as the concrete specification is issued. In some cases, particularly for aggregates and other constituent materials, early testing can be performed as soon as test methods and acceptance criteria are specified by the designer. This qualification is an important step in the construction planning and must be started early due to the duration of certain required tests.

Laboratories should be familiar with the relevant standard test methods and shall possess the required equipment. It may be necessary and recommended, depending on the project locations and experience of the local laboratories, to conduct any special qualification testing (e.g. NT Build 492 testing) at a different facility that is known to have experience in the test method. Assurance that tests are properly conducted may avoid false negatives, possibly saving time and cost of developing a project-specific concrete mix design.

The prequalification step should result in concrete mix designs that can achieve all requirements simultaneously, and commonly a mock-up or full-scale trial testing is completed at the end of the process. Mock-ups may be included as part of Owner's requirements, if so desired.

The extent and frequency of testing during the prequalification phase may not be appropriate for the production/construction phase, and therefore the Construction Specification should also stipulate requirements for this phase. During construction, monitoring of the key properties should be done by testing samples obtained from production concrete. Tested parameters and associated frequencies should be determined on a project-specific basis and may include:

- Compressive strength
- Plastic and/or hardened air content
- Chloride migration coefficient
- Concrete cover thickness

Production-phase requirements should also cover other service life-related topics including pre-pour QC checks of dimensional tolerances including cover thickness. It is noted that test methods exist for measurement of hardened concrete cover over embedded reinforcement, using cover-meters, which may be included as part of the Construction Specification for spot checking of the final as-built cover thickness. Section 8.19 in Appendix E (Supplementary Concrete Specification Example) includes such a provision.

Finally, Construction Specifications may include material and construction requirements for concrete repairs if needed. Repair materials for typical anticipated defects (e.g., honeycombing or surface pores; filling of voids from temporary anchors, inserts or boxouts; cracks) should comply with the intent of the service life design process. Therefore, certain service life-related requirements to concrete may be necessary to be applied to repair materials.

5.2 Steel Components

For steel components, myriad standard specifications are available on coatings, steel types, and other topics. Typically, a specification for coatings would include requirements to the following:

- Materials
- Construction, including:
  - Cleaning and pretreatment of the surface (e.g., sandblasting)
  - Application of the coating (e.g., painting process, hot-dipping or other galvanization process)
- Testing and inspection both during construction and final inspection after erection, including:
  - Application procedures including weather conditions and coating procedure
  - Evaluation of achieved dry film thickness of individual layers and the total coating systems
  - Pinholes, holidays, and pore density of the coating
  - Adhesion
  - Repair of damages

Various standards are available for structural steel coatings and surface preparation, including but not limited to the follow list of typically references standards:

- EN ISO 12944 series standards
- EN ISO 1461 [32]
- ISO 2063 Parts 1 [33] and 2 [34]
- ISO 4628-3 [35]
- ASTM A123 [36]
- ASTM A153 [37]

- SSPC SP 1 to SP 16

6.0 Guidance for the Operation (Monitoring, Inspection, and Maintenance)

As mentioned in Table 2-3, plans for maintenance, inspection, and monitoring are developed to ensure assumptions of the service life design are continually fulfilled and corrective actions (e.g., cyclical or condition based maintenance) be carried out in a timely manner to preserve and delay deterioration of the bridge. Periodic inspection and corresponding maintenance of a bridge is therefore important that minor damages are proactively repaired, preventing the possible development of more critical damage. Deferring planned inspection and maintenance works may result in situations outside the consideration of the design and could potentially result in accelerated deterioration of a component or the overall bridge.

In general, easily accessible bridge components will be easier to inspect, maintain, and will ultimately stand a better chance of reaching its intended service life. It is important to consider the accessibility of replaceable and non-replacement elements for the foreseen inspection and maintenance practices necessary to meet service life requirements. For major or signature bridge projects, it is common to include numerous inspection and maintenance facilities including under-deck travelers, gantries, walkways, and rope anchors as part of a coordinated access and inspection plan to ensure all surfaces are accessible. Additionally, the designer will typically be required to produce an operation and maintenance manual with detailed descriptions of all anticipated maintenance procedures. Similarly, it is important to consider, to an appropriate level of detail, the accessibility and maintenance of components on a 'standard' bridge.

The interface between the structural designer and inspection and maintenance specialists is of increased importance on Public-Private Partnership (P3) projects. As discussed in Sections 2.6 and 3.2, the Owner may include conditional requirements for the end of the operating period (the handback date). During the design phase, the designer must coordinate with the inspection and maintenance specialists to understand the inspection system that will be used and type of maintenance that will be performed. This understanding will help the designer optimize the design so that it will meet the handback criteria.

For future verification and updating of the service life prognosis of a bridge, more in-depth monitoring inspections of reinforced concrete elements exposed to chloride is recommended and may be considered at a relatively infrequent interval (e.g., every 10 years). Concrete cores may be extracted from locations in the actual structure, representative of the various exposure zones identified during the design process, and chloride profiling could be evaluated for comparison of the actual service life performance against the designed situation. Controlling parameters, including surface chloride concentration, apparent chloride migration coefficient, and the chloride profile itself would provide insight on the remaining service life of the reinforced concrete elements. Note, in case coring of the actual structure is not desirable, non-structural exposure samples could be considered for this purpose. However, such samples shall be constructed from the same concrete and in accordance with the same Construction Specification as the structural concrete.

7.0 References

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