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SECTION 1: INTRODUCTION

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INTRODUCTION

1.1—BACKGROUND

Recent shifts in expectations regarding long term performance of bridges have focused attention on the design of bridges to meet specific service lives. This has resulted in a call to expand traditional limit states to include a focus on durability in addition to the strength design and serviceability requirements already contained in the *AASHTO LRFD Bridge Design Specifications*. In North American structural design codes, durability-focused limit states are not explicitly identified and are addressed primarily through “deemed-to-satisfy” requirements. Over the last few decades, research has been conducted to identify specific durability limit states and to develop rational and quantitative approaches to consider them during the design phase. The *AASHTO Guide Specification for Service Life Design of Highway Bridges* was developed to incorporate these quantitative approaches along with proven deemed-to-satisfy provisions into a single comprehensive design document for implementation on a national level. This Guide Specification is a product of the efforts of NCHRP 12-108 and include the service life guidance contained in a number of AASHTO, FHWA, NCHRP, Strategic Highway Research Program 2 (SHRP2), and other national and state-sponsored research documents. A framework is established for service life design while providing opportunities for refinement and expansion, especially as new models capable of simulating deterioration mechanisms become available.

Key features of the Guide Specification include:

- The establishment of target service life categories
- Explicitly defined exposure classes
- Design requirements for reinforced concrete for the chloride-induced corrosion limit state calibrated using a full probabilistic method
- A novel protection index approach for foundations and retaining walls
- Case studies exemplifying the application of the Guide Specification
- A framework for using probabilistic design approaches for durability limit states on a project-specific level

C1.1

The term “shall” denotes a requirement for compliance with this Guide Specification.

The term “should” indicates a strong preference for a given criterion.

The term “may” indicates a criterion that is usable, but other local and suitably documented, verified, and approved criteria may also be used in a manner consistent with this Guide Specification.

The term “recommended” is used to give guidance based on past experiences.

1.2—OBJECTIVE

The objective of this Guide Specification is to provide practical guidance to designers and owners on design decisions that affect the durability of highway bridges. The understanding of the influence of design decisions on durability is evolving, and there are different levels of rigor that can be applied, but often the uncertainty remains large regardless of the methods used.

It is the intent of this Guide Specification to allow for the incorporation of improved deterioration and service life models as they become available.

1.3—ORGANIZATION

The subsequent sections of this Guide Specification included within are organized as follows:

- Section 2—Classification
- Section 3—General Design Guidelines
- Section 4—Concrete Structures
- Section 5—Steel Structures
- Section 6—Foundations and Retaining Walls
- Section 7—Renewable Elements
- Section 8—Life Cycle Cost Analysis
- Appendix A—Probabilistic Service Life Design Framework
- Appendix B—Case Studies

1.4—DEFINITIONS

Actual Service Life—The period of time a bridge remains in operation, without rehabilitation or significant repair, and with only routine maintenance. This includes replacement of renewable elements.

Avoidance of Deterioration—A strategy that assumes deterioration will not take place by: separating the environmental deterioration action from the affected structure/component, using non-reacting materials, separating the reactants from the structure/components, or suppressing the reaction.

Bridge Preservation—Actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements, restore the function of existing bridges, keep bridges in good condition, and extend their life. Preservation actions may be preventative or condition-driven.

Deemed-to-Satisfy—A strategy that is based on a set of rules related to dimensioning, material selection, and execution. The set of design rules is based on either statistical evaluation of experimental data or long-term experience. Where feasible, calibration using probabilistic methods will be considered.

Design Life—Period of time on which the statistical derivation of transient load is based: 75 years for the *AASHTO LRFD Bridge Design Specifications*.

Durability—A series of properties required of a structure in order to resist the effects of the particular environment that the structure will be exposed to during its service life.

Exposure Class—Exposure to a specific type of deterioration mechanisms.

Exposure Zone—A set of specific environmental exposure conditions.

Full Probabilistic Design—A design method that uses models representative of the deterioration process of a particular component/material subjected to a specific action. The model parameters are determined through testing, observation, and experience.

Global Index—An overall ranking that indicates the level of aggressiveness of the ground environment.

Highest Low Water—The height of the highest low water level over a 19-year period.

Lowest Low Water—The height of the lowest low water level over a 19-year period.

Mean Higher High Water—The average height of the higher high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

Mean Lower Low Water—The average height of the lower low waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

Partial Factor Design (Semi-Probabilistic)—An application of the full probabilistic method in which uncertainties in the deterioration process are accounted for through factors applied to the action and component/material resistance.

Protection Index—An index used to select an appropriate protection strategy against deterioration for a given foundation or retaining wall system.

Renewable Element—An element designed to be replaceable within the service life of the structure.

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

1.5—NOTATION

a	=	lower bound of beta distribution (wt.-%/CM) (C4.2.4.2.2)
$AADT$	=	annual average daily traffic (C6.1)
AAR	=	alkali aggregate reaction (4.1.4)
AC	=	agency costs (8.1.1)
ACR	=	alkali carbonate reaction (C4.1.4)
ADT	=	average daily traffic (2.1.1) (C6.1)
ASR	=	alkali silica reaction (C4.1.4)
AV	=	uniform payment or equivalent average value of the expenditure (8.6.1)
BCR	=	bridge condition rating (C8.6.2)
BCV	=	opportunity cost of the bridge structure and land including decommissioning (8.6.2)
b	=	upper bound of beta distribution (wt.-%/CM) (C4.2.4.2.2)
$C_{initial}$	=	cost of initial construction (8.1.1)
CSL	=	cross-hole sonic logging (6.2.4.2.1)
$c(h_i)$	=	estimated cost associated with a hazard event h_i of intensity i (8.5.2)
DR	=	discount rate (8.6.1)
DT_C	=	average delay time per vehicle due to congestion and closure (8.5.2)
DT_D	=	average delay time per vehicle diverted (8.5.2)
d	=	exponent (C8.6.2)
FA	=	fly ash (4.2.1)
FC_n	=	average fuel consumption rate for vehicle type n (8.5.2)
FP	=	prevailing average price of fuel (8.5.2)
FRP	=	fiber-reinforced polymer (3.2.3.2)

FV_n	=	future value of an expenditure made at time N (8.6.1)
GDL	=	gamma density logging (6.2.4.2.1)
$GGBS$	=	ground granulated blast slag; ordinary portland cement with 36-65 percent ground granulated blast furnace slag grade 100 or higher by mass of total cementitious material (4.2.1) (4.2.4.2.2)
$GGBS+SF$	=	ordinary portland cement with 36-65 percent ground granulated blast furnace slag grade 100 or higher and 5-8 percent silica fume by mass of total cementitious material (4.2.4.2.2)
$HLMR$	=	high load multi-rotational (C3.2.3.3)
i	=	the set of estimated intensities (8.5.2)
$LCCA$	=	life cycle cost analysis (8.1)
MSE	=	mechanically stabilized earth (C3.2.3.6) (6.3.4)
N	=	number of time units between the present and future times (8.6.1)
NDE	=	non-destructive evaluation (3.4.1)
OPC	=	ordinary portland cement (4.2.4.2.2)
$OPCFA$	=	ordinary portland cement with 20-50 percent fly ash type F by mass of total cementitious material (4.2.4.2.2)
$OPCFA+SF$	=	ordinary portland cement with 20-50 percent fly ash type F and 5-8 percent silica fume by mass of total cementitious material (4.2.4.2.2)
P_i	=	protection index (6.1)
$PCCP$	=	portland cement concrete pavement (C6.3.4.1.4)
$PTFE$	=	Polytetrafluorethylene (7.2.1)
PV	=	present value (8.6.1)
$PV_{recurring}$	=	present value of all recurring costs over the analysis period (8.1.1)
$PV_{residual}$	=	present value of the residual value at the end of the analysis period (8.1.1)
$p(h_i)$	=	probability of hazard event h_i occurring in any single analysis period, typically expressed on an annual basis (8.5.2)
QA	=	quality assurance (6.1.1)
QC	=	quality control (6.1.1)
RC	=	estimated replacement cost for the bridge (C8.6.2)
RV	=	residual value (8.6.2)
S_n	=	average speed of vehicle n (8.5.2)
SCM	=	supplementary cementitious material (4.1.4)
SF	=	silica fume (4.2.1)
SL	=	service life (8.6.2)
T_a	=	analysis period (8.6.2)
TDC_c	=	traffic delay costs due to congestion and closure (8.5.2)
TDC_d	=	traffic delay costs due to diversion (8.5.2)
TV	=	terminal value (8.6.2)
TV_n	=	unit value of time for vehicle type n (8.5.2)
TIP	=	thermal integrity profiling (6.2.4.2.1)
$TLCC$	=	total life cycle cost (8.1.1)
tdc_n	=	delay cost per vehicle per unit time (8.5.2)
UC	=	user costs (8.1.1)
v_n	=	number of vehicles of type n delayed by the action; number of vehicles of type n diverted to another route (8.5.2)
VC	=	vulnerability costs (8.1.1)
W/CM	=	water to cement ratio (4.2.1)
μ	=	mean (wt.-%/CM) (C4.2.4.2.2)
σ	=	standard deviation (wt.-%/CM) (C4.2.4.2.2)

1.6—APPROACH

In general, the approach taken in this Guide Specification is one of “good-better-best” in terms of the design practices that can be implemented during design. For each subdivision of this Guide Specification, design provisions to provide each of the three levels were developed based on the available literature as well as the use of full probabilistic approaches where applicable.

C1.6

The strength I limit state calibration of the *AASHTO LRFD Bridge Design Specifications* for a reliability index of about 3.5 was predicated on little or no loss of resistance with time. Subsequently, the calibration of the *Manual for Bridge Evaluation* allowed for an in-service reliability index of about 2.5 for the operating condition which could involve some reduced resistance due to

Guidance on which practices to use will be provided with factors such as corrosive environment and importance of the structure taken into account.

The primary focus of this Guide Specification is on design decisions; however, the importance of construction quality and regular maintenance in achieving a target service life must be acknowledged.

The intended use of this Guide Specification is for the design of typical slab and girder highway bridges. However, sections of this Guide Specification that are applicable to a wider range of bridge types can optionally be applied to bridges other than slab and girder highway bridges, at the discretion of the Designer.

This Guide Specification uses all available methods of designing for durability; however, two design methods are primarily used: deemed-to-satisfy and avoidance-of-deterioration. Where applicable, the full probabilistic approach is used to calibrate deemed-to-satisfy provisions. Guidance as to the implementation of full probabilistic methods for the design of concrete structures subject to chloride-induced corrosion is provided in Appendix A.

deterioration or loads heavier than the original design contemplated. Recent calibration of some of the Service Load Combination (Kulicki et al., 2015) involved much lower reliability indices which were compatible with the perceived consequence of exceeding the limit states and the generally lower factored loads that apply to the service limit states, but still assumed that the resistance would be essentially preserved over time.

Good service life practices help to promote the preservation of suitable strength and service limit state reliability in the range cited above through the selection of durable materials and detailing as well as timely and effective maintenance activities. Service life practices may include those based on a reliability approach when sufficient data and verified behavior models are available for probability-based calculations, or experience-based avoidance or “deemed-to-satisfy” strategies where a probability basis cannot yet be codified.

For example, the specifications related to reinforced concrete decks could be applied to a suspension bridge, but there are no provisions for suspension bridge specific components such as cables or towers.

Experience based design and detailing provisions intended to enhance serviceability, but which do not have a mathematical basis are most often based on “avoidance” or “deemed-to-satisfy” approaches. Previous requirements for concrete cover of embedded reinforcement bars and the corrosion allowance for buried MSE wall reinforcement are examples of “deemed-to-satisfy.” Probabilistically-based requirements include fatigue design, settlement, service limit state design of prestressed components, and the reinforcing bar cover provided in Table 4.2.4.2.2-1.

Ideally, the provisions for service life design would be based on a rational analysis of the bridge’s field performance. However, multiple challenges need to be overcome to perform this rational analysis. These include the following:

- insufficient design and construction data;
- insufficient data to characterize the environmental exposure conditions;
- available data is from bridges built using obsolete design and construction practices that are not representative of current industry practices;
- data contains significant gaps during the structure's life; and
- the type and quantity of performance data gathered varies by jurisdiction, making direct performance comparisons between structures difficult.

A rigorous analysis and comparison of structures’ performance would need to consider the large number of factors influencing durability (exposure conditions, materials, geometry, design details, construction

fib Bulletin 34 (2006) and ISO 16204 (2012) define the following four generally accepted approaches to service life design:

- Full Probabilistic Method—An approach in which the following principles shall be applied:
 - Probabilistic models that are sufficiently validated to give realistic and representative results are applied
 - The parameters of the models and their associated uncertainty shall be quantifiable by means of tests, observations, and/or experience
 - Reproducible and relevant test methods shall be available to assess the action and material parameters
 - Uncertainties associated with the models and test methods shall be taken into account
- Partial Factor Method—An approach using the same models as the full probabilistic method, but based on design values. The applied actions and material resistances are separated, with each having associated partial safety factors that account for uncertainties and variabilities (e.g., load and resistance factors). It shall be verified that the target reliability index for not passing the relevant limit state is not exceeded when design values for actions or effects of actions and resistances are used in the design methods. The partial factors shall take into account:
 - the possibility of unfavorable deviations of action values from the representative values,
 - the possibility of unfavorable deviations of materials and product properties from the representative values, and

practices, and maintenance procedures) thereby increasing significantly the complexity of the analysis and, importantly, the volume of data required for statistical significance. Finally, there is no consensus in the industry on a rational methodology to be used for the durability assessment of existing structures or for service life design of new structures.

The focus on deemed-to-satisfy and avoidance approaches should not be construed as a rejection of probabilistic methods. In fact, an ideal method for channeling probabilistic approaches into more widespread use is through the calibration of deemed-to-satisfy methods using the full probabilistic approach. At present, only the combination of concrete covers, concrete transport properties, and type of reinforcing steel provisions for the chloride-induced corrosion of reinforced concrete limit states are candidates for this approach.

The four service life design approaches defined in *fib* Bulletin 34 (2006) and ISO 16204 (2012) are used for deterioration limit states of concrete structures (and embedded steel), but are applicable to the deterioration of other materials.

The definitions for each of the four service life design approaches were adopted from *fib* Bulletin 34 and ISO 16204. Modifications to these definitions, where appropriate, were made to match the methodology set forth in this Guide Specification.

- model uncertainties and dimensional variations.

The numerical values for the partial factors shall be determined in one of the following two ways:

- On the basis of statistical evaluation of experimental data and field observations according to the requirements of the full probabilistic method
 - On the basis of calibration to long-term performance of traditional design practices
- Deemed-to-Satisfy Method—A set of rules for
 - dimensioning,
 - material and product selection, and
 - execution procedures

for which the target reliability for not passing the relevant limit state during the design service life is not expected to be exceeded when the structure or component is exposed to the design situations. The specific requirements for design, materials selection, and execution shall be determined in either of two ways:

- On the basis of statistical evaluation of experimental data and field observations according to the requirements of the full probabilistic method
- On the basis of comparison to long-term performance of traditional design practices

For the purpose of this Guide Specification, deemed-to-satisfy provisions shall be assumed to adhere to the latter basis.

The limitations to the validity of the provisions shall be clearly stated.

- Avoidance-of-Deterioration—This method assumes that the deterioration process will not take place due to, for example:
 - Separation of the environmental action from the structure or component by protective systems (e.g., cladding or membranes)
 - Using nonreacting materials (e.g., certain stainless steels or nonreactive aggregates)
 - Separation of reactants (e.g., keeping the structure or component below a critical degree of moisture)

- Suppressing the harmful reaction (e.g., by electrochemical methods)

The specific requirements for design, material selection, and execution for the avoidance-of-deterioration method may in principle be determined in the same way as for the deemed-to-satisfy method. The limitations to the validity of the provisions shall be clearly stated.

1.7—FUTURE SERVICE LIFE DESIGN IMPROVEMENTS

As improved deterioration and service life models become available, they can be implemented into probability-based limit states in this Guide Specification. Both full probabilistic and semi-probabilistic (e.g., partial factor) methods have the advantage of control over the probability of failure through target reliability indices. However, significant amounts of data are required in order to develop accurate models of specific deterioration mechanisms. This is the main reason why probabilistic based service life design approaches have not seen broad implementation.

One option for future service life design is the use of performance-based design requirements. The requirements would appear to be deemed-to-satisfy provisions on the surface, but in reality would be calibrated using full probabilistic methods. As an example, performance-based concrete material resistance classes would allow for target reliabilities to be reached through calibration of concrete durability specifications. The results would be increased safety and improved life-time economy of designs.

While performance-based design requirements are a viable option for durability design approaches, there is still the lack of accurate deterioration models that must be addressed first. Further development of models will allow for broad use of probabilistic methods, either through direct implementation or as a basis for deemed-to-satisfy approaches.

1.8—REFERENCES

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SECTION 2: CLASSIFICATION

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CLASSIFICATION

2.1—TARGET SERVICE LIFE

2.1.1—Service Life Category

Design should be based on the following categories of service life:

- **Renewable**—For elements designed to be replaceable within the service life of the structure. The renewable components shall be identified by the Engineer.
- **Normal**—For nonreplaceable components of typical highway bridges.
- **Enhanced**—For nonreplaceable components of bridges with either high initial capital cost, high ADT, significant social context, high consequences of serviceability failure, or other criteria warranting an increased service life as determined by the Owner.
- **Maximum**—For nonreplaceable components of bridges with either higher initial capital cost, higher ADT, critical social context, unacceptable consequences of serviceability failure, or other criteria warranting a maximum service life as determined by the Owner.

The Owner shall specify the service life categories applicable to the project. The service life category assigned to each component shall be documented in the Service Life Design Report, as defined in Article 3.4.1.

This Guide Specification is centered around a three-tiered approach of qualitative levels of practice: good-better-best. The levels of qualitative practice are correlated to the service life categories, as shown in Table 2.1.1-1. For subsequent sections, design provisions to meet each of the three levels are presented.

C2.1.1

The target service life is based on two main categories of bridge components: renewable (or replaceable components) and nonrenewable (or nonreplaceable components).

Renewable components are components that are designed for replacement. These typically include joints, bearings, drainage systems, deck wearing surfaces, access components, barriers, railings, etc.

Nonrenewable components are major structural components that are permanent and not designed to be replaced within the service life of the structure. These typically include components of foundations (piles, pile caps, footings, soil, and rock anchors), substructures (piers, pier caps, abutments, retaining walls, mechanically stabilized earth structures) and superstructures (main structural members).

Decks may be designed to be either renewable or nonrenewable. See Article C3.2.2.2 for guidance.

At this time, the methodology for service life design is not sufficiently developed to provide specific year values for service life. See Article C1.6 for more detail.

The severity of the environment has a direct effect on the service life. As the aggressivity of the environment increases, achievement of longer service lives may not be economically feasible. For example, use of the “better” level of qualitative practice in combination with a highly aggressive environment may result in a reduced service life compared to the same level in a less aggressive environment.

Table 2.1.1-1—Definition of Service Life Categories

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

2.1.2—Renewable Elements

2.1.2.1—Joints and Bearings

Guidance on anticipated service lives of various joint and bearing systems is provided in Section 7. When selecting a joint system, the Designer should consider factors that affect joint performance such as environment and ADT, in addition to the required movement capacity of the joint. When setting a target service life for bearings, consideration should be given to the joint condition above the bearing (i.e., with a joint versus jointless).

2.1.2.2—Barriers and Railings

The target service life for barriers and railings should be selected based on the type of connection to the deck or superstructure. Barriers and railings integral with the deck should have a target service life equal to the deck target service life. Barriers and railings with connections to the deck or superstructure that allow for replacement may be designed for a shorter target service life in comparison to the deck.

2.2—ENVIRONMENTAL CLASSIFICATION

2.2.1—Exposure Zones

Bridge environmental classification should take into account both the macro-environmental exposure conditions at the bridge site and the micro-environmental conditions specific to each bridge component, or bridge element exposure zones.

2.2.1.1—Macro Environment Exposure Zones

Macro environmental considerations shall include proximity to the coast, the local area (urban vs. rural), and climate conditions. The following macro environment exposure zones may be assigned to the structure:

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

C2.1.2.1

The service lives of joints vary depending on the joint system employed, while the service lives of bearings vary depending on the type of bearing as well as the existence and condition of the joint above. Therefore, it is impractical to assign a single target service life value for all joint systems and bearing types. Guidance on the design and selection of joint systems is provided in NCHRP (2003).

C2.2.1.1

Additional guidance on macro environment exposure zones for steel is included in SSPC (1996) and FHWA (2015).

The limiting distance from a coast that constitutes a marine environment varies depending on conditions such as wind and other weather patterns. Some transportation agencies provide guidance on the limiting distance in their specifications. Examples are provided in AS (2004),

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

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

2.2.1.2—Micro Environment Exposure Zones

The following exposure zones may be assigned to bridge elements and regions of elements depending on the condition of surrounding environment and the geometry of the structure. An illustration of exposure zones and approximate limits is shown in Figure 2.2.1.2-1. Exposure zones should be evaluated on a project-specific basis and for each bridge element individually.

- Buried Zone: Zone permanently buried in soil.
- Submerged Zone: Zone permanently submerged in water, below the tidal zone or water-level zone.
- Tidal Zone or Water-Level Zone: Zone not permanently submerged in water, subject to wet-dry cycles. Any portion of the structure between Mean Lower Low Water (MLLW) and Mean Higher High Water (MHHW). For areas with minimal tides, portion of the structure between Mean Sea Level (MSL) and Mean High Water (MHW). For structures not in a tidal environment but where water level can vary, the water level zone is the portion between low water level and high-water level.
- Direct deicing salts zone: zone directly exposed to the use of deicing salts.
- Indirect Deicing Salts Zone, Splash Zone, Spray Zone: Zone indirectly exposed to deicing salt whether through roadway spray and/or roadway splash. For roadway splash/spray, the zone extends horizontally a distance x from the edge of the roadway and vertically a distance y from the roadway surface, as shown in Figure 2.2.1.2-2. In lieu of other guidance, x and y may be taken as 30 feet and 20 feet, respectively. Splash and spray zone also exists above the tidal zone and is subject to splash and spray from waves. See Figure 2.2.1.2-1 for limits of splash/spray zone in a tidal environment. All elements that would be subject to splash, spray, or runoff due to joint failure should be considered in the indirect deicing salts zone, splash zone, or spray zone at a minimum.

Caltrans (2014), ODOT (2016), FDOT (2017), and NYSDOT (2017).

C2.2.1.2

Bridge element exposure zones are used in the subsequent classification of exposure class in Article 2.2.2. The existence and limits of specific exposure zones will vary by location, and Figure 2.2.1.2-1 provides general definitions and limits.

Bridge elements typically in the buried zone or submerged zone are foundation and substructure elements such as piles, footings, abutments.

A tidal zone is subject to tidal effects. The limits of a tidal zone are adapted from West et. al. (1999); NACE (2012); and UFGS (2012). The water-level zone is intended for structures in an environment where the water level can vary between a low-water level and high-water level (e.g., a river or lake).

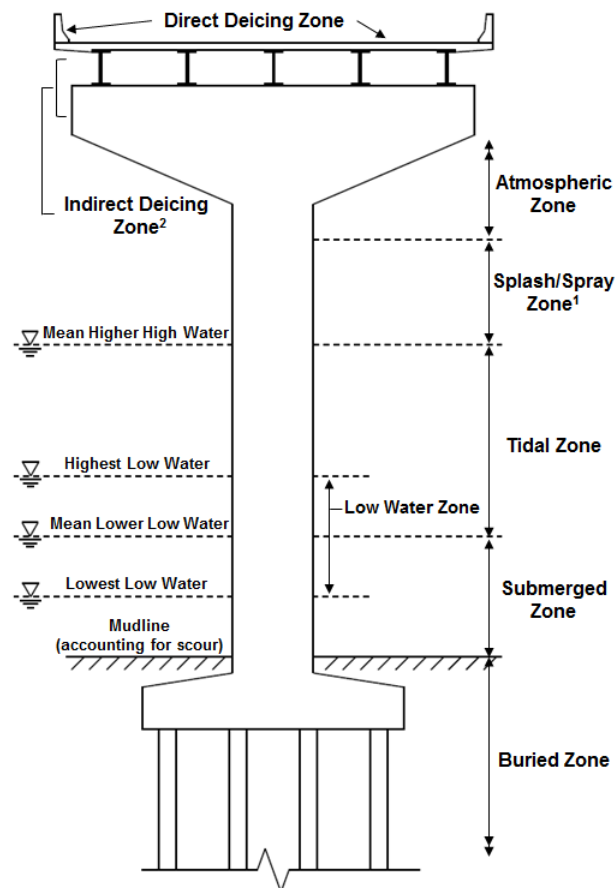
For steel piles in a marine environment, a low water zone is often defined within the bottom of the tidal zone and top of the submerged zone between the lowest low water and highest low water, with elevated corrosion rates reported (Morley and Bruce, 1983) (Tomlinson and Woodward, 2008) (Hannigan et. al., 2016). See Article 6.2.2.2.1 for guidance.

Elements typically in the direct deicing salts zone are decks, deck fascias, curbs, sidewalks, medians, and barriers.

Elements subject to splash, spray, runoff, or snow laden with salts are considered to be in the splash zone. Splash zones exist in portions of bridges over bodies of salt water and in regions that receive splash and spray from traffic (FHWA, 2015). The lateral and vertical distances from roadway surface that define the splash zone due to traffic vary depending on the governing agency; guidance is provided in CSA (2014), EN 1992-2 (2005), and *fib* (2006). For the purpose of defining exposure zones, it should be assumed during design that all joints, if present, will fail at some point during their service life.

For structures in a marine environment, the splash zone extends a vertical distance from high water/high tide and a horizontal distance from the edge of a shore. Additional guidance is provided in UFGS (2012), Caltrans (2014), and DNV (2014).

- Atmospheric Zone: Zone not exposed to soil, water, or deicing salts. Portion of the structure above the splash/spray zone when in a tidal environment.

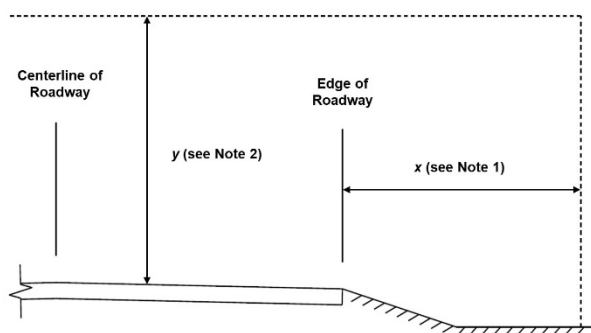


Notes:

¹For unprotected locations, the 20 feet area above the tidal zone (UFGS, 2012) (Caltrans, 2014). For locations protected by seawalls or otherwise sheltered from open-ocean waves, 6 feet area above tidal zone (UFGS, 2012).

²If subject to splash/spray/runoff due to joint failure.

Figure 2.2.1.2-1—Micro Environment Exposure Zones



Notes:

¹x may be taken as 33 feet in absence of other guidance

²y may be taken as 20 feet in absence of other guidance

Figure 2.2.1.2-2—Roadway Splash/Spray Zone Limit

Figure 2.2.1.2-1 is adapted from figures and limits from Morley and Bruce (1983), West et al. (1999), Caltrans (2010), UFGS (2012), and Hannigan et al. (2016).

The values used for “x” and “y” in Figure 2.2.1.2-2 are adopted from EN 1992-2 (2005) and CSA (2014). EN 1992-2 recommends values of 6 meters for x and 6 meters for y . CSA recommends 10 meters for x and 3 meters for y . The recommended values of 30 feet and 20 feet for x and y , respectively, represent an envelope of the EN 1992-2 and CSA recommendations and are conservative. Other examples of the roadway splash/spray zone limits are provided in *fib* (2006), FHWA (2015), PennDOT (2016), and VDOT (2016).

2.2.2—Exposure Classes

2.2.2.1—Concrete Structures

Exposure classes for reinforced concrete are grouped by their associated deterioration mechanisms. The following are the most common deterioration mechanisms associated with reinforced concrete:

- Chloride-induced corrosion
- Carbonation-induced corrosion
- Freeze–thaw attack
- Alkali–aggregate reaction
- Sulfate attack
- Abrasion
- Preservice cracking

Based on the exposure zones defined in Article 2.2.1 and the deterioration mechanisms above, exposure classes are assigned according to Table 2.2.2.1-1.

C2.2.2.1

Chloride-induced corrosion is generally caused by the ingress of chlorides into concrete via diffusion or through cracks of sufficient size. Once the chloride concentration at the level of the reinforcing steel reaches a critical concentration, corrosion initiates.

Carbonation-induced corrosion is caused by the ingress of atmospheric carbon dioxide, which reacts with the hydrated cement paste. This reaction acts to lower the pH of the pore water and break down the passive protection layer surrounding the reinforcing steel, thus permitting corrosion to initiate.

Freeze–thaw damage in concrete is caused by the expansion (approximately 9 percent by volume) of water within moist concrete as it freezes. This expansion can induce tension stresses in excess of the tensile strength of the concrete, which results in cracking. During thaw cycles, these cracks serve as additional pathways for water to enter the concrete, thereby exacerbating damage during subsequent freeze cycles.

Alkali–aggregate reaction refers to damage that results from a reaction between the alkali hydroxide in concrete and certain forms of silica, which is commonly found in aggregates. This reaction results in the formation of a gel that expands in the presence of water, inducing tension stresses in concrete that can cause cracking.

Sulfate attack is caused by the ingress of sulfate ions that react with hydrated cement paste to form an expansive crystalline product known as ettringite. This can occur when concrete is in contact with either soil or water (e.g. seawater, ground water, sewage water) that contain sulfates.

Abrasion refers to the progressive section loss of concrete due to mechanical wear (e.g. studded tires). This section loss can lead to inadequate cover which can reduce resistance to carbonation- and chloride-induced corrosion, ride quality issues for bridge decks, and, in some cases, reduced structural capacity.

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

Concrete deterioration mechanisms and environmental considerations are discussed in greater detail in ACI 201.2R (2016) and ACI 365.1R (2017).

Table 2.2.2.1-1—Concrete Exposure Classes

Category	Class	Condition	Description
Corrosion (C)	Nonaggressive (NA) Environment		
	C-NA1	Interior exposure	Sheltered surfaces, mostly dry conditions.
	C-NA2	Other exterior exposure	
	Buried (B) Environment		
	C-B	Buried	Buried elements (deep and shallow foundations, retaining structures, etc.) below finished grade (mudline in case of submergence).
	Deicing (D) Environment		
	C-D1	Atmospheric in a deicing salts environment	Surfaces exposed to airborne chlorides. Examples: abutments, piers, pile caps, girders, underside of decks.
	C-D2	Indirect deicing salts	Surfaces exposed or potentially exposed to drainage water containing deicing salts or roadway spray. Examples: surfaces below expansion joints or drains, deck fascia, substructure surfaces near a roadway.
	C-D3	Direct deicing salts – low	Direct exposure to deicing salts with low to medium application rate of deicing salts. Examples: top of decks, curbs, sidewalks, barriers.
	C-D4	Direct deicing salts – high	Direct exposure to deicing salts with high application rate of deicing salts. Examples: top of decks, curbs, sidewalks, barriers.
	Marine (M) Environment		
	C-M1	Marine – atmospheric	Surfaces exposed to airborne chlorides. Examples: superstructure.
	C-M2	Marine – submerged	Permanently submerged with salt water present. Examples: substructures between mudline and tidal zone.
	C-M3	Marine – tidal or splash/spray zone	Surfaces in contact with salt water in the tidal zone or splash/spray zone. Examples: substructures within tidal zone or splash/spray zone.
Freezing and Thawing (FT) ¹	FT0	Not exposed to cycles of freezing and thawing	Members in climates without freezing temperatures; foundations not exposed to freezing; members buried below frost line.
	FT1	Exposed to cycles of freezing and thawing, limited exposure to water but without exposure to chlorides	Vertical surfaces exposed to water and freezing; walls and columns.
	FT2	Exposed to cycles of freezing and thawing, frequent exposure to water but without exposure to chlorides	Horizontal surfaces exposed to water and freezing.
	FT3	Exposed to cycles of freezing and thawing, frequent exposure to water and chlorides	Bridge decks exposed to deicing chemicals; surfaces exposed to direct spray containing deicing chemicals and freezing; splash zones of marine structures exposed to freezing.

Table 2.2.2.1-1 (continued)—Concrete Exposure Classes

Category	Class	Condition			Description
		Water-soluble sulfate (SO_4^{2-}) ³	In recycled aggregate (% mass)	Dissolved sulfate (SO_4^{2-}) in water (ppm) ⁴	
Sulfate (S) ^{1,2}		In soil (% mass)	In recycled aggregate (% mass)		
	S0	$\text{SO}_4^{2-} < 0.10$	$\text{SO}_4^{2-} < 0.20$	$\text{SO}_4^{2-} < 150$	Conditions where water-soluble sulfate concentration in contact with concrete is low and injurious sulfate attack is not a concern
	S1	$0.10 \leq \text{SO}_4^{2-} < 0.20$	$0.20 \leq \text{SO}_4^{2-} < 0.60$	$150 \leq \text{SO}_4^{2-} < 1,500$ or seawater	Moderate sulfate exposure; sea water exposure
	S2	$0.20 \leq \text{SO}_4^{2-} < 2.00$	$0.60 \leq \text{SO}_4^{2-} < 2.00$	$1,500 \leq \text{SO}_4^{2-} < 10,000$	Severe sulfate exposure
	S3	$\text{SO}_4^{2-} > 2.00$	$\text{SO}_4^{2-} > 2.00$	$\text{SO}_4^{2-} > 10,000$	Very severe sulfate exposure
In contact with water (W) ¹	W0	Concrete dry in service; concrete in contact with water and low permeability is not required			Dry in service or in contact with water but no specific requirements for low permeability
	W1	Concrete in contact with water and low permeability is required			Need for concrete with low permeability to water; penetration of water into concrete might reduce the durability of the member

Notes:

¹Adopted from ACI (2014)²Adopted from CSA (2009)³Percent sulfate by mass in soil shall be determined by ASTM C1580⁴Concentration of dissolved sulfates in water, in ppm, shall be determined by ASTM D516 or ASTM D4130**2.2.2.2—Steel Structures**

Exposure classes for steel are grouped by their associated deterioration mechanisms. The most common deterioration mechanisms associated with structural steel are corrosion and fatigue.

Based on the exposure zones defined in Article 2.2.1 and the deterioration mechanism above, exposure classes are assigned according to Table 2.2.2.2-1.

C2.2.2.2

Corrosion is the main cause of steel bridge deterioration and hence the determining factor for service life.

Fatigue in metals is the process of initiation and growth of cracks under action of repetitive tensile loading cycles.

While ISO (2017) defines exposure Classes C1 through C5I/C5M for corrosion, not all classes have been adopted (Helsel and Lanterman, 2018). See Section 5 for more detail.

Table 2.2.2.2-1—Steel Exposure Classes

Category	Class	Condition	Description
Corrosion (C) ¹	C1	Very low	
	C2	Low	Atmospheres with low level of pollution, mostly rural areas
	C3	Medium	Urban and industrial atmospheres, moderate sulphur dioxide pollution, coastal areas with low salinity
	C4	High	Industrial areas and coastal areas with moderate salinity
	C5I	Very High Industrial	Industrial areas with high humidity and aggressive atmosphere
	C5M	Very High Marine	Offshore areas with high salinity and sub-tropical and tropical atmospheres
Fatigue (F)	See Article 6.6 of <i>AASHTO LRFD Bridge Design Specifications</i>		

Notes:

¹Adopted from ISO (2017)

2.2.2.3—Foundations and Retaining Walls

See Section 6 for exposure classes for foundation and retaining wall elements.

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GENERAL DESIGN GUIDELINES

3.1—SERVICE LIFE CONSIDERATIONS IN THE DESIGN PROCESS

The design of a bridge for a specific service life involves the consideration of a broad range of factors. Decisions at all stages of the design process have an impact on the durability of the structure. These include:

- planning decisions on the siting and general structural arrangement;
- preliminary design decisions such as span layout and number and location of joints—the latter is perhaps the single most critical decision regarding durability;
- design details, particularly the drainage systems; and
- material selection.

This Section addresses some of the general design aspects of service life design and includes examples of common durability issues that can develop based on design decisions.

3.2—DESIGN FOR SERVICE LIFE

3.2.1—Planning

Decisions regarding which target service life is selected for the bridge, and which methods are most appropriate to achieve that target life, should be part of the project scoping stage. There are a myriad of factors that should be considered by the designer throughout the design process. Some of these factors regarding service life design are reasonably well-established, and include but are not limited to traffic volumes, geometry, clearances, construction clearances, and the deterioration environment. However, the Owner should consider other more indeterminate factors, such as but not limited to change in functionality demands over the entire service life of the bridge and future maintenance and preservation actions

Therefore, during the scoping stage of project development, considerations that bear directly on the service life and maintenance demands which should be explicitly addressed include:

- Environmental Appropriateness—Particularly as it relates to natural salt exposure and use of deicing salts, but also as it relates to human-caused contamination. While it is usually not practical to change alignments to avoid aggressive environments, the identification of issues is key to mitigation.

C3.2.1

The terms “target service life” and “intended life” are used interchangeably herein.

While these indeterminate factors are not under the control of the Designer, it is incumbent to give thought to these issues and incorporate practical measures that can best facilitate changed demands and bridge management policies.

The single most effective step that a Designer can take to improve the durability and extend the service life of a bridge is to minimize the number of joints in a bridge, with complete elimination, where possible, as the ultimate goal.

Optimum design for service life also entails elimination of as many vulnerabilities to premature deterioration as practical. In general, this is achieved in part through elimination of as many elements as possible, such as numbers of intermediate substructures consistent with clearance constraints, elimination of as many moving parts consistent with accommodating thermal movement requirements, and protecting the remaining elements and components. Future widening of pier caps and possible addition of columns should be facilitated.

- **Drainage Control**—The collection and disposal of water from the roadway is not only a major safety issue, it is also a key process in reducing contaminant exposure, particularly salt exposure, and should be thoroughly considered. Key design features that should be considered include grades and cross slopes and resulting ponding, reduction or elimination of roadway joints over intermediate supports and abutments, elimination of moisture entrapping details, control of spray from downspouts if used to eliminate or minimize contact with supporting superstructure and substructure elements, and differential movements. Drainage behind walls is vital to serviceability and preventative maintenance.
- **Material Selection**—Once the environmental and drainage issues are understood, their relation to the selection of basic materials, e.g., selection of a steel or concrete bridge, and the refinement to materials, e.g., selection of corrosion resistance level of structural or reinforcing steel or design of concrete mixes, should proceed through the evaluation of cost for life enhancement or maintenance reduction.
- **Design Efficiency**—Adaptability to changing functional demands should be considered within the budgetary constraints of the project, such as incorporating details that facilitate future widening, added lanes, conversion to high speed rail or mass transit, and increased waterway opening.
- **Resiliency to Climate Change**—The design should include considerations for adaptation to local climate changes and anticipated impacts.
- **Substructures and Foundations**—The type and location of substructures and their foundations should be decided upon with an eye upon both the established information and potential future factors discussed above. The type, configuration, and location of intermediate substructures should be selected considering the feature crossed.

As noted in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2020) Article C2.5.2.1.1, other than the deterioration of the concrete deck itself, the single most prevalent bridge maintenance problem is the deterioration of beam ends, bearings, pedestals, piers, and abutments due to percolation of waterborne road salts and moisture in general through the deck joints. Experience appears to indicate that a structurally continuous deck provides the best protection for components below the deck. See also Article 2.6.6 and the Articles thereafter in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2020) regarding roadway drainage. Even at this early stage of project development some drainage control details may be identified for use as the project is designed such as drip notches; water flow interruption plates; type of deck drains, if needed; and the location to which deck drainage is to be conveyed and disposed of consistent with environmental commitments. Cold joints in concrete substructure and retaining walls should be protected from water intrusion, particularly salt water intrusion, by barriers such as water stops or by sloping of adjacent ground or concrete to convey water away from the joint.

At a minimum, the strength of exterior beams should not be less than a typical interior beam in the adapted configuration and the stiffness of exterior and interior beams should be similar. Where such adaptive reuse is deemed to be likely, consideration should be given to providing suitable foundations as part of the original construction, especially in congested or environmentally sensitive areas.

For stream crossings, debris collection potential, chemical and particulate load, and abrasiveness of flow should be considered. For roadway and railway crossings, collision potential, safety standards for side clearance, construction clearances, and aesthetic commitments are known factors which should be considered, while future expansion of facilities, over and under the bridge, are unknown.

Provisions for additional lanes or tracks, and increased stream discharge, all based on the potential for future growth, should be considered and weighed against the budgetary constraints of the project. Foundation type and configuration should be selected based upon the existing geological conditions, slope stability, scour, roadway or railway expansion potential, environmental commitments and constraints, as well as the potential for uniform and differential settlement which may lead to cracking of concrete and other serviceability issues. For abutments, the same factors are applicable and in the special case of integral abutments the thermal movement demands must be added.

- **Use of Approach Slabs**—Where practical, approach slabs should be used to reduce or eliminate the “bump before the bridge,” reducing impact to bridge end details such as expansion joints and bearings, and to improve rideability. They also help to control drainage by conveying water away from the bridge end.
- **Component Maintenance and Inspection Demands**—Access to all elements and component parts for the purpose of maintaining, inspecting, and component replacement is of great importance to meeting the service goals of the bridge.
- **Construction Inspection**—The level of construction inspection should be considered during project scoping and throughout the design process. Any unique features of the design requiring inspection and documentation beyond typical for similar projects should be identified in the contract documents.

While careful efforts to use robust and quality materials and component parts are keys to achieving a given target service life, it would be unrealistic to assume that a bridge will remain maintenance-free. Likewise, it is of great importance that all component parts be as accessible as possible for periodic inspection to the levels specified under the National Bridge Inspection Standards and that reasonable provisions be made to facilitate replacement of bearings, roadway expansion devices, and other components identified as replaceable.

No amount of planning, design, or preparation of contract plans and specifications to improve service life can guarantee that the goals will be met without competent, conscientious, and strict construction oversight. Unless the specified level of quality of materials and proper installation of component parts are met in the field, the service life goals will not be achieved.

3.2.2—Global Design

3.2.2.1—Interaction of Bridge Components

Global design decisions should consider that one component can affect the performance of other components, e.g., leaking roadway expansion joints can damage all adjacent components: beam ends, diaphragms, cross frames, bearings, and surfaces of supporting substructure elements. Integral abutments should have robust connections between superstructure components and backwalls, abutment beams to any piles, and wing walls to back walls and abutment beams. Areas adjacent to component parts that are expected to move should be scrutinized to assure adequate clearance, etc.

C3.2.2.1

Examples of consequences from unintended interactions have included:

- Damaged beam ends, anchorages, and supporting concrete caused by frozen expansion bearings;
- Damage to underlying components of the superstructure and substructures resulting from poor provisions that do not properly convey and dispose of deck drainage; and
- Splitting, delamination, and general deterioration of concrete components caused by inadequate protection of reinforcing steel.

3.2.2.2—Plan for Component Replacement

While all components of a bridge would ideally last for the lifespan of the bridge as a whole, some components cannot practically or economically be designed or constructed for long service lives, and need to be replaced, possibly several times, over the service life of the bridge. The design of the bridge shall accommodate this replacement and facilitate the process to the extent possible. The Owner, possibly in conjunction with the Designer, should identify which components should be considered replaceable and establish their intended service life. At a minimum, the following elements should be designed to be replaceable: cast in place and precast non-pre-stressed concrete decks, bearings, roadway expansion joints, guardrails, approach slabs and sleeper slabs, traffic and pedestrian barriers, light poles, sign structures, drainage elements, cable stays and hangers, and noise walls. Overhead sign gantries should be designed for an extended service life. Provisions for replacement of these components should be incorporated into the design details, such as jacking points and redundant cables or cable paths. For most projects, it should not be necessary to fully design for replacement at the original design stage; rather it will usually be sufficient to know that the structure can accommodate the anticipated temporary staging operations. Where it is not expected to be able to take a bridge out of service for replacement of components such as slabs and roadway expansion devices, viable stage removal in the presence of reduced traffic lane operation, acceptable to the Owner Agency, should be conceptualized and clustering of stud shear connectors and stirrups projecting from beams into slabs should be employed to the extent possible. Access to work areas for replacement of components should be verified. Consideration of replacement should include worker safety and public safety, cost, and disruption.

C3.2.2.2

This list of replaceable elements, while not exhaustive, was compiled from a review of requirements from several states and international sources. The anticipated replacement interval of the following components should be included in the documentation requirements of Article 3.4:

- Conventional decks (e.g., ordinary portland cement concrete decks with uncoated steel reinforcement, no wearing surface or other surface treatment, and without enhanced crack control measures), cast-in-place and precast decks
- Bearings
- Strip seals
- Expansion joints
- Guardrails
- Traffic and pedestrian barriers
- Sign structures
- Paint and other coating systems
- Approach slabs
- Sleeper slabs
- Deck overlays

Decks may be designed to be either renewable or nonrenewable, depending on the bridge type. It is recognized that for certain bridge types the deck cannot practically be replaced (e.g., post-tensioned box girders). In these cases, the deck needs to be treated as a nonrenewable element, and appropriate service life design measures taken to ensure the target service life is met. For other structures where deck replacement is practical, the designer should consider the cost of deck replacement, and the disruption to traffic such activities entail when selecting a target service life.

Bearing replacement in-kind is an example of a component for which minimal engineering, other than that indicated in the original contract documents, would normally be needed. An example where more detailed engineering for replacement is necessary is stay cable replacement which is typically fully evaluated at the original design stage.

3.2.2.3—Total Bridge Service Life

In the process of setting the target service life of a bridge, it is imperative to assess the service life of its individual components and their potential effect on the service life of other components. For components that are often replaced at the same time, or when the replacement of one component results in the replacement of other components, the target service lives of these components should be set such that the end of service life occurs concurrently. This limits the undesirable replacement of components prior to the end of their service lives, and results in a more economical design.

3.2.3—Deterioration Experience

Guidance presented in subsequent Articles of this Guide Specification will discuss in detail deterioration mechanisms inherent in various materials used in the component parts of bridges; measures to prevent, delay, and mitigate deterioration of the constituent materials; as well as protection strategies that should be implemented to protect the resulting component from the rigors inherent in its role as part of the bridge.

3.2.3.1—Drainage

As stated in Article 3.2.1, drainage, particularly proper collection and discharge of roadway drainage, is one of the most important service life design issues and should be considered as a key to reducing maintenance needs. Drainage at the deck level should involve one or more of the following: appropriate deck joints, under-joint troughs, piping, scupper, and parapet slots.

C3.2.2.3

This exercise involves consideration of how many times the various components (e.g., decks, bearings, roadway expansion devices) need major rehabilitation and/or replacement; and how many cycles of other protection strategies, such as repainting of structural steel or application of sealants to vulnerable concrete surfaces will be needed over the target service life desired. As an example, consider the case of a reinforced concrete deck supported by a steel girder superstructure. If the target service life of the girders is 100 years, then the target service life of the deck should be set to match, or set in factors of 100 years, since the replacement of the superstructure requires the deck to be replaced simultaneously. This could be achieved with a deck service life of 100 years or more realistically with two deck replacements, one at 50 years and one at 100 years.

The following sections of this Guide Specification will discuss in detail how to estimate the expected lives of these components under defined environmental conditions and current technologies/strategies for manufacturing of the basic materials used in these components. Various levels of protection strategies available to extend component service life employed at the time of construction will be presented in a three-tiered format such as “Good, Better, Best” where practical.

C3.2.3

The following Articles will draw upon important lessons learned from past practice, strategies for improvement that have proven to be unsuccessful as well as identifying the most common continuing issues that currently limit service life, and current best practices to extend service life. These issues will be presented by material type and components as applicable. Examples include what materials are constituent parts of concrete and how are they best combined to produce a durable mix; how can the constituent parts of the concrete mix be optimized to reduce chloride ion penetration and shrinkage cracking so that reinforcing steel corrosion is retarded; and what protection strategies can be added to further retard or eliminate corrosion by protecting the reinforcing steel itself. Unless noted otherwise, photographs used in this Article to illustrate deteriorated conditions of bridge components are from the bridge inspection files of Modjeski and Masters, Inc.

C3.2.3.1

Typical types of maintenance issues on components of a bridge drainage system are shown in Figures C3.2.3.1-1 through C3.2.3.1-4.

Experience has shown that no perfect all-purpose drainage solution is available; all systems are prone to leakage and clogging and should be assumed to require periodic cleaning and repairs and be detailed accordingly. The use of jointless bridges eliminates some of the components of the drainage system, usually the deck joints and associated troughs and piping, but the diverted flow increases the demand on scuppers or parapet slots, which must be considered. Experience has shown that this trade-off is beneficial to service life and reduced maintenance. See Articles 3.2.3.3, 3.3.3, and 3.3.4 for more information on joints and bearings.

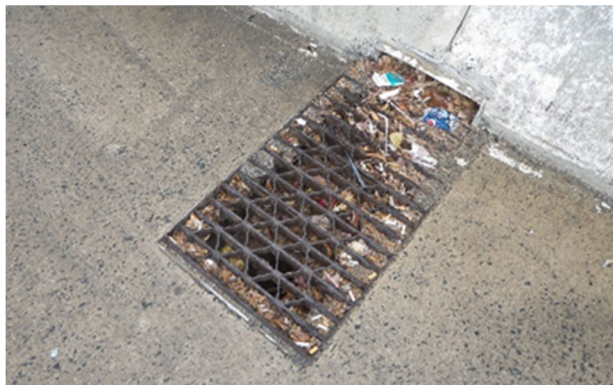


Figure C3.2.3.1-1—Scupper Grate and Parapet Drain Slot Clogged with Debris



Figure C3.2.3.1-2—Damage to Concrete at Free Drop Deck Drains



Figure C3.2.3.1-3—Drain Pipe beneath a Joint Clogged with Debris



Figure C3.2.3.1-4— Clogged and Leaking Drain (FHWA, 2015)

3.2.3.2—Decks

Selection of the type of deck system to be employed in the design of a bridge including protection methods, detailing, and materials should be considered as fundamental in determining how the resulting stresses from load application are distributed and therefore has an influence on the target service life.

C3.2.3.2

Deck systems for which the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2020) provide design guidance are: timber, concrete, steel grid, and aluminum and steel orthotropic decks. By far, the most common is the concrete deck system. The primary focus in this document is on the concrete and steel orthotropic systems. Service life issues dealing with concrete decks will be discussed in Section 4 and steel orthotropic deck issues will be discussed in Section 5. Fiber-reinforced polymer (FRP) bars are becoming more widely used as

concrete deck reinforcement by several state DOTs. Guidance is provided in the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings* (AASHTO, 2018). Pultruded FRP deck sections have been used in recent years, however deployments of these deck systems in the United States has been limited to special circumstances such as rehabilitation of moveable bridges and demonstration projects.

Typical types of deterioration found on concrete deck components are shown in Figures C3.2.3.2-1 through C3.2.3.2-4.



Figure C3.2.3.2-1—Deck Repairs Required by Delamination and Potholes Attributed to Corrosion of Reinforcement



Figure C3.2.3.2-2—Delamination of Underside of Deck from Either Water from Above Penetrating Cracks or from Salt-Laden Spray from Roadway Below



Figure C3.2.3.2-3—Ponding on a Deck Surface Near a Scupper



Figure C3.2.3.2-4—Cracked and Spalled Concrete at a Damaged Deck Joint

3.2.3.3—Joints and Bearings

Historically, joints and bearings have been among the most problematic components on bridges. Therefore, the selection of configuration, materials, and construction procedures should be based on best local practices.

C3.2.3.3

This problematic experience has often been associated with drainage system problems and the direct impact of truck tires and snow plows. Joints have been subject to broken or loose sliding plates and finger plates including fatigue damage to teeth and hold-down bolts and studs, clogging, dislodging or failure of seals,

unequal movement of support bars, and fatigue of internal welds. Malfunctioning joints and bearings usually apply unintended force to other parts of the bridge due to restrained movement from temperature change and traffic which can result in cracking, anchor bolt damage, or other service problems.

A recent domestic scan on bearings and joints reported in Stanton et. al. (2018) lists experiences and lessons learned by participating states.

Table 14.6.2-1 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2020) lists 14 types of bearings which have been used on bridges at various times, some of which may no longer be available.

Unsatisfactory experience such as that shown in Figures C3.2.3.3-2 and C3.2.3.3-4 has generally led to the near elimination of steel rocker and roller bearings in new bridges.

There has been increased usage of elastomeric bearing pads, and steel reinforced elastomeric bearings in particular. Stanton et. al. (2018) documents some early issues with elastomeric bearings, particularly “walking” which has been associated with wax in the elastomer and with natural rubber. It is believed that this issue has been largely eliminated by changes in composition as well as the use of bonding of the pad to the superstructure and the use of vulcanizing. Stanton et. al. (2018) also indicated that fiberglass reinforced elastomeric bearings are no longer available, that the future availability of neoprene requires attention due to changes in the supplier, and that natural rubber may become more attractive as an elastomer due to changes in processing.

Where reaction loads are heavy or complex structural geometry requires consideration of rotations about multiple axes, High Load Multi-Rotational (HLMR) bearings such as pot, disc, or spherical bearings should be considered.

Pot bearings have been prone to ejection of the elastomer disc, although improvements in materials and fabrication have resulted in better recent performance (Stanton et. al., 2018).

Base isolation bearings have unique behaviors that can make them especially effective, particularly for seismic rehabilitation.

Section 7 contains more guidance on the selection and application of joints and bearings.

Typical types of deterioration found on joints and bearings are shown in Figures C3.2.3.3-1 through C3.2.3.3-4.



Figure C3.2.3.3-1—Damaged Strip Seal Joint Clogged with Debris



Figure C3.2.3.3-2—Corroded Steel Bearing Attributed to Leaking Joint



Figure C3.2.3.3-3—Tooth Plate with Missing Bolt. Loss of Bolts Can Result in a Loose Plate that May Cause Damage to Joint and Other Members



Figure C3.2.3.3-4—Rocker Bearing Failure as a Result of Excessive Leaning Caused by Build-Up of Debris and Corrosion between the Bearing Bottom Surface and Bearing Plate

3.2.3.4—Concrete Structures

Designs should consider the deterioration of concrete bridges and bridge components including those historically related to:

- material issues such as high permeability, reactive aggregates, insufficient air entrainment, and improper W/CM ratio for the application;
- aggressive environment factors such as sulfates, sulfur, acid, sea water exposure, and carbonation;
- freeze-thaw cycles;
- water and salt intrusion, leading to chloride-induced corrosion, particularly exacerbated by poor roadway drainage control, high concrete permeability, and extensive cracking;
- abrasion due to ice or traffic;

C3.2.3.4

- detailing issues such as inadequate cover, construction joints located in zones of vulnerability, or inadequate crack control measures;
- underestimated force effects such as creep and shrinkage, settlement, particularly differential settlement, unanticipated or underestimated restraint and torsion, and thermal effects that lead to extensive cracking; and
- poor construction quality control, inadequate construction means and methods.

Design and details should consider combinations of causes such as cracking caused by shrinkage, thermal effects, and load effects leading to accelerated chloride penetration attack of reinforcement.

The provisions of this Guide Specification may be augmented by applicable provisions in Sections 2 and 5 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2020) related to self-protection, drainage control, detailing, and durability; the *AASHTO LRFD Bridge Construction Specifications* (AASHTO, 2017); the various AASHTO material specifications; and Owner-supplied requirements. ACI 201 (ACI, 2001) provisions may also be used to supplement the AASHTO and Owner supplied documents as appropriate for bridge applications. See further discussion in Article 4.2.

Such combinations of causes have been experienced and may be more common than deterioration from single isolated causes.

Current material, design, and construction specifications provide guidance intended to avoid many of these historical shortcomings or treat them through deemed-to-satisfy provisions.

Typical types of deterioration found on concrete components are shown in Figures C3.2.3.4-1 through C3.2.3.4-8.



Figure C3.2.3.4-1—End of a Prestressed Concrete Beam with Corroded Strands that Have Caused Cracking and Spalling of Concrete



Figure C3.2.3.4-2—Ends of Prestressed Concrete Beams Showing Delaminated Concrete and Exposed, Corroded Strand Attributed to Exposure to Roadway Drainage



Figure C3.2.3.4-3—Cracked and Delaminated Concrete with Corroded Reinforcement on End of Pier Cap under Roadway Joint



Figure C3.2.3.4-4—Spalled Concrete and Corroded Reinforcement at End of Pier Cap. Condition Shown in Figure 3.2.3.4-3 May Reach this Stage If Not Addressed



Figure C3.2.3.4-5—Cracked Concrete Abutment Attributed to Reactive Aggregate



Figure C3.2.3.4-6—Crazeing of Concrete Pier Cap under Roadway Joint Attributed to Reactive Aggregate



Figure C3.2.3.4-7—Concrete Surface Scaling Attributed to Freeze–Thaw

3.2.3.5—Structural Steel

Historically, corrosion and fatigue damage have been the most common serviceability issues with steel bridges and should be considered in the design process. Brittle fracture has also been observed, usually resulting from a fatigue crack or triaxial restraint due to improper weld details, and should also be considered. Design actions for improving service life issues related to corrosion and fatigue discussed in Section 5 should be considered, as appropriate.

Paint does not adhere well to sharp edges, so they should be rounded slightly, which is sometimes referred to as “breaking” the edge. Proper preparation and inspection, especially in repainting, should be provided to improve paint longevity.



Figure C3.2.3.4-8—Deteriorated Concrete Footing with Exposed Reinforcement. Possible Sources Include Reactive Aggregate and Freeze–Thaw.

C3.2.3.5

Additional guidance on steel durability considerations is provided in Volume 19 of FHWA (2015), available for download from <https://www.fhwa.dot.gov/bridge/steel/pubs/if12052/>.

Fatigue cracking has been essentially eliminated in bridges designed using the fatigue provisions adopted in the early 1970s and fabricated to standards in place since that time, but may still be an issue in older bridges.

Experience with older steel bridges has shown the need to avoid crevice corrosion through adherence to sealing and stitching requirements specified in Articles 6.13.2.6.2 and 6.13.3.7 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2020), as well as caulking of crevices during fabrication. This is especially important in areas where salt spray and snow plow discharge can settle on structural members.

Many older bridges, particularly those of riveted construction, do not have adequate sealing of built-up sections and experience crevice corrosion that can ultimately lead to their replacement. There are few effective remedies once crevice corrosion reaches an advanced state.

Current code-based guidance generally focuses on:

- prescribing specific coatings for various exposure classes;
- the estimation of section loss in areas that either cannot be accessed or cannot be reliably protected from corrosion;
- accessibility to provide access to inspect, repair, etc.; and

Guidance in Technical Advisory 5140.22, (FHWA, 1989), may be used to assess the suitability of weathering steel for a use in a given location, as well as a source of detailing suggestions. See also Article 5.2.1.2.

- geometry requirements to avoid “corrosion traps” and to provide for drainage.

Some problematic experience with early applications of weathering steel used in situations which interfered with the development of proper patina is well documented.

Typical types of deterioration found on structural steel components are shown in Figures C3.2.3.5-1 through C3.2.3.5-14.



Figure C3.2.3.5-1—Corroded Beam Ends and Bearings Under Leaking Deck Joint and Splash from Drain Pipe



Figure C3.2.3.5-2—Local Corrosion and Paint Failures Within a Splash Zone



Figure C3.2.3.5-3—Crevice Corrosion Resulting In Distorted Gusset Plate, and Local Corrosion and Paint Failures



Figure C3.2.3.5-4—Debris and Moisture Traps Formed by Bearing Stiffeners without Drain Holes (FHWA, 2015)

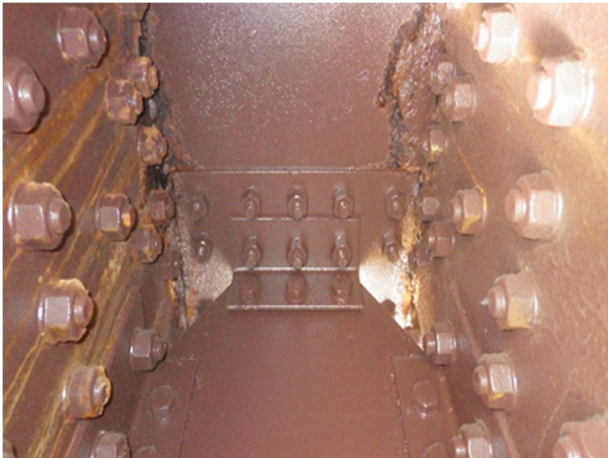


Figure C3.2.3.5-5—Moisture and Debris Resulting in Local Corrosion in Box Truss Member



Figure C3.2.3.5-6—Trapped Moisture within a Box Girder Resulting in Local Corrosion



Figure C3.2.3.5-7—Debris Collected Along Horizontal Web of I-Shaped Truss Members, One Welded and One Riveted Built-Up Member. Note No Drain Holes Evident in Picture.



Figure C3.2.3.5-8—Debris on Lateral Gusset Plate Leading to Local Corrosion and Section Loss



Figure C3.2.3.5-9—Moisture and Debris on Lateral Gusset Resulting in Corrosion and Section Loss



Figure C3.2.3.5-10—Bird Droppings which Entrap Moisture, Often Resulting in Local Corrosion and Section Loss



Figure C3.2.3.5-11—Localized Paint Failures and Corrosion on Built-Up Truss Member



Figure C3.2.3.5-12—Loss of Patina on Weathering Steel Girders Leading to Corrosion



Figure C3.2.3.5-13—Leaking Deck Joint Leading to Loss of Patina and Corrosion of Weathering Steel



Figure C3.2.3.5-14—Fatigue Crack at the Bottom of a Connection Plate

3.2.3.6—Foundations and Retaining Walls

Design of every foundation or retaining wall system should consider the desired level of serviceability depending on the environment, the consequences of adverse performance, and whether the structure is considered to be a critical part of an overall facility.

Most foundation elements are buried in geomaterials. Similarly, most retaining walls are in contact with geomaterials. Thus, the long-term performance of the foundation and retaining walls can be affected by the environment provided by the geomaterials, and this should be considered in the design process. Consideration of the effect of deterioration of foundations and wall elements through corrosion or degradation on serviceability is important and should be considered in the design process.

The assumed settlement used in the design process should be selected with consideration of the sensitivity of the whole system to total and differential settlement.

Several approaches to ensure desired performance of foundations and retaining walls are well established in the industry and should be considered. The most common approach is to estimate the rate of deterioration

C3.2.3.6

These factors are used to select a Protection Index for use in design. See Article 6.1 for further discussion.

The avoidance and deemed-to-satisfy approaches are commonly used for foundations and retaining walls.

The term “corrosion” applies to metal components, and “degradation” applies to nonmetallic components such as polymeric soil reinforcements in mechanically stabilized earth (MSE) walls.

The total and differential settlement experienced by foundations and walls can also affect the serviceability and maintenance requirements of the supported components including structure, embedded elements, and fill and roadway.

Intended service life may or may not be equal to design life. The quantification of intended service life, to be done at the Owner level, is a function of many factors including, but not limited to, maintenance protocols,

(commonly corrosion or degradation) over the intended service life of the structure due to ground (geomaterial) environment and provide additional sectional or strength properties (or both) that will permit the structure to perform within its strength and serviceability requirements. Other approaches include use of coatings, encapsulation in various materials such as concrete and plastics, use of an appropriate cement type, controlling crack width, etc.

The approach used to enhance service life for a particular foundation or wall type is often a function of the specific industry and historical use and these characteristics should be considered where indicated by successful local experience.

choice of Protection Index, P_i , etc. which is treated in Article 6.1 and elsewhere in Section 6.

For example, ground anchors typically rely on grout cover and encapsulation, drilled shafts usually rely on cover and field verification of cover, while metallic reinforcements in MSE walls use zinc coatings and sacrificial steel.

Typical types of deterioration found on foundation and wall components are shown in Figures C3.2.3.6-1 through C3.2.3.6-7.



Figure C3.2.3.6-1—Corrosion of Steel H-Pile Taken out of Service (Photo Courtesy of FHWA)



Figure C3.2.3.6-2—Corroded Steel Soil Reinforcing Strip Exposed behind MSE Wall Attributed to Runoff Containing Deicing Salt. A Membrane is Now Required at Top of Reinforced Fill to Address this Situation (Photo Courtesy of FHWA)

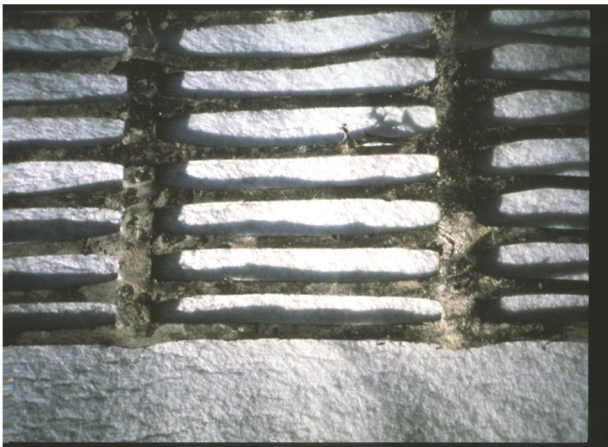


Figure C3.2.3.6-3—Damage to Extruded Polyethylene Geogrid (Photo Courtesy of FHWA)



Figure C3.2.3.6-4—Joint Leakage Due to Differential Settlement (Photo Courtesy of NCS GeoResources, LLC)



Figure C3.2.3.6-5—Outward Movement of Wing Wall Leading to Reinforcement Pull-Out and Corrosion (Photos Courtesy of NCS GeoResources, LLC)



Figure C3.2.3.6-6—Excavated Drill Shaft Showing Cover Defect (Photo Courtesy of NCS GeoResources, LLC)



Figure C3.2.3.6-7—Close-Up of Cover Defect (Photo Courtesy of NCS GeoResources, LLC)

3.2.3.7—Utilities and Appurtenances

Experience has shown that utilities and appurtenances and their attachments are often debris traps, bird roosts, and other moisture retention points. These can result in spalling and cracking of concrete and premature paint failures and corrosion of steel. The location and details of these items should be selected and implemented to avoid these issues.

C3.2.3.7

Utilities such as water and sewer lines, gas lines, and communication conduits, and appurtenances such as traffic and pedestrian railings, sign supports, light poles, and noise walls usually have attachments to main bridge components such as deck slabs, girders, or barriers. These items are often added after the bridge is in service using details that are less well-conceived and -executed than the original construction.

Typical types of deterioration found on utilities and appurtenances are shown in Figures C3.2.3.7-1 through C3.2.3.7-4.



Figure C3.2.3.7-1—Deflection of a Utility Line at a Deck Joint Causing Damage to the Utility Fitting



Figure C3.2.3.7-2—Local Cracking and Corrosion in the Area of an Electrical Box within a Parapet Causing Staining of the Concrete



Figure C3.2.3.7-3—Parapet with Spalled Concrete at the Anchor Bolts Attaching a Pedestrian Fence



Figure C3.2.3.7-4—Corrosion of Steel Railing Due to Lack of Paint Maintenance Resulting in Section Loss

3.3—GENERAL DURABILITY CONSIDERATIONS

It cannot be over-emphasized that the Designer's meticulous attention to the details presented in the contract documents should be considered as equal in importance to the design itself to assure that the various components of the bridge are combined to achieve the desired result and to achieve the target service life.

3.3.1—Drainage

Drainage systems should be designed and detailed to minimize blockage and facilitate cleaning. The design should consider that closed drainage systems, drop inlets, open drain pipes, and even slot opening drains through curbs and parapets will clog unless properly maintained.

C3.3.1

For closed systems, adequate and accessible cleanout plugs are required. For drop inlets, removable grates are needed unless easy flushing ability exists. Regardless of the type selected, the outlets should be located such that the resulting discharge does not expose, or at least minimizes exposure to, underlying super and substructure components.

For additional drainage design criteria, see the *AASHTO Drainage Manual* (AASHTO, 2014), *AASHTO's A Policy on Geometric Design of Highways and Streets* (AASHTO, 2018), and *Bridge Deck Guidance* (FHWA, 1986).